A Study on Geotechnical Engineering Characteristics of Soft to Firm Clays in Yangon

（ヤンゴンにおける粘性土の地盤工学特性に関する研究）

D155086

Yoshimitsu Yamada

（山田　義満）

Dr. THESIS

Supervisor: Professor Takashi Tsuchida

Hiroshima University

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ABSTRACT

In Myanmar, democracy has progressed more than before, and a number of foreign investors have entered into Myanmar's social infrastructure market since the power change in general elections took place in 2010. Many construction projects are underway, especially in the Yangon area. During this construction boom, research on cohesive soils such as soft clay widely distributed in Yangon that cause problems in the design and construction of structures are very few in existing studies. Therefore, the information of the soil properties of each project is not shared, and it is limited to the handling in each project site only. These information sharing is considered to be very important for the engineers in order to promote the project smoothly in the planning and implementation in the future. Accordingly, it is thought that information sharing is very significant.

In addition, such uncertainty of ground information is one of the major risks especially in overseas construction projects and sharing ground information is also very important to reduce the hidden risks in overseas construction projects. **This thesis is the first comprehensive research result in Myanmar on geotechnical engineering characteristics of cohesive soil in Yangon.**

In this paper, the following items have been studied.

1) The past researches on geotechnical engineering characteristics of cohesive soil
2) The effects of sample disturbance due to different sampling methods with regard to the quality of undisturbed samples of cohesive soil
3) Comparison and study of geotechnical engineering characteristics of cohesive soil in Yangon (Grasping the sedimentary condition of clays, comparison of soil characteristics in different sedimentary basins, comparison of soil characteristics among 7 sub-areas in Yangon and marine clays of other countries
4) Geotechnical engineering characteristics of soft clay in Thilawa area along the Yangon River where typical soft ground in Yangon is distributed

In this paper, from the results of boring and laboratory tests conducted in Myanmar, the ones conducted in the Yangon area are selected, focusing on the Yangon area, and the soil properties of the cohesive soil are described. The test data used in this paper is the laboratory

test of clay collected by the boring and sampling methods defined by the Japan Geotechnical Engineering Society (JGS) standard.

The list of the above examination in this paper is summarized as follows.

1) Past researches on geotechnical engineering characteristics of cohesive soil

The past researches on this paper are classified into the following five categories, and the past research results are summarized for each classification.

1) Study on the correlation of each parameter of physical property and mechanical property of cohesive soil
2) Study on the shear strength and the shear strength ratio of cohesive soil
3) Study on the coefficient of secondary consolidation of cohesive soil
4) Study on minerals and physical properties of clay
5) Study on the quality of undisturbed sample of cohesive soils, in particular on the effect of disturbance due to the differences of samplers

2) Effects of sample disturbance due to different sampling methods with regard to the quality of undisturbed samples of cohesive soil in Yangon

Here, the quality of the sample is discussed, which is a major premise of this study. The clay sample used this time was collected by a sampler for undisturbed sampling, but in past studies it has been pointed out that the quality of undisturbed samples differs due to differences in samplers and is affected by disturbance.

In Myanmar, a Shelby tube sampler, which is generally defined by the ASTM standard, is mainly used as a mainstream, but so far, no attention has been paid to the effect on the disturbance of undisturbed sample of cohesive soils. On the other hand, in Japan, undisturbed sampling with a fixed piston sampler has already been standardized, and undisturbed sampling with a Shelby tube is hardly seen. Therefore, with regard to how much disturbance affects the samples collected by the different samplers, it was clarified for Yangon clays. Samples were actually collected using a fixed piston sampler and a Shelby tube sampler at a total of three points in Yangon and its suburbs. The clarification of the difference in the disturbance degree, and the comparison with the case of Thi Vai clay collected in the estuary of the Mekong Delta in Ho Chi Minh, which is very soft clay, and the soft clay of Hachirogata in Japan, were carried out using such past study results.

As a result, differences in the effects of disturbance due to differences in samplers are not as large as those of Thi Vai soft clay in Vietnam and marine clays in Japan. That is, clays in Yangon and its suburbs, although the degree of difference in disturbance varies depending on the state of clays, the sample quality obviously affects the results of unconfined compression tests, and the sample with the fixed piston sampler is less affected by the disturbance. It was also found that the disturbance in quality of the sample appears depending on the stress ~ strain relationship that the undisturbed sample has.

3) Comparison and study of geotechnical engineering characteristics of cohesive soil in Yangon

Here, the characteristics of cohesive soils in the whole Yangon area is described. In the evaluation of the ground in the whole Yangon area, as Yangon is located at the boundary between two basins (Irrawaddy Delta Sub-Basin, Pegu-Yoma Sittaung Basin), whether the soil properties differ depending on the basins or not are confirmed. In addition, Yangon area was divided into seven sub-areas centered on the Central Sub-area consisting of Tertiary sediments that runs the central part of Yangon from north to south and clarified the differences in soil properties among sub-areas. Furthermore, comparison with the average value of soil properties of the clay of whole Myanmar and with the marine clays of other countries were examined.

As a result, it became clear that there was little difference in the soil properties between two basins and between sub-areas except for the Central sub-area with different ground structures. However, in comparison with physical properties of foreign clays with completely different sedimentary environments, Yangon clay has a little large over-consolidation ratio (OCR = 1.6) and a large unit weight (average 18 kN/m^3), a small natural moisture content (average 40 %). Thus, it was confirmed that there was a clear difference in soil properties from clays in other countries. Moreover, in comparison of mechanical properties, by comparing the relationship between unconfined compressive strength q_u and consolidation yield stress p_c, q_u of Yangon clay is about 60% of the value of marine clay in Japan for the same p_c. One of the reasons of this is thought to be due to the effect of stress release by over-consolidation, as Yangon clay has a little higher over-consolidation ratio than Japanese marine clay. Furthermore, when the compression index C_c of Yangon clay is compared in the case of an average water content ratio of 40% of Yangon clay, C_c of Yangon clay is smaller as well as 5/9 of Japanese marine clay and 2/3 of Vietnam's Thi Vai clay. Finally, in the relationship between the coefficient of consolidation c_v and the liquid limit w_L, although the distribution of c_v varies widely, the c_v of Yangon clay is smaller than Japanese marine clay and Ho Chi Minh clay for the same liquid limit w_L. Therefore, the time required for Yangon clay to complete a required consolidation degree will be four times that of Japanese marine clay and 1.3 to twice that of Ho Chi Minh clay.

4) Geotechnical engineering characteristics of soft clay in Thilawa area along the Yangon River where typical soft ground in Yangon is distributed

Among the Yangon area, a homogeneous and soft cohesive soil is thickly deposited in Thilawa area (thick up to about 20 m), and the soil characteristics of soft clay of the Thilawa area distributed along the Yangon River in the Thanlyin Sub-area were clarified here. In the study of soil characteristics of Thilawa soft clay, the comparison of physical and mechanical properties with Japanese marine clay and other countries' clays were carried out using samples collected from five locations in the Thilawa port area (about 7.5 km long along the Yangon River). In addition, examination of secondary consolidation properties and mineral compositions of clay by X-ray diffraction method were carried out, and the differences from Japanese marine clay was confirmed.

As a result, the over-consolidation ratio of Thilawa clay is a little larger than that of Japanese marine clay, and the average over-consolidation ratio (OCR) of Thilawa clay except surface layer (GL-5m to -6 m) is 1.40, while average OCR of Japanese marine clay is 1.25. Also, the relationship between unconfined compressive strength q_u and elevation (C.D.L.) of the Thilawa clay can be expressed by the same trend line and formula for clays of both on-land and in-river. This is probably implies that original ground of both in-river and on-land areas was the same flat level before, then after that the river eroded the ground and it became finally the present topography. Also, from the regression linear equation $q_u = 0.445\ p_c$ obtained from the relationships of the unconfined compressive strength q_u and the consolidation yield stress p_c from the standard consolidation test, the normalized shear strength (equivalent to the shear strength ratio) c_u/p_c of Thilawa soft clay was about 0.22. In addition, this is about 2/3 of the average value of Japanese marine clay (c_u/p_c= 0.3), and it turned out that caution is necessary when adopting a preload method etc. The friction angle of effective stress ϕ' of Thilawa soft clay obtained from triaxial compression test (CIU) is $\phi'= 25°$ which is smaller than that of Japanese marine clay, $\phi'= 30°$.

The average coefficient of consolidation c_v in the normal consolidation state of Thilawa soft clay was found to be 50 cm^2/day which is not so much different from the one of Japanese marine clay. However, focusing the case of the same liquid limit w_L, cv of Thilawa soft clays is as small as 1/6 to 1/3 compared to Japanese marine clay. However, the c_v of them is larger than Bangkok clay, Singapore clay and Thi Vai clay (Vietnam), which suggests that the average coefficient of consolidation c_v of Japanese marine clay and Thilawa soft clay are greater than that of other parts of Asia, one reason of this is that Japanese marine clay and Thilawa soft clay has a low clay content.

Also, in the long-term consolidation test of Thilawa soft clay, the coefficient of secondary consolidation $C_{\alpha\varepsilon}$ (expressed with strain) under normally consolidation state was about 0.8%, but gradient became smaller with about 1/2 from about 10,000 minutes and later.

Furthermore, although the mineral test results by X-ray diffraction method were compared between the Thilawa soft clay and Tokyo Bay clay, it was confirmed that they had substantially the same mineral composition.

ABBREVIATIONS

A'	clay activity (clay grain size ≦ 5μm)	γ_w	unit weight of water
A_f	coefficient of pore-water pressure	γ_d	dry unit weight
C_c	compression index	R_a	cross-sectional area ratio
C_s	swelling index	S_t	sensitivity ratio
c_v (C_v)	coefficient of consolidation	*SD* (σ)	standard deviation
$C_{\alpha\varepsilon}$	coefficient of secondary compression/ consolidation by strain	*S.S.*	specific surface
C (CC)	clay content	*SCP*	sand compaction pile
CDL	chart datum level	s_u (c_u)	undrained shear strength
CR (Cr)	compression ratio = $C_c/(1+e_o)$	u	pore-water pressure
CPT	cone penetrometer test	V_p	P-wave velocity
DMT	dilatometer test	V_s	S-wave velocity
e_o	initial void ratio	w_L	liquid limit
E_{50}	Secant deformation factor through 50 % of qu	w_P	plastic limit
FVT	field vane shear test	w_n	natural water content
GDP	gross domestic product	*XRD*	X-ray diffraction test
G_s	specific gravity	σ'_{vo}	effective overburden pressure
h	dumping constant	σ'	effective stress
I_P	plasticity index	σ'_r	Effective residual stress
I_L	liquidity index	σ_f	final load
k	coefficient of permeability	ϕ	angle of friction
m_v	coefficient of volume compressibility	τ	shear stress
MSL	mean sea level	τ_v	vane shear strength
OCR	over-consolidation ratio	ε	strain
p' (σ'_c)	consolidation pressure	ρ_t	wet density
p_c	consolidation yield stress	ρ_d	initial dry density
PVD	prefabricated vertical drain	ρ_s	density of soil particles
q_u	unconfined compressive strength	ρ	mass density

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A Study on Geotechnical Engineering Characteristics of Soft to Firm Clays in Yangon

Yoshimitsu Yamada

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1. INTRODUCTION

1.1 Background of Study

In Myanmar, democracy has progressed more than before, and a number of foreign investors have entered into Myanmar's social infrastructure market together with the economic growth of Myanmar since the power change in general elections took place in 2010 as shown in Figure 1-1 and Figure 1-2. Many construction projects are underway, especially in the Yangon area. During this construction boom, a great number of engineers from various countries have been joining the projects of the infrastructure developments. However, soil characteristics and properties usually have unique localities and are different from each other by regions or by countries. Therefore, civil and geotechnical engineers who came from other regions and countries have to pay attention to those localities of soils.

However, the research on cohesive soils such as soft clay widely distributed in Yangon that cause problems in the design and construction of structures are very few in existing studies. Therefore, the information of the soil properties of each project is not shared, and it is limited to the handling in each project site only. These information sharing is considered to be very important for the engineers in order to promote the project smoothly in the planning and implementation in the future. Accordingly, it is thought that information sharing is very significant.

In addition, such uncertainty of ground information is one of the major risks especially in overseas construction projects and sharing ground information is also very important to reduce the hidden risks in overseas construction projects. This thesis is the first comprehensive research result in Myanmar on geotechnical engineering characteristics of cohesive soils in Yangon.

In this thesis, focusing on the Yangon including Thilawa area which is the previous capital of Myanmar, the differences of soil properties between sub-areas divided by the Tertiary sedimentary ridge that is running from the north to the south at the center of Yangon were studied with the comparison to clays in other regions and countries.

As shown in Figure 1-3 [1)] and Figure 1-4 [2)], Yangon city area is located at southern part of Myanmar and surrounded by Hlaing River in the west, Yangon River in south and Bago River in the east. The center is a hilly area running from the north to the south and which is composed of Tertiary deposits such as sandstone and mud stone [1]. The ridge of hilly area is extending to Thanlyin and Thilawa areas in south beyond Bago River. At both east and west sides of this ridge of Tertiary deposit, soft clays which have less than 5 of SPT N-value deposit with more than 10 to 20 meters thick are distributed. Especially at the riversides along Yangon River, soft clay layers with more than 20 meters thick were identified [3].

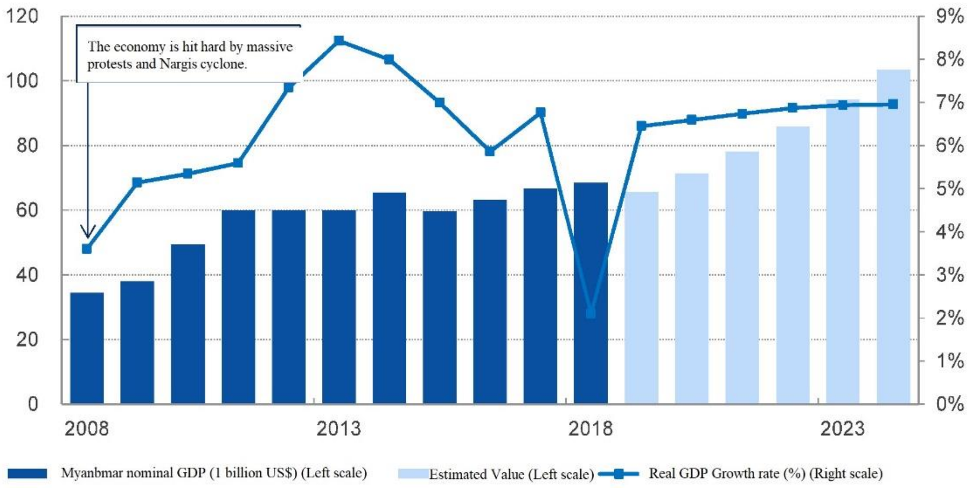


Figure 1-1 Growth rate of nominal GDP and real GDP [1]

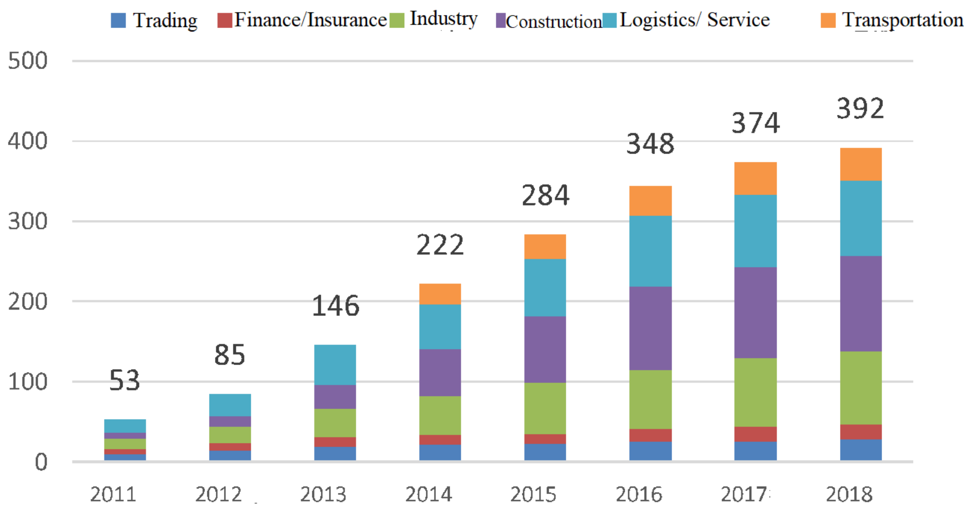


Figure 1-2 Number of members of Japan Chamber of Commerce and Industry, Myanmar [1]

Logistics in Yangon utilize the rivers surrounding the Yangon Area. Thus, the development projects for an industrial park and port projects are ongoing along those rivers. In the center area of the city, many high-rise hotels, condominium buildings, commercial complex buildings and the projects for improvement of circular railway and flyovers crossing the main roads have been planned and constructed as well. Under these conditions, investors from many countries are carrying out a lot of infrastructure projects such as Thilawa Industrial Zone

Project, Thilawa International Port Project and commercial complex building construction projects, etc. [4] as shown in Figure 1-5.

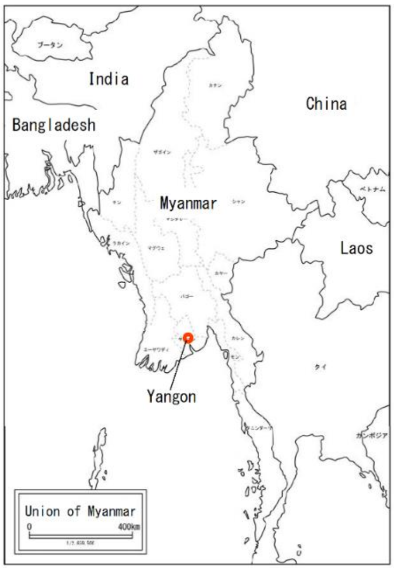


Figure 1-3 Location of Yangon [1]

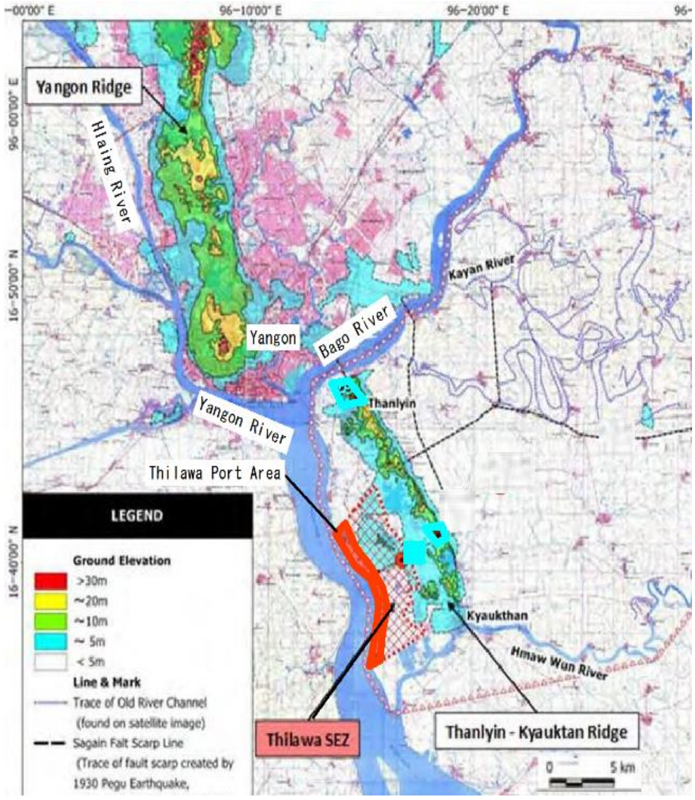


Figure 1-4 Topography and Elevation for Yangon City including Thilawa Area (Retouched to Reference [2])

Therefore, for the construction of infrastructure, it becomes more important how to deal with problematic soils such as soft clay deposit mentioned above. Especially, in the biggest city Yangon and its surrounding delta area, high compressible clays are distributed so that sharing of information on these problematic soils are very important and useful for planning of future development of Yangon area.

*1) https://www.kajima.co.jp/news/press/201711/20m1-j.htm *2) https://release.nikkei.co.jp/attach_file/0452591_01.pdf
*3) https://www.kensetsunews.com/archives/299122 *4) https://www.nikkei.com/article/DGXLRSP505313_Y9A310C1000000/
*5) https://www.rncc.co.jp/news/press/196

Figure 1-5 Planned and on-going Projects in Yangon and Myanmar

1.2 Purpose of Study

In Yangon Area, a great number of engineers from various countries have been joining the projects of the infrastructure developments. In this situation, it is very important to understand the localities on the geotechnical properties on soft to firm clays, which often become problematic soils for the construction projects. The soil properties in Yangon including Thilawa Area which are shown in this study would be useful to conduct construction projects smoothly to success without any accidents or troubles.

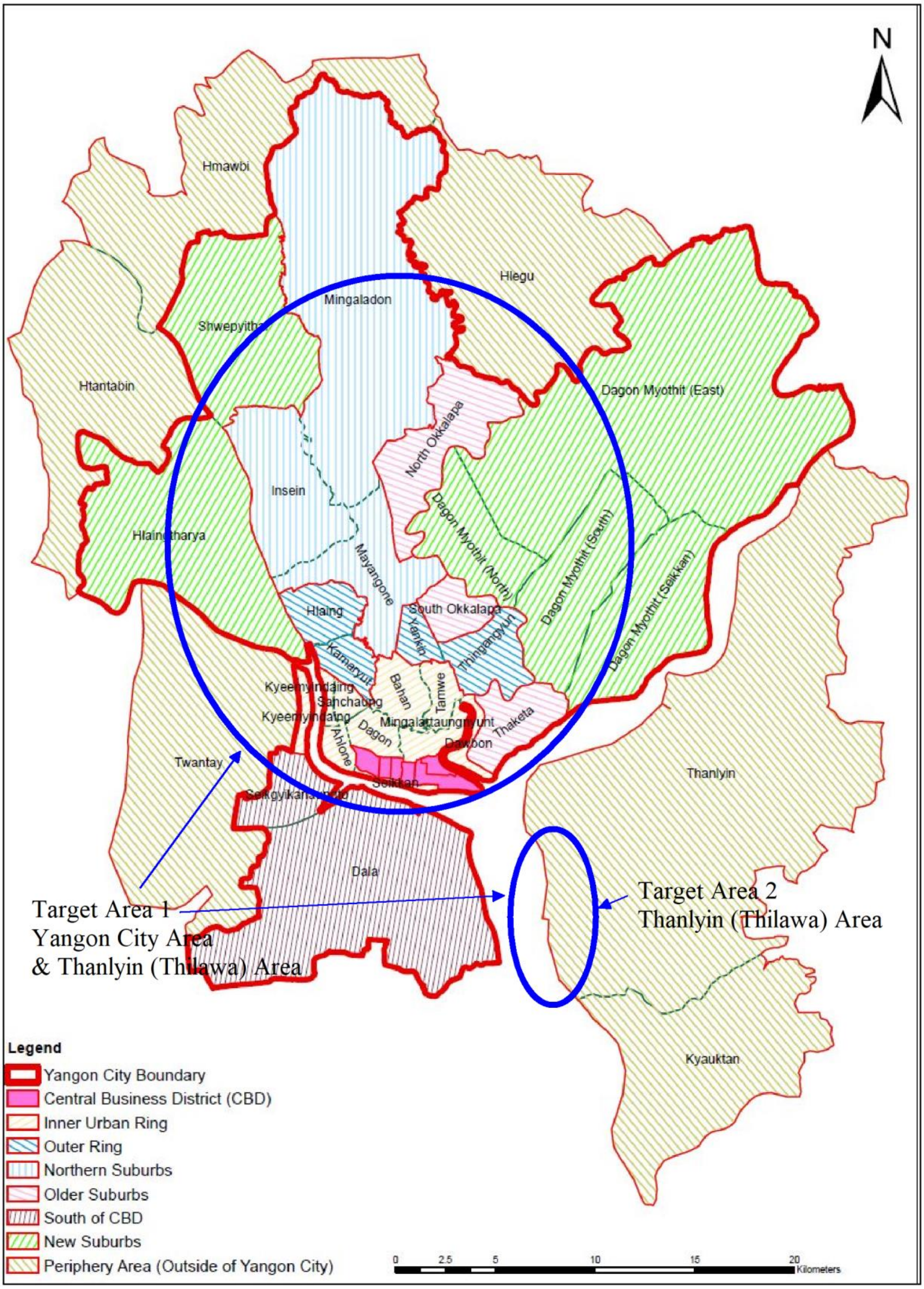
Accordingly, the following purposes are targeted for this study.

1) The soil properties such as physical properties and mechanical properties of soft to firm clays ($N \leq 8$, N: N-value of Standard Penetration Test) in Yangon Area are to be identified and clarified based on actual soil investigation data using the methods and

equipment prescribed in JGS (Japanese Geotechnical Society, hereinafter called as JGS) Standards [5].

2) Yangon Area is located at the border of different river basins between Lower Irrawaddy Delta Sub-Basin and Pegu-Yoma Sittaung Basin [6]. The differences of soil properties between two basins are identified and clarified by separating Yangon Area into 7 Sub-areas.
3) In Thilawa Area, SEZ (Special Economic Zone) construction projects and International Port Construction Projects are on-going along the Yangon River where soft clay ($N \leq 4$, N: N-value of Standard Penetration Test) is accumulated more than 20m thick. Accordingly, among the Yangon Sub-areas, Thilawa Sub-area is focused in detail to identify and clarify the soil properties of soft clays in Thilawa Sub-area along the Yangon River.
4) In Myanmar, mainly two different type of samplers for taking undisturbed samples are using at the moment, one is Shelby Tube sampler prescribed in ASTM Standards [7] and another is fixed piston sampler prescribed in JGS Standards [8]. Several past researches [9][10][11][12] indicate that apparent influences onto sample quality is due to different types of samplers. Therefore, to confirm the influence on to quality of undisturbed sample of soft clay in Yangon and its suburbs, some soil properties are compared by using undisturbed soft clay samples taken with Shelby tube sampler and fixed piston sampler.
5) Finally, this study is carried out in order to share the information on soft to firm clays in Yangon Area including Thilawa Area. In Chapter 4, the study results of clays in Yangon including Thilawa (Target Area 1) were described and, in Chapter 5, soft clay along the Yangon River in Thilawa (Target Area 2) was studied as shown in Figure 1-6. The results of this study are planned to utilize for carrying out smooth project planning, design and construction works for all engineers who work for the projects in Yangon.

From the described above, we focused on the clays distributed in whole Yangon, which often causes problems in construction projects, and the distribution of the clays in Yangon and characteristics of clays in whole Yangon area were identified due to the following procedures as shown in Figure 1-8.



Source: Retouched to JICA Final Report of The Project for the Strategic Urban Development Plan of the Greater Yangon, 2013

Figure 1-6 Target areas of this Study

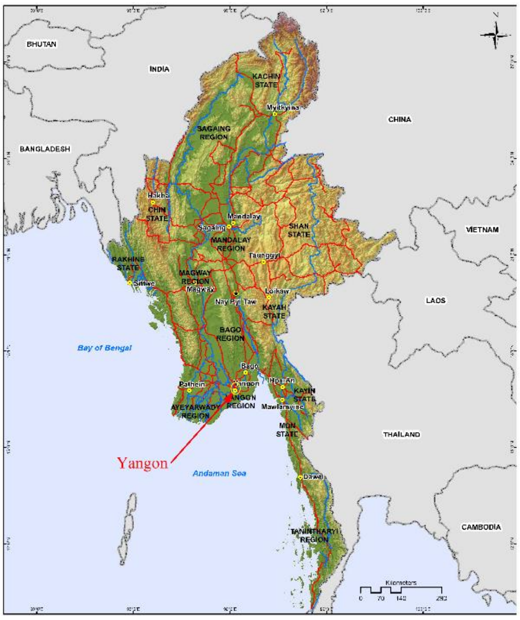


Figure 1-7 Topographic Map of Myanmar
Source: Country Map MIMU982v01_01Mar13_Topo_A4.pdf by MIMU 2013

Figure 1-8 Procedure of this Study

1.3 Configuration of Papers

This Thesis is written based on the above research purposes as a core. The main contents are described as below.

Chapter 1 Introduction

Clarification of the background and purposes of this study is done.

Chapter 2 Past researches on geotechnical engineering characteristics of cohesive soils

Past studies on physical and mechanical properties of cohesive soils are reviewed and confirm the position of this study.

Chapter 3 Effects of sample disturbance due to different sampling methods with regard to the quality of undisturbed samples of cohesive soils in Yangon

Past studies mentioned on influence onto sample quality due to the different sampling methods, the comparison of qualities of cohesive soil samples taken by different type of samplers are mentioned here.

In this Chapter, comparison of quality of samples especially taken with the fixed piston sampler and the Shelby tube sampler is carried out for samples taken in Yangon and its suburbs.

Chapter 4 Comparison and study of geotechnical engineering characteristics of cohesive soils in Yangon

Data on soil properties such as physical and mechanical properties in Yangon are collected, compiled and analyzed based on the actual soil investigation results carried out in Yangon. Then characteristics of soil properties of soft to firm clays in Yangon are identified and clarified.

Chapter 5 Geotechnical engineering characteristics of soft clays in Thilawa area along the Yangon River

Data on soil properties such as physical and mechanical properties in Thilawa area are collected, compiled and analyzed based on the actual soil investigation results carried out in Thilawa area. Then characteristics of soil properties of soft clays in Thilawa area are identified and clarified.

Chapter 6 Conclusion

After summarizing this study, the results obtained and future issues are mentioned.

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2. Past researches on geotechnical engineering properties of cohesive soils

2.1 Outline of Past Researches

As the past researches related to the field of this thesis, the following research papers were studied and those were classified into 5 categories as shown in Table 2-1 below.

Table 2-1 Classification of past researches on related field

Category	Group	Related Matters	Key Words	Past studies
1	Correlations among Physical and Mechanical Properties	Marine clay	Correlations among physical properties, Correlations between mechanical properties and physical properties	Ogawa, Matumoto [1] Tanaka, Sakakibara [2] Nacci, Wang, Demars [3] Murakami. Tsuchida et al. [4] Balasubramaniam, Cai et al. [5]
		All kinds of clays	Correlations between compression index C_c and physical properties	Yoon, Kim, Jeon [6] Abdrabbo, Mahmoud [7] Azzouz, Krizek, Corotis [8] Cozzolino [9] Saxena, Ramaswamy et al. [10] Al-Khafaji, Andersland [11] Yamauchi, Miyoshi [12] Yamada, Tsuchida et al. [13]
2	Shear Strength and Shear Strength Ratio of Cohesive Soils	Shear strength ratio c_u/p'	Vane shear test	Shibata, Tagawa[14] Fujikawa, Takayama[15]
		Strength ratio between different test types	Unconfined compression test Tri-axial compression test	Tsubota [16]
		Evaluation/ Determination of shear strength	Unconfined compression test Vane shear test	Tsuchida, Tanaka [17] Tanaka et al. [18]
3	Coefficient of Secondary Consolidation	Laboratory Test	Isotache Model	Watabe, Tanaka et al. [19]
			Correlation between $C_{\alpha\varepsilon}$ and C_c	Kanayama, Hiyama et al. [20]
		Field monitoring	Long-term observation records	Kumamoto, Tsuchida et al. [21] H. Aboshi [22] Shirako, A. Tonosaki et al. [23]
			Preconsolidation, Reduction of $C_{\alpha\varepsilon}$	Oohara, Matsuda, Aboshi [24]
4	Relations between Clay Minerals and Physical Properties	Changing of minerals	Sequential Action Smectite	Yoshimura[25]
		Correlation with soil properties	Specific surface area Liquid limit, Compression index Coefficient of Consolidation	Maeda & Souma[26]) Kitagawa[27] Kamon & Kawamae[28]
5	Effect of Sample Disturbance due to Different Sampling Methods	Shear Strength	Effect of different sampler types Unconfined compressive strength	Tanaka, Sharma, Tsuchida et al [29] Tsuchida, Kobayashi et al. [30]
		Shear Strength & Consolidation properties	Effect of sample disturbance	Shogaki, Kaneko [31]
		Evaluation of Undrained shear strength	SHANSEP method Recompression method by triaxial compression test	Tsuchida [32] Tsuchida, Noguchi et al [33]
		Evaluation of material properties	Effect of investigation method	Tanaka [34] Ladd & DeGroot [35]

* C_c: Compression index, $C_{\alpha\varepsilon}$: Coefficient of secondary consolidation

2.2 Correlations among Physical and Mechanical Properties

There are many studies showing statistical correlation between physical properties and mechanical properties, classification properties such as natural moisture content, Atterberg limits, grading etc. based on a lot of data. But Ogawa and Matsumoto's research, 1978) [1] showed correlation between the soil properties on the basis of the data of alluvial clays from the port and harbor areas of all over Japan except Hokkaido and Okinawa regions.

In this research, among then, the correlation of the data across the country is discussed, and for the data of 6 specific regions, the correlations of soil properties were discussed region by region. Source locations of samples are shown in Figure 2-1 and the representative correlation diagrams were shown in Figure 2-2 and Figure 2-3. According to Figure 2-2, distributions of soil properties at port and harbor areas in Japan can be understood easily.

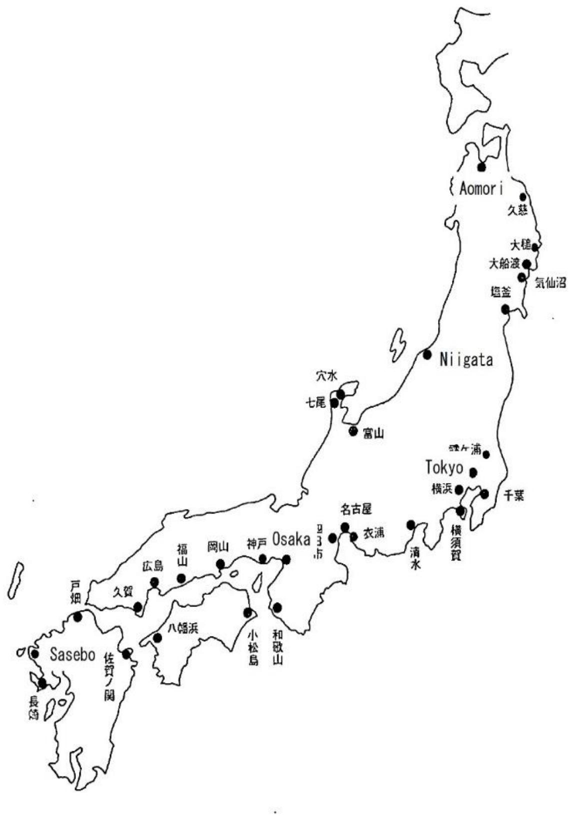


Figure 2-1 Source locations of Samples

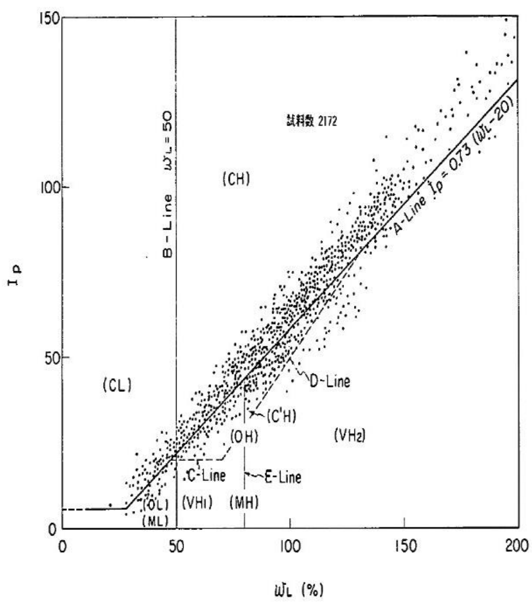


Figure 2-2 $I_p \sim w_L$ relation (Plasticity Chart) for Whole Samples

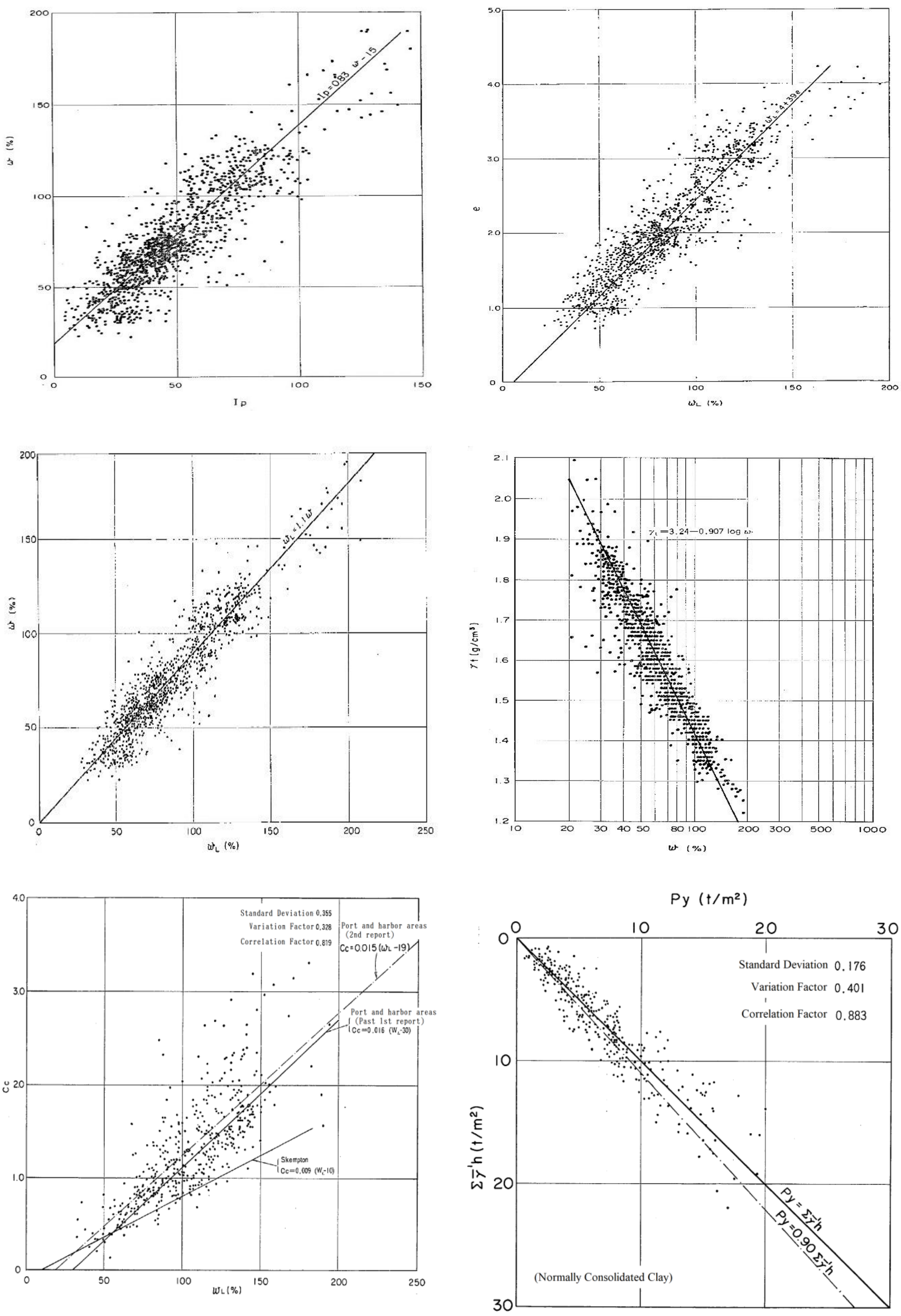


Figure 2-3 Correlation Graphs between Physical properties/Mechanical properties

From these results, it was concluded as follows.

i) Relatively good correlation is recognized from correlation between classification properties, and approximate characteristics of soil properties for each region can be seen from classification Properties for each region.
ii) From the relationships between consolidation properties and classification properties, it is possible to obtain approximate values of consolidation parameters in port and harbor areas by knowing classification properties.
iii) Although no clear correlation was found with regard to the strength properties, a powerful clue is found to know the soil properties of the port and harbor areas. The effective utilization of these results is expected as preliminary soil data, and to know approximate design parameters for evaluation of project costs.

Likewise, there is a study by Tanaka and Sakakibara, 1991 [2)] focusing on the primary properties of soils, in the port and harbor areas in Japan, such as a density of soil particle, Atterberg limits, grain size distribution which are not affected by the stress history of soil and compression index C_c, coefficient of consolidation c_v of normally consolidated clay were also considered as the primary properties for consolidation of clay. This research showed a statistical analysis focusing on the physical properties and consolidation properties of the soil in the port and harbor areas throughout Japan except for Hokkaido region, and divided the Japanese archipelago into 11 regions from geographical and geological points of views as shown in Figure 2-4. Correlation of various soil properties were studied. Obtained primary properties of 11 regions of Japan by this research are shown in Figure 2-5, Figure 2-6 and Figure 2-7 .

According to these figures and tables, differences of primary properties at the port and harbor areas can be understood well.

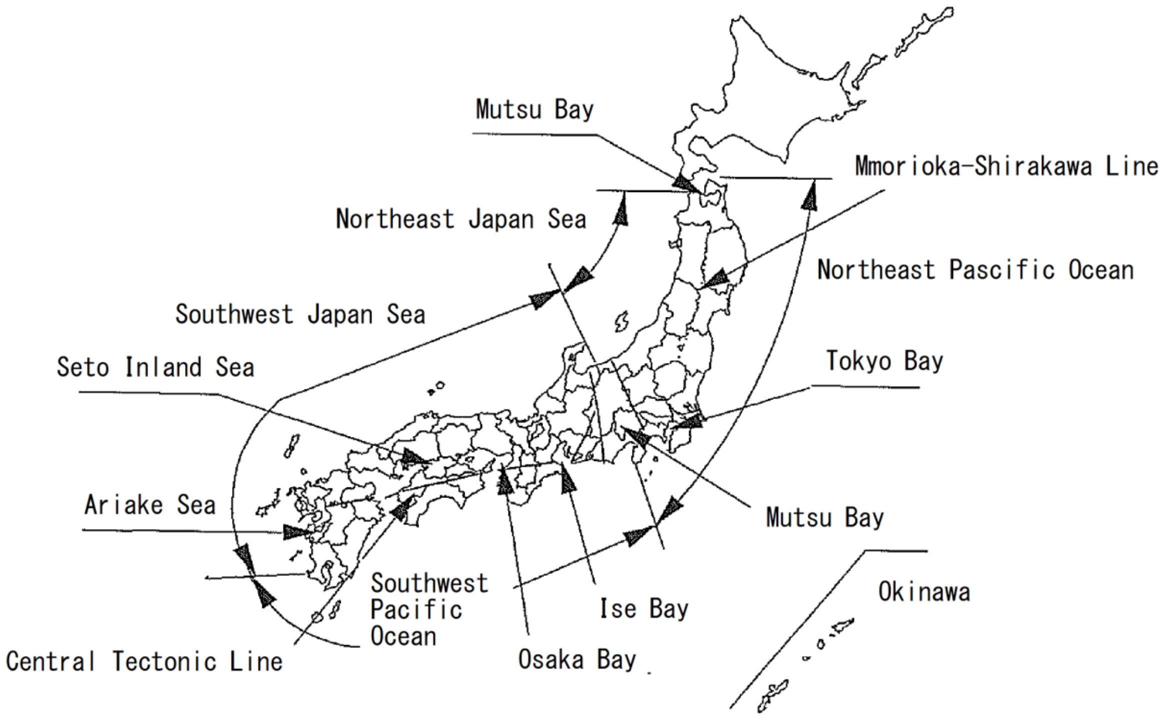


Figure 2-4 Divisions into 11 Regions

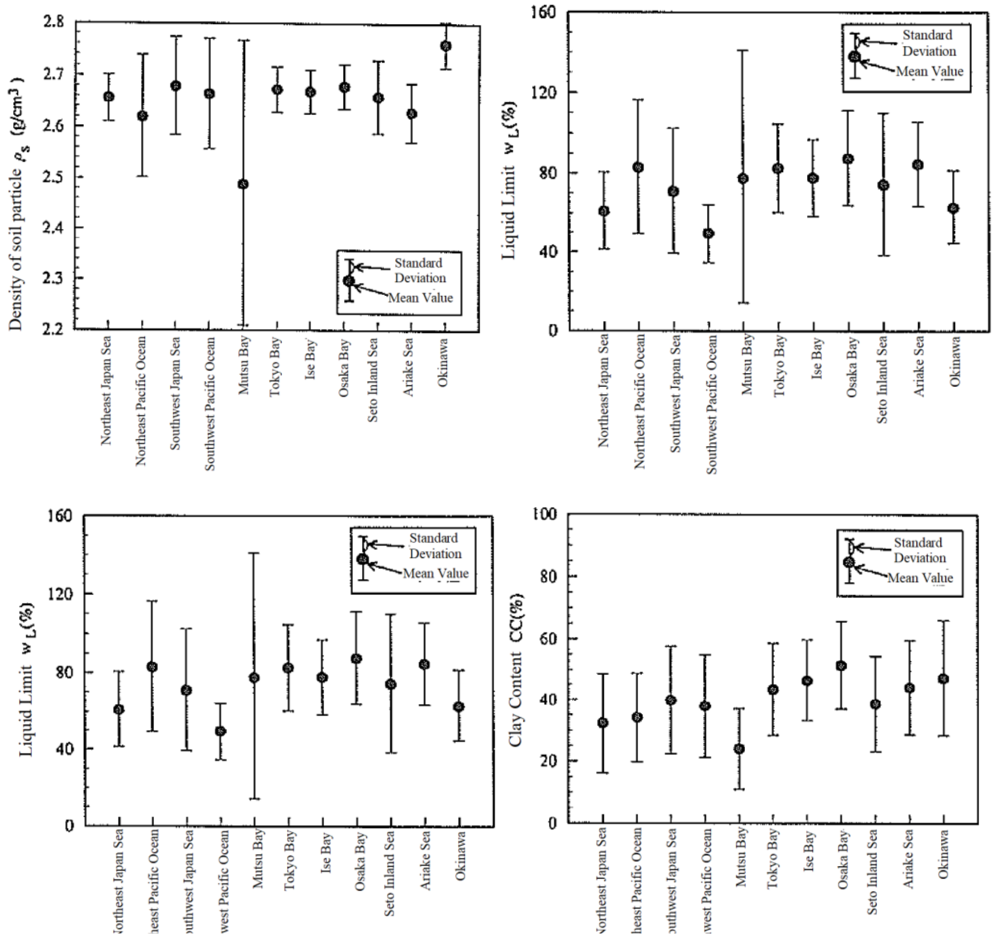


Figure 2-5 Primary Properties of Soils at Japanese Coastal Region

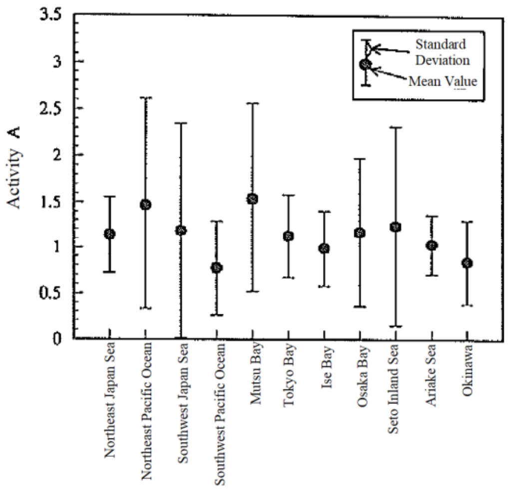


Figure 2-6 Activities of each Region

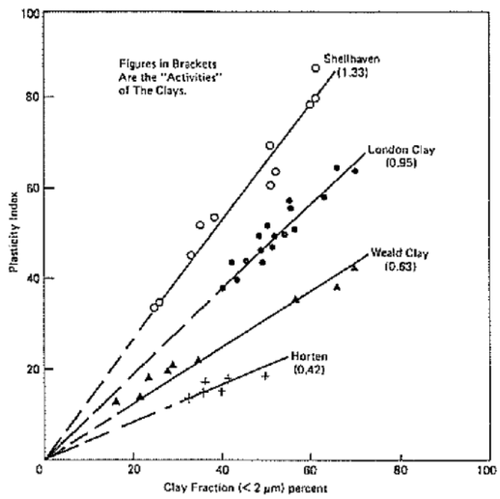


Figure 2-7 Relation between Clay Content and Activity

It has been made by many researchers all over the world to empirically connect the consistency limits expressing the fundamental nature of the soil with the parameters of the mechanical properties and consolidation properties, and so many empirical formulas have been proposed up to now. Among them, there are well known studies by Skempton (Skempton, 1957) [36)] relating to the relationship between compression index C_c and liquid limit w_L and Bjerrum (Bjerrum et al., 1960) [37)] relating to the relationship between the shear strength ratio and plasticity index I_P. Those relational equations are as follows.

$$C_c = 0.009\,(w_L - 10) \qquad \text{(Skempton, 1957)}^{36)} \qquad (1)$$

$$c_u/p' = 0.045\, I_P^{\,0.5} \qquad \text{(Bjerrum et al., 1960)}^{37)} \qquad (2)$$

These equations are quite well known and often quoted in soil survey reports or research reports, and extent of agreement with these equations are discussed. However, Tanaka and Sakakibara, 1991 [2)] pointed out that the soils they targeted were strongly affected by the ice ages in northern Europe and North America, and the properties are quite different from those of Japanese clay. For example, applying the relational equation (1) between C_c and I_P to clays in Japan is known to underestimate C_c, and Hanzawa and Tanaka, 1992 [38)] pointed out that the relationship between Bjerrum's shear strength ratio and I_P cannot be applied to clays in Japan.

From the above study results, the following conclusions on correlation of primary properties of clays in the port and harbor areas of Japan are obtained.

i) The density of soil particles ρ_s was distributed between 2.6 and 2.7, and the average value was 2.67. However, ρ_s in Mutsu Bay is smaller than in other areas, and ρ_s in Okinawa is larger than others as shown in Figure 2-5.

ii) The plasticity limit w_P is distributed between 20% and 50%. Liquid limit w_L and plasticity index I_P show large variations, but seeing maximum values, w_L is 110% and I_P is about 75. Regarding I_P regionally, the clays in Osaka bay is highly plastic and the Northeast Japan Sea and the Southwest Pacific tend to be low plastic as shown in Figure 2-5.

iii) The relationship figures between I_P and w_L of clays in Japan is distributed along A-line. However, looking at this relationship in details, the samples in the Southwest Sea of Japan and Osaka Bay are above A-line, and the samples in Ariake Sea and Mutsu Bay are distributed under A-line.

iv) The value obtained by dividing I_P with the clay content (particle size is 5 μm or less), that is, the activity of clays is distributed in the range between 0.8 and 1.5. Regionally, the activities of clays in Northeast Pacific, Southwest Japan Sea, Mutsu Bay, Osaka Bay and Seto Inland Sea are high as shown in Figure 2-6.

v) The relationship between compression index C_c and liquid limit w_L varies considerably from region to region. In the relationship between them, Skempton's equations, 1957 [36] give the lower limit. From correlation line of $C_c = 0.015\ (w_L - 19)$ proposed by Ogawa and Matsumoto, 1978 [1] plotted data also deviates largely depending on the regions.

vi) The relationship between the coefficient of consolidation c_v and w_L varies widely, but it lies between the lines of "$\log c_v = 5 - 0.05\ w_L$" and "$\log c_v = 4.6 - 0.02\ w_L$".

On the other hand, many studies have proposed the correlations between compression index C_c and physical properties. K. R. Sexena et al., 1995 [10] classified correlation equations that estimated C_c proposed by past studies so far as shown in Table 2-2 . These correlation equations are classified into the following four categories.

Category A $\quad C_c = f$ (Liquid Limit; w_L)

Category B $\quad C_c = f$ (Natural Moisture Content; w_n)

Category C $\quad C_c = f$ (Initial Void Ratio; e_0)

Category D $\quad C_c = f\ (e_0, w_L, w_n)$

As a result of examination by their studies, the following conclusions are obtained.

i) The equation for compression index C_c based on liquid limit w_L and natural moisture content w_n cannot be explained fundamentally and therefore possess no physical significance.

ii) The equation for compression index C_c based on initial void ratio e_0 can be explained fundamentally and possesses physical significance but suffer due to the errors in the estimation of initial void ratio.

iii) The equation for compression index C_c based on initial dry density ρ_d can be explained fundamentally and possess physical significance and relatively superior over the equations based on void ratio.

iv) Relational equation $C_c = 0.5\ (\gamma_w/\gamma_d)^{2.5}$ has proved to possess a universal applicability and is therefore recommended for adoption for all ordinary soils.

v) Relational equation $C_c = 0.5\ (\gamma_w/\gamma_d)^{2.5}$ is found to bear a strong testimony for the effective stress concept and for the applicability of elastic theory for constrained compression.

Table 2-2 Equations for Prediction of compression index C_c

	Equation	Applicability	Reference
	(a) C_c = f(LL)		
A1	C_c=0.007(LL-7)	Remolded clays	Skempton(1944)
A2	C_c=0.0046(LL-9)	Brazilian clays	Cozzolino(1961)
A3	C_c=0.009(LL-10)	Normally consolidated clays	Terzaghi and Peck (1967)
A4	C_c=0.006(LL-9)	All clays with liquid limits less than 100%	Azzouz et al. (1976)
A5	C_c=(LL-13)/109	All clays	Mayne (1980)
	(b) C_c = f(W_n)		
B1	$C_c=0.85(W_n/100)^{3/2}$	Finninish muds and clays	Helenelund (1951)
B2	C_c=0.0115 W_n	Organic Soils, peat, organic silt, and clays	Moran et al. (1958)
B3	C_c=0.01(W_n-5)	All clays	Azzouz et al. (1976)
B4	C_c=0.01 W_n	All clays	Koppula (1981)
B5	C_c=0.01(W_n-7.549)	All clays	Herrero (1983)
	(c) C_c = f(e_o)		
C1	C_c=0.54(e_o-0.35)	All clays	Nishida (1956)
C2	C_c=0.29(e_o-0.27)	Inorganic, cohesive soil, silt some clay; silty clay; clay	Hough (1957)
C3	C_c=0.35(e_o-0.50)	Organic, fine grained soil, organic silt, little clay	Hough (1957)
C4	C_c=0.43(e_o-0.25)	Brazilian clay	Cozzolino (1961)
C5	C_c=0.75(e_o-0.50)	Soils with low plasticity	Sowers (1970)
	(d) C_c = f(e_o, LL, W_n)		
D1	C_c=0.37(e_o+0.003 LL-0.34)	All clays	Azzouz (1976)
D2	C_c= -0.156+0.411 e_o+0.00058 LL	All clays	Al-Khafafi (1962)
D3	C_c=0.009 W_n + 0.002LL-0.10	All clays	Azzouz (1976)

A.S. Balasubramaniam et al., 2010 [5] evaluated the consolidation parameters from many empirical correlations between mechanical properties and physical properties, then calculated consolidation settlement of highway embankment using these parameters obtained from empirical correlations as shown in Table 2-3, Table 2-4, Table 2-5 and Table 2-6.

Table 2-3 Compression Ratio C_R from moisture content

Compression Ratio from moisture content		
Authors	**Formula**	**Range of w_n**
Simons and Menzies (1975)	$CR = 0.006w_n - 0.03$	$20 \leq w_n \leq 140$
Simons (1957)	$CR = 0.006w_n^{1.68}$	$28 \leq w_n \leq 57$
Wilkes (1974)	$CR = 0.26\ln(w_n) - 0.83$	$30 \leq w_n \leq 90$
Lamb and Whitman (1969)	$CR = 0.12\ln(w_n) - 0.28$	$10 \leq w_n \leq 100$

Table 2-4 OCR from Plasticity index

OCR from Plasticity index	
Authors	**Formulae**
Skempton and Henkel (1953)	$OCR = 0.0017I_P + 0.5$
Osterman (1959)	$OCR = 2\times10^{-6}I_P^3 - 3\times10^{-4}I_P^2 + 3.1\times10^{-2}I_P + 0.41$
Bjerrum and Simons (1960)	$OCR = 2\times10^{-6}I_P^3 - 4\times10^{-4}I_P^2 + 3.35\times10^{-2}I_P + 0.28$

Table 2-5 Undrained Shear Strength from Plasticity Index

Undrained shear strength from plasticity index	
Authors	**Formulae**
Skempton and Henkel (1953)	$s_u/\sigma'_{vo} = 0.004I_P + 0.1$
Osterman (1959)	$s_u/\sigma'_{vo} = 5\times10^{-7}I_P^3 - 8\times10^{-5}I_P^2 + 6.8\times10^{-3}I_P + 0.08$
Bjerrum and Simons (1960)	$s_u/\sigma'_{vo} = 5\times10^{-7}I_P^3 - 8\times10^{-5}I_P^2 + 7.4\times10^{-3}I_P + 0.06$

Table 2-6 Values of $C_{\alpha\varepsilon}/C_{c\varepsilon}$ for Geotechnical Material (Mesri et al., 1994)

Material	$C_{\alpha\varepsilon}/C_{c\varepsilon}$
Granular soils including rockfill	0.02 ± 0.01
Shale and mudstone	0.03 ± 0.01
Inorganic clays and silts	0.04 ± 0.01
Organic clays and silts	0.05 ± 0.01
Peat and muskeg	0.06 ± 0.01

As for the Myanmar clay, there is a paper by Murakami and Tsuchida et al., 2015 [4] which discussed the comparison of the soil properties of Myanmar clay with the properties of clays in the port and harbor areas of Japan. In this paper, they showed the comparison results between soil properties of clays in Myanmar and Japan (Port and harbor areas) according to the testing methods based on Standards of Japanese Geotechnical Society (JGS), using samples collected by the fixed piston sampler same as those used in Japan. Based on these test results carried out in this study, the following conclusions are obtained.

i) In the comparison of the basic properties of clays, the density of soil particle and the activity showed almost same values in both clays of Myanmar and Japan as shown in

Figure 2-8 and Figure 2-9. However, the liquid limits of Myanmar were distributed in the narrow range of 20 to 100%, while the clay of Japan (Port and harbor areas) was distributed widely as shown in Figure 2-10. In addition, the natural moisture contents of clays in Myanmar are as small as 62% of the average of liquid limit, the wet density is ranging between 1.6 and 2.1 g/cm^3, which is larger than that of clays in Japan by about 0.3 g/cm^3 as shown in Figure 2-11.

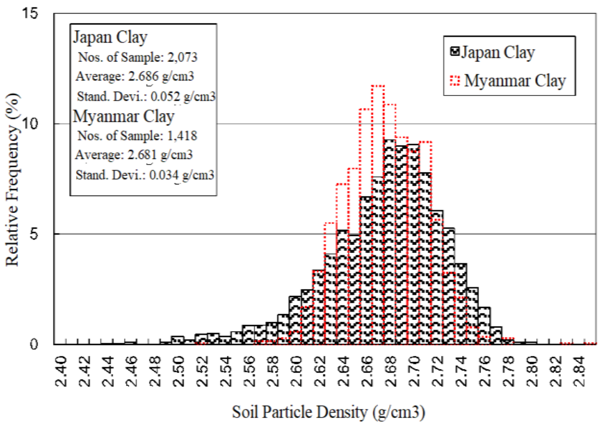


Figure 2-8 Histogram for Particle Density ρ_s

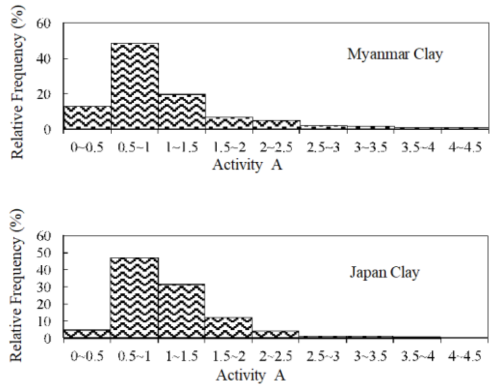


Figure 2-9 Histogram for Activity of Soil

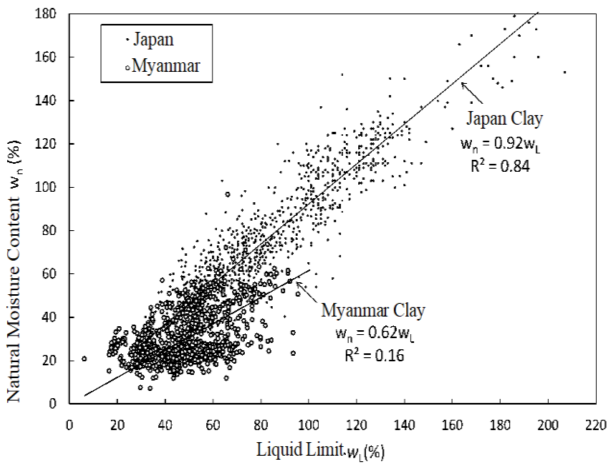


Figure 2-10 $w_n \sim w_L$ Relation

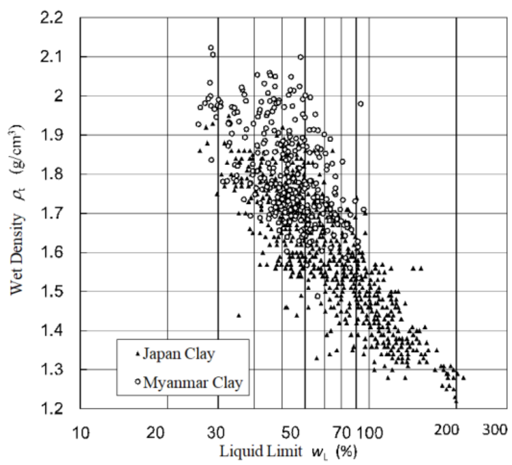


Figure 2-11 $\rho_t \sim w_L$ Relation

ii) The relationship between the compression index C_c of clays in Myanmar and the liquid limit w_L was obtained as $C_c = 0.007(w_L+8.7)$. Compared to clay of the same liquid limit, the C_c of clays in Myanmar clay was about half of the C_c of clays in Japan (port and harbor areas) as shown in Figure 2-12. Meanwhile, when the coefficient of consolidation c_v with the liquid limit w_L. c_v of clays in Yangon has about 0.44 times of the average of clays in Japan (port and harbor areas) when the liquid limit is 30%, and 0.31 times and 0.22 times when the liquid limit is 50% and 70% as shown in Figure 2-13. Accordingly, when estimating the coefficient of consolidation c_v from the liquid limit w_L of clays, overestimating the coefficient of consolidation might occur due to experience on clays in Japan.

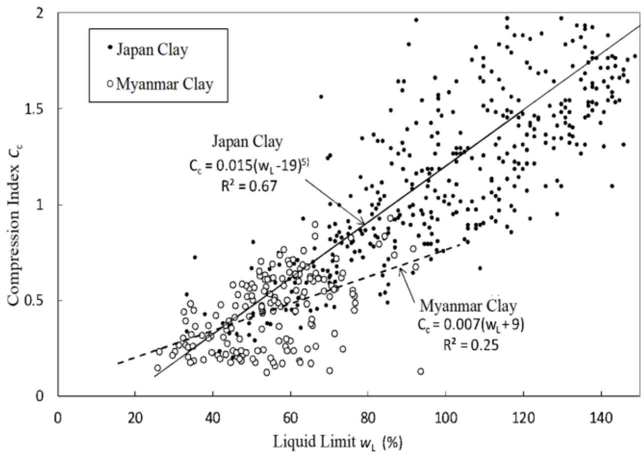


Figure 2-12 $C_c \sim w_L$ Relation

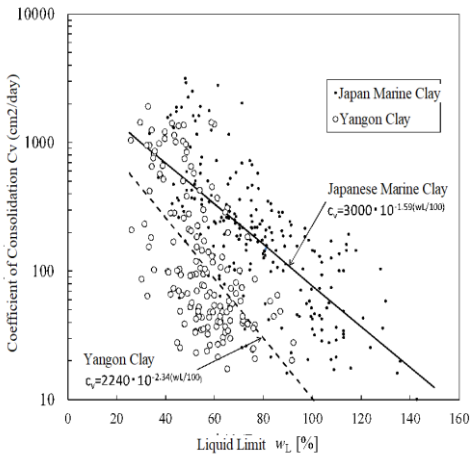


Figure 2-13 $c_v \sim w_L$ Relation

iii) Then unconfined compressive strength of clays in Myanmar was found to be 30% smaller than that of clays in Japan (port and harbor areas) compared with the case of the same consolidation yield pressure p_c as shown in Figure 2-14. Estimation of the shear strength ratio from the relationship between q_u and the effective overburden pressure of normally consolidated clay showed that the average shear strength ratio of clays in Myanmar clay was 0.225 which is about 2/3 of that of clays in Japan (port and harbor areas). As for the failure strain of unconfined compressive strength, there was no significant differences between clays in Myanmar clay and Japan as shown in Figure 2-15.

iv) Regarding the compression index C_c of clays in Myanmar clay, multiple regression analysis is performed with two combinations, the initial void ratio e_0, the natural moisture content w_n, the liquid limit w_L, and the initial void ratio e_0, the liquid limit w_L, and the plasticity index I_P. Two correlation equations have a good determination factor with coefficient of determination factor with $R^2 = 0.86$ and showed that the compression index C_c can be predicted with useful accuracy form engineering points of views considering variation of the design parameters of soil as shown in Figure 2-16 and Figure 2-17.

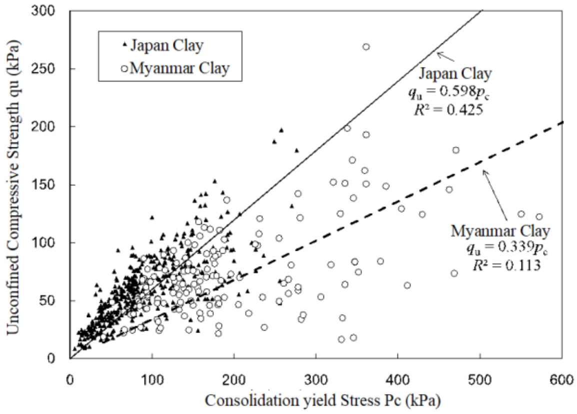


Figure 2-14 q_u and p_c Relation

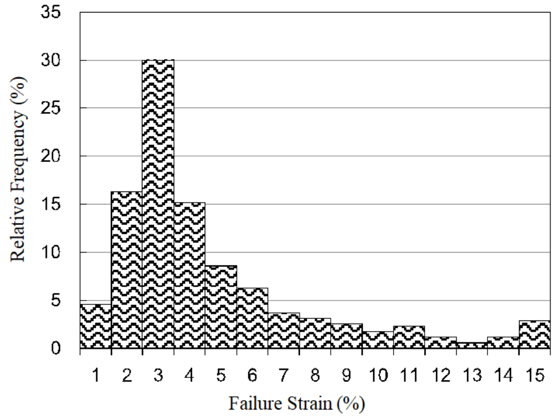


Figure 2-15 Failure Strains of Myanmar clay

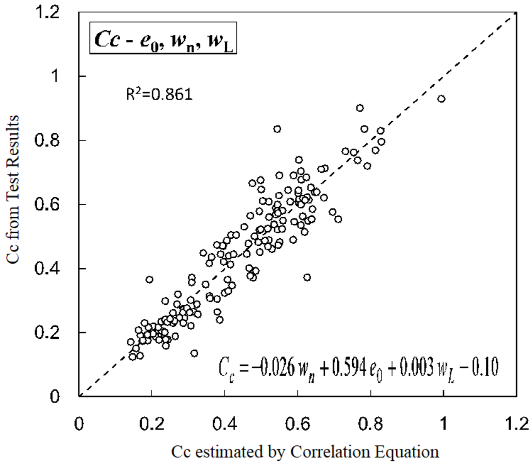


Figure 2-16 Determination Factor of Correlation Equation

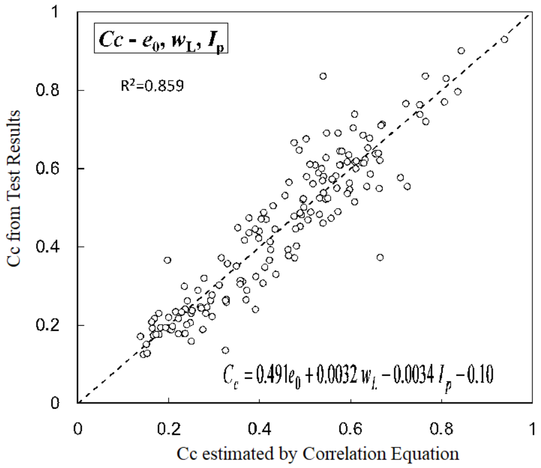


Figure 2-17 Determination Factor of Correlation Equation

In addition to the above past study, Yoon, Kim and Jeon, 2004 [6)] who studied the correlation equations about the marine clays in the coastal areas of Korea, which are statistically obtained from correlation between compression index C_c and liquid limit w_L, natural moisture content w_n, initial void ratio e_0, plasticity index I_P, etc. obtained from a total of about 1,200 consolidation test results in West Coast, East Coast and South Coast of Korea as shown at the locations in Figure 2-18. And compared them with correlation equations prepared by past researches such as shown in Table 2-7. Soil properties of marine clays in Korea are shown in Table 2-8.

As a result, the following conclusions were drawn.

Empirical equations proposed in past studies associated with most soil parameters are not applicable to predicting the Compression Index C_c in marine clays in Korea. The empirical equations associated with natural water content w_n are applicable to marine clays, however, and those suggested by Koppula, 1981 [39] and Herrerro, 1983 [40] are reasonable for use in the south and east coast in Korea respectively as shown in Table 2-9.

Relationship between the compression index and (a) natural moisture content, (b) initial void ratio, (c) liquid limit, (d) dry unit weight and (e) plasticity index are shown in Figure 2-19.

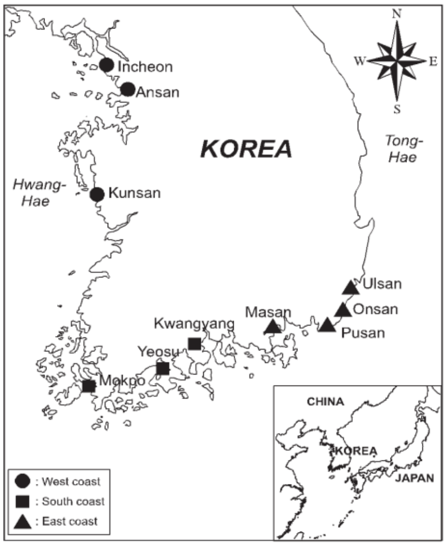


Figure 2-18 Location of Borings in three coastal area in Korea

Table 2-7 Empirical equations for Compression Index C_c

Equation	Reference	Applicability
For $C_c = f(\omega_l)$		
$C_c = 0.013(\omega_l - 13.5)$	Yamagutshi 1959	All clays
$C_c = 0.017(\omega_l - 20)$	Shouka 1964	All clays
$C_c = (\omega_l - 13)/109$	Mayne 1980	All clays
For $C_c = f(\omega_n)$		
$C_c = 0.01(\omega_n - 5)$	Azzouz et al. 1976	All clays
$C_c = 0.01\omega_n$	Koppula 1981	All clays
For $C_c = f(e_0)$		
$C_c = 0.54(e_0 - 0.35)$	Nishida 1956	All clays
$C_c = 0.43(e_0 - 0.25)$	Cozzolino 1961	Brazilian clays
$C_c = 0.75(e_0 - 0.50)$	Sowers 1970	Clays with low plasticity
For $C_c = f(\mathrm{PI})$		
$C_c = 0.02 + 0.014\mathrm{PI}$	Nacci et al. 1975	North Atlantic clay
$C_c = 0.046 + 0.0104\mathrm{PI}$	Nakase et al. 1988	Best for PI < 50%

Note: PI, ω_l, and ω_n are in percentages.

Table 2-8 Soil Properties of Marine Clay in Korea

Statistical parameter	ω_n (%)	e_o	G_s	γ_d (t/m^3)	ω_l (%)	PI (%)	C_c
West coast (356 samples)							
Range	22.40–75.40	0.636–2.022	2.600–2.750	0.903–1.644	24.500–77.900	1.000–52.100	0.050–0.840
Average, μ	39.07	1.079	2.694	1.312	41.150	17.940	0.292
SD, σ	9.24	0.257	0.020	0.147	9.173	8.612	0.139
East coast (603 samples)							
Range	23.50–112.90	1.008–3.048	2.370–2.760	0.667–1.350	23.000–107.000	2.600–64.600	0.130–1.560
Average, μ	61.59	1.860	2.707	0.954	63.130	35.760	0.649
SD, σ	11.32	0.317	0.026	0.107	11.760	9.410	0.213
South coast (278 samples)							
Range	30.70–134.40	0.750–3.347	2.420–2.750	0.567–1.510	28.400–120.200	6.300–74.200	0.170–1.910
Average, μ	80.77	2.190	2.682	0.875	67.800	38.120	0.996
SD, σ	21.92	0.556	0.064	0.182	20.530	15.250	0.389

Note: C_c, compression index; e_o, initial void ratio; G_s, specific gravity; PI, plasticity index; SD, standard deviation; γ_d, dry density; ω_l, liquid limit; ω_n, natural water content.

Table 2-9 Reliability between the predicted and measured compression indecies using a single parameter

		$C_{c(\text{predicted})}/C_{c(\text{measured})}$		
Soil parameter	Reference	West coast	South coast	East coast
ω_l	Yamagutshi 1959	1.299 (0.558)	0.762 (0.291)	1.040 (0.265)
	Shouka 1964	1.261 (0.606)	0.857 (0.337)	1.171 (0.311)
	Mayne 1980	0.935 (0.400)	0.543 (0.207)	0.742 (0.189)
	Terzaghi and Peck 1967	1.024 (0.428)	0.567 (0.217)	0.774 (0.196)
ω_n	Azzouz et al. 1976	1.344 (0.644)	0.844 (0.275)	0.934 (0.259)
	Koppula 1981	1.559 (0.754)	0.908 (0.310)	1.020 (0.286)
	Herrero 1983	1.235 (0.590)	0.811 (0.258)	0.890 (0.247)
e_o	Nishida 1956	1.518 (0.740)	1.084 (0.316)	1.303 (0.342)
	Cozzolino 1961	1.394 (0.676)	0.919 (0.277)	1.110 (0.294)
	Sowers 1970	1.624 (0.829)	1.362 (0.381)	1.620 (0.423)
PI	Nacci et al. 1975	—	—	0.838 (0.221)
	Nakase et al. 1988	—	—	0.676 (0.175)
Smallest value[a]	—	1.102 (0.463)[b]	1.070 (0.310)[c]	1.015 (0.257)[b]

Note: The values in the table are averages, with standard deviations in parentheses.
[a]Based on the compression index predicted by the regression equation.
[b]For liquid limit.
[c]For natural void ratio.

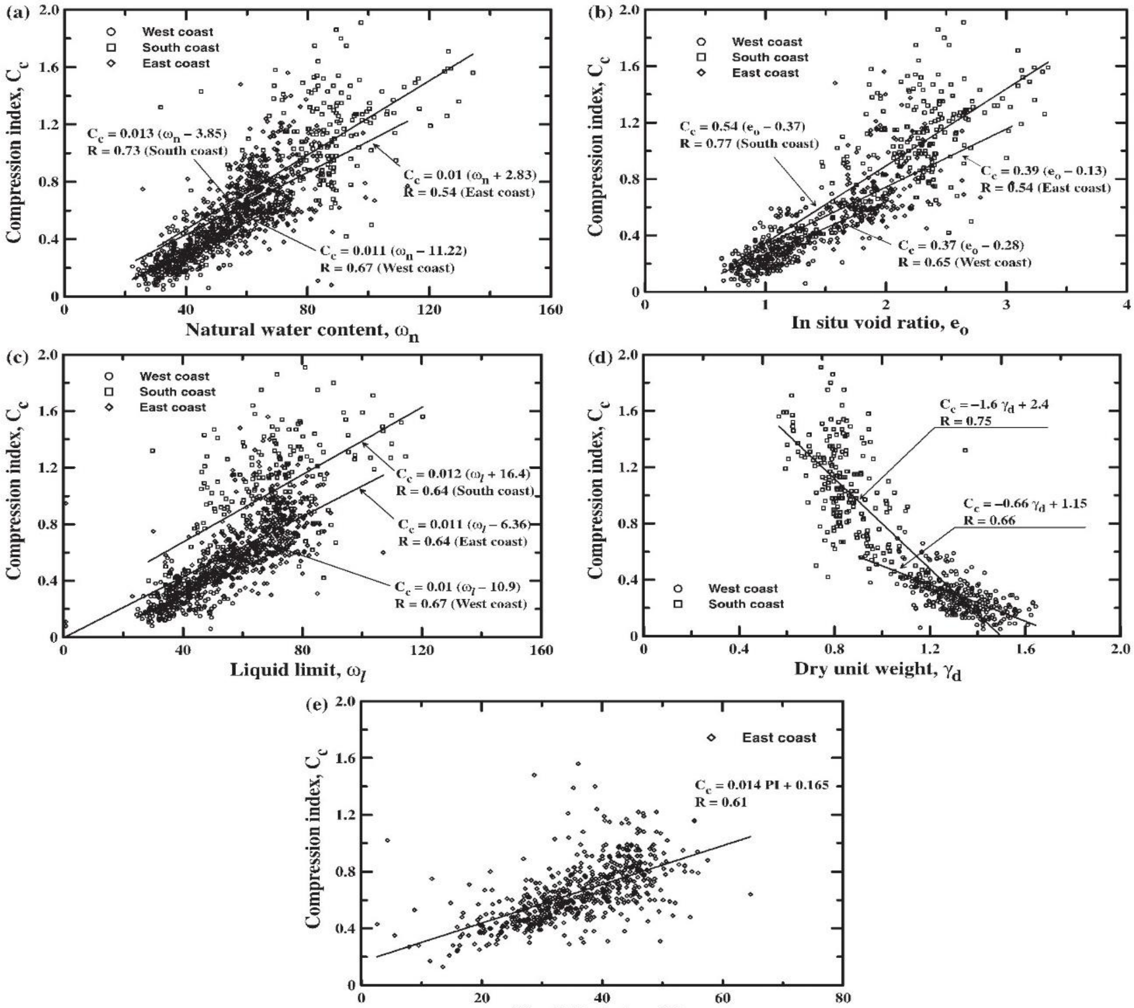


Figure 2-19 Relationship between the compression index and (a) natural moisture content, (b) initial void ratio, (c) liquid limit, (d) dry unit weight and (e) plasticity index

Abdrabbo and Mahmoud, 1990 [7] proposed equations to estimate compression index C_c from liquid limit w_L, natural moisture content w_n, void ratio e_0, etc. based on the test results of Egyptian clay around Alexandria and derived the following conclusions.

i) Terzaghi and Peck, 1948 [41] equation, $C_c = 0.009(w_L-10)$, for estimating compression index C_c of undisturbed clay overestimates the compression index of Egyptian clays by about 50% as shown in Figure 2-20.
ii) The value of $C_c/(1+e_0)$ of Egyptian clays is found to vary between 0.05 and 0.20, this value increased linearly as the natural moisture content of clays increased as shown in Figure 2-22.
iii) The compression index C_c of Egyptian clays increases as the in-situ void ratio e_0 increases, natural moisture content w_n increases, and liquid limit w_L increases as shown in Figure 2-20 to Figure 2-22.

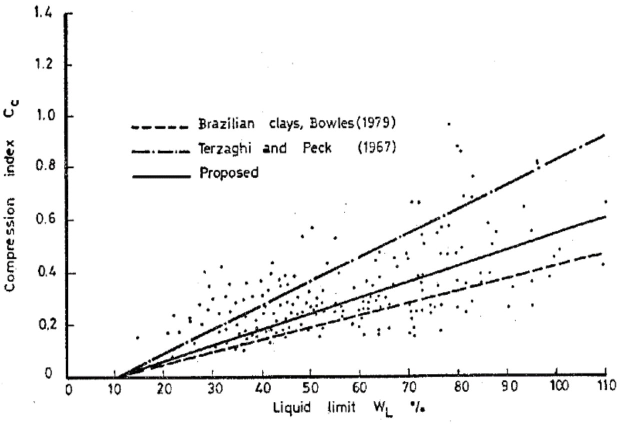


Figure 2-20 Compression index versus Liquid limit

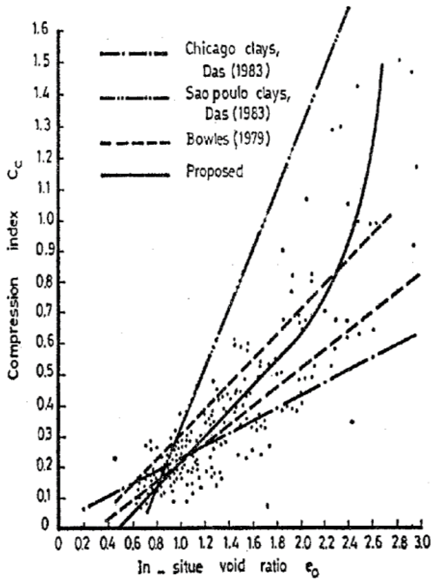


Figure 2-21 Compression index versus Initial void ratio

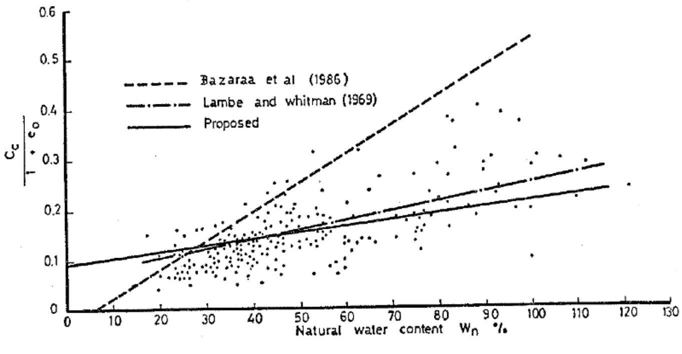


Figure 2-22 $C_c/(1+e_0)$ versus Natural moisture content

Azzouz, Krizek & Corotis et al., 1976 [8] performed a regression analysis using a consolidation test results of about 700 numbers (among those clays, 3/4 is Greek clay, 1/4 is clay in the USA), regression equations between the compression index C_c and liquid limit w_L, natural moisture content w_n, initial void ratio e_0 and other classification properties were obtained. Published regression equations for prediction of compression index C_c and compression ratio C_r were as shown in Table 2-10 and plasticity chart of those clay samples were as shown in Figure 2-23. Finally, they gave the following conclusions.

i) Both the compression index C_c and the compression ratio $C_c/(1+e_0)$ have best correlation with initial void ratio e_0 by means of simple linear regression models as shown in Figure 2-24.
ii) Introduction of other independent variables did not significantly improve the accuracy of the regression models.
iii) The correlations deduced are examined within the context of a large variety of similar, but more limited, relationships that have been reported by other researchers, and considerable differences were found as shown in Figure 2-25.

Table 2-10 Summary of published regression equations for prediction of compression index C_c and compression ratio C_r

Regression Equation	Regions of Applicability	Reference
$C_c=0.007(w_L-7)$	Remolded clays	Skempton, 1944
$C_r=0.208e_o+0.0083$	Chicago clays	Peck and Reed, 1954
$C_c=17.66\times10^{-5}w_n^2+5.93\times10^{-3}w_n-1.35\times10^{-1}$	Chicago clays	Peck and Reed, 1954
$C_c=1.15(e_o-0.35)$	All clays	Nishida, 1956
$C_c=0.30(e_o-0.27)$	Inorganic, cohesive soil; silt, some clay; silty clay; clay	Hough, 1957
$C_c=1.15\times10^{-2}w_n$	Organic soils-meadow mats, peats, and organic silt and clay	Moran, Proctor, Mueser and Rutledge, 1958
$C_c=0.256+0.43(e_o-0.84)$	Brazilian clays	Cozzolino, 1961
$C_c=0.0046(w_L-9)$	Brazilian clays	Cozzolino, 1961
$C_c=1.21+1.055(e_o-1.87)$	Motley clays from Sao Paulo city	Cozzolino, 1961
$C_c=0.0186(w_L-30)$	Motley clays from Sao Paulo city	Cozzolino, 1961
$C_c=0.009(w_L-10)$	Normally consolidated clays	Terzaghi and Peck, 1967
$C_c=0.75(e_o-0.50)$	Soils of very low plasticity	Sowers, 1970
$C_r=0.156e_o+0.0107$	All clays	Elnaggar and Krizek, 1971
$C_c=0.01w_n$	Chicago clays	Osterberg, 1972

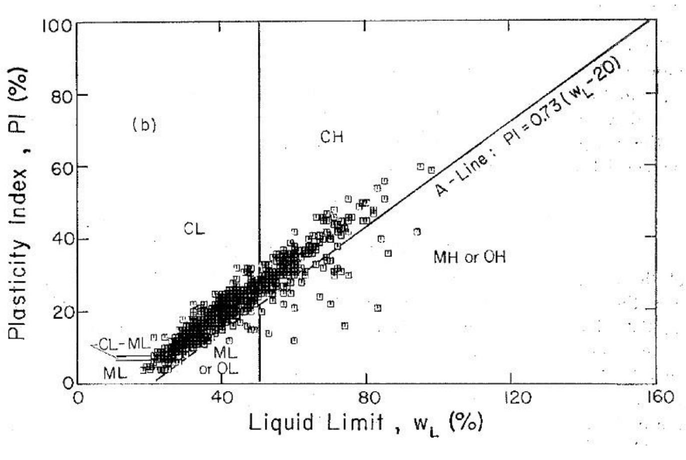


Figure 2-23 Classification of soil samples of Greece and U.S.A

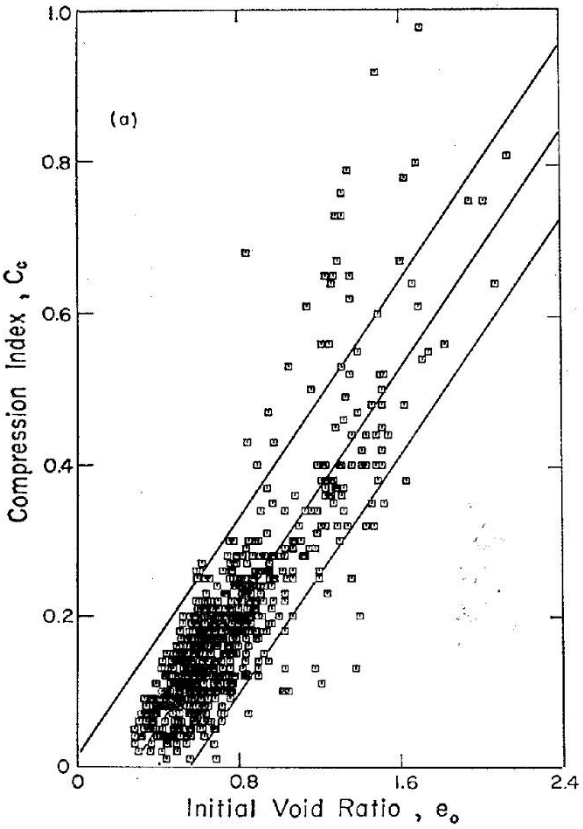


Figure 2-24 Relation between Compression Index C_c and Initial Void Ratio e_0

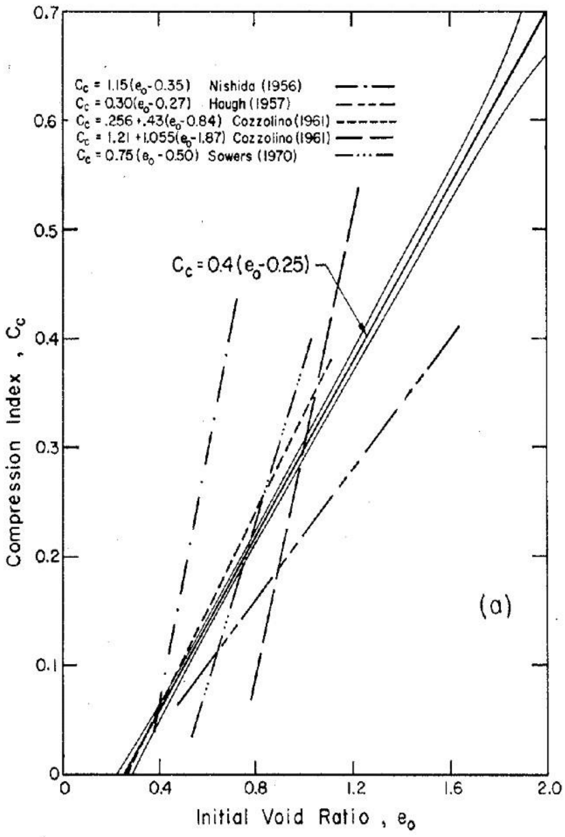


Figure 2-25 Empirical relationship between Compression Index C_c and Initial Void Ratio e_0

Table 2-11 Summary of regression equations used to predict compression index C_c and compression ratio C_r of clays in Greece and U.S.A

Dependent Variable	Independent Variable(s)	R	S	Regression Equation	Number of Samples
C_c	e_o	0.85	0.077	$C_c=0.40(e_o-0.25)$	717
	w_n	0.79	0.089	$C_c=0.01(w_n-5)$	717
	w_L	0.59	0.117	$C_c=0.006(w_L-9)$	678
	e_o and w_L	0.86	0.074	$C_c=0.37(e_o+0.003w_L-0.34)$	678
	e_o and w_n	0.85	0.007	$C_c=0.40(e_o+0.001w_n-0.25)$	717
	w_n and w_L	0.81	0.085	$C_c=0.009w_n+0.002w_L-0.10$	678
	e_o, w_n and w_L	0.86	0.074	$C_c=0.37(e_o+0.003w_L+0.0004w_n-0.34)$	678
C_r	e_o	0.74	0.038	$C_r=0.14(e_o+0.007)$	717
	w_n	0.68	0.042	$C_r=0.003(w_n+7)$	717
	w_L	0.53	0.047	$C_r=0.002(w_L+9)$	678
	e_o and w_L	0.76	0.039	$C_r=0.126(e_o+0.003w_L-0.06)$	678
	e_o and w_n	0.74	0.038	$C_r=0.142(e_o-0.0009w_n+0.006)$	678
	w_n and w_L	0.71	0.040	$C_r=0.003w_n+0.0006w_L+0.004$	678
	e_o, w_n and w_L	0.76	0.037	$C_r=0.135(e_o+0.01w_L-0.002w_n-0.06)$	678

Regression equations used to predict compression index C_c and compression ratio C_r of clays in Greece and U.S.A were shown in Table 2-11.

Al-Khafaji and Andersland, 1992 [11)] performed the comparison of compression index C_c estimated ones using the relationship between compression index C_c and liquid limit w_L and between compression index C_c and initial void ratio e_0 (Lambe and Whitman, 1969 [42)], Harrero, 1980 [43)], Mayne, 1980 [44)]) which were already researched before as shown in Table 2-12.

As a result, the following conclusions were obtained.

i) Equations from past researches for compression index C_c show considerable inconsistencies. Differences are directly attributable to the nature of the data used in the development of empirical relationships.
ii) Often, information pertaining to stress history and soil type are missing. In some cases, slope of laboratory curve is taken as the compression index C_c without correlations relative to disturbance.

Table 2-12 Equations for prediction of Compression Index C_c

(1)	Equation (2)	Applicability (3)	Reference (4)
	(a) $C_c = f(LL)$		
A1	$C_c = 0.007(LL - 7)$	Remolded clays	Skempton (1944)
A2	$C_c = 0.0046(LL - 9)$	Brazilian clays	Cozzolino (1961)
A3	$C_c = 0.009(LL - 10)$	Normally consolidated clays	Terzaghi and Peck (1967)
A4	$C_c = 0.006(LL - 9)$	All clays with liquid limits less than 100%.	Azzouz et al. (1976)
A5	$C_c = (LL - 13)/109$	All clays	Mayne (1980)
	(b) $C_c = f(w_n)$		
B1	$C_c = 0.85\sqrt{(w_n/100)^3}$	Finnish muds and clays	Helenelund (1951)
B2	$C_c = 0.0115w_n$	Organic soils, peat, organic silt, and clay	Moran et al. (1958)
B3	$C_c = 0.01(w_n - 5)$	All clays	Azzouz et al. (1976)
B4	$C_c = 0.01w_n$	All clays	Koppula (1981)
B5	$C_c = 0.01(w_n - 7.549)$	All clays	Herrero (1983)
	(c) $C_c = f(e_0)$		
C1	$C_c = 0.54(e_0 - 0.35)$	All clays	Nishida (1956)
C2	$C_c = 0.29(e_0 - 0.27)$	Inorganic, cohesive soil, silt some clay; silty clay; clay	Hough (1957)
C3	$C_c = 0.35(e_0 - 0.50)$	Organic, fine-grained soil, organic silt, little clay	Hough (1957)
C4	$C_c = 0.43(e_0 - 0.25)$	Brazilian clay	Cozzolino (1961)
C5	$C_c = 0.75(e_0 - 0.50)$	Soils with low plasticity	Sowers (1970)

Note: e_0 = in-situ void ratio; LL = liquid limit; and w_n = natural water content.

2.3 Shear Strength and Shear Strength Ratio of Cohesive Soils

There are many past studies on the shear strength ratio of the clay. In Japan, it is said that Shibata and Tagawa's study, 1969 [14)] is relatively old one.

In case of construction of a structure on soft clay, if the bearing capacity is insufficient, a construction method expecting an increase in strength due to consolidation of the ground is taken. It is needed to increase the strength of clay with respect to consolidation pressure p', which is usually called the c_u/p' value. The problem with this c_u/p' value is that its value varies even with the same clay depending on the method of shear test. That is, according to the in-situ vane shear test results, the c_u/p' value increases as the plasticity index I_P increases, but according to the triaxial compression test results, the tendency is totally reversed. By chance, in the range of I_P = 30 to 80% in clays in Japan, since the $c_u/p' \sim I_P$ curves intersect, there is no big doubt and a value of around c_u/p'= 0.3 is adopted, but it varies depending on the testing method.

Therefore, in order to compare the c_u/p' value obtained mainly based on the vane shear test results, K_0 triaxial consolidated specimens were used, and in addition to this, vane shear test of isotropic consolidated specimen and also studied the issues on c_u/p' obtained from unconfined compressive strength. Index properties of those samples are shown in Table 2-13 and. As a result, the following conclusions are drawn.

i) The Skempton-Bjerrum line showing the relationship between the c_u/p' value obtained from the in-situ vane shear test and I_P was almost consistent with the vane shear test results for K_0 triaxial consolidated specimen. That is, the c_u/p' value by the vane shear test becomes larger as the high plasticity clay. However, when compared with the c_u/p' value estimated from the relation shown by Skempton and Bjerrum, the measured values are larger than that for low plasticity soil and smaller for high plasticity soil as shown in Figure 2-26.

ii) c_u/p' value obtained from the vane shear test results for K_0 triaxial consolidated specimens, some of which were extremely low depending on the samples. A remarkable difference from other samples is that the coefficient of consolidation c_v is small, but it is unknown whether the cause is related to the magnitude of c_v value or coefficient of permeability k. In these samples, there is also a large difference between the vane shear test results for the isotropic consolidated specimens and the triaxial compression test results as shown in Figure 2-27 and Figure 2-28.

iii) Significant tendency of the c_u/p' values with respect to I_P obtained from both the vane shear test and triaxial compression test could not be found. Rather it is better to conclude that the c_u/p' value is invariant regardless of the type of soil. In addition, although the relationship between the c_u/p' value calculated from triaxial test results and I_P are well cited, considering the calculation process of this, as the internal friction angle $\phi'$of the effective stress indication is given by ϕ'_{max}, it should be noted that c_u/p' does not correspond to the maximum shear stress τ_{max}. It was found that when c_u/p' of ϕ'_{max} was corrected to the value at τ_{max}, it decreases by about 30% in sensitive low plastic soil.

iv) As usually done in Japan, the c_u/p' obtained from unconfined compression test results has a wide range of variation as shown in Figure 2-29, so there is a problem in interpretation of the result. From now on, in-situ tests such as vane shear test, etc. are expected to replace unconfined compression test.

Table 2-13 Index Properties of Test Samples

Sample No.	*LL* %	*PL* %	*PI* %	<2μ %	Activity	c_v cm²/s
1	36.0	25.6	10.4	23.0	0.45	2.6×10^{-3}
2	63.5	27.4	36.1	18.5	1.95	3.5×10^{-4}
3	52.5	23.6	28.9	18.0	1.61	
4	47.0	27.9	19.1	8.0	2.38	
5	107.0	30.6	76.4	9.5	8.04	
6	82.6	24.5	58.1	27.0	2.15	1.7×10^{-5}

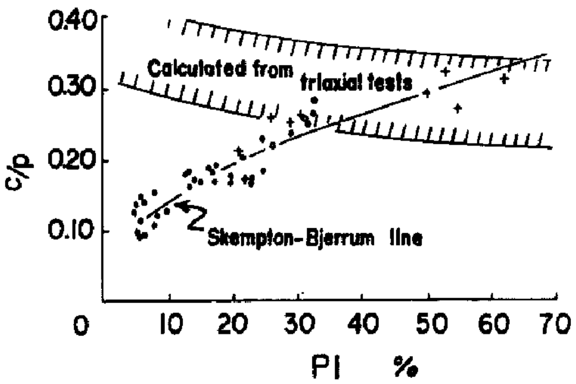


Figure 2-26 Relationship between *c/p* and Plasticity Index *PI*

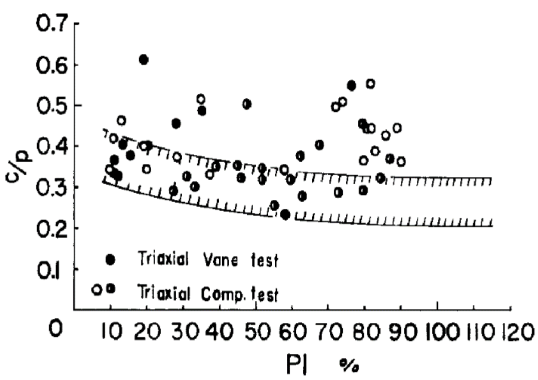


Figure 2-27 Relationship between *c/p* and *PI* (Isotropically consolidated samples)

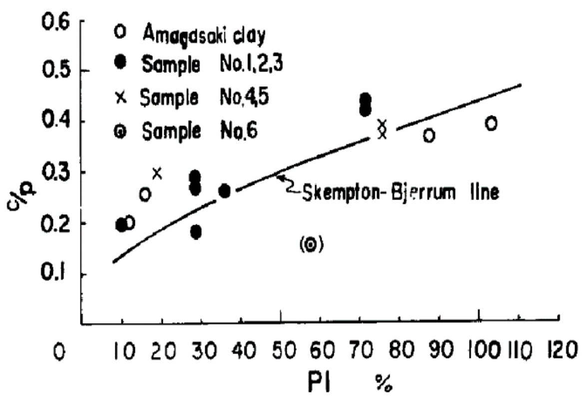


Figure 2-28 Relationship between *c/p* and *PI* (K_0-consolidated vane-triaxial test)

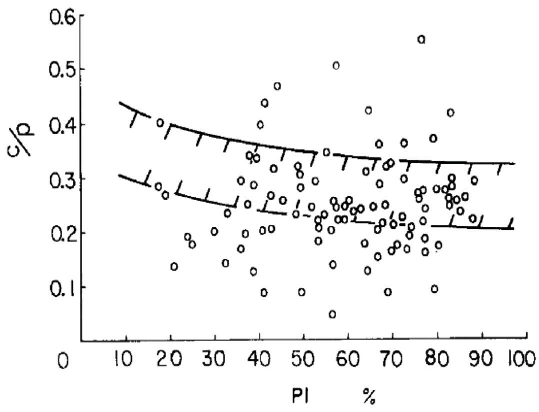


Figure 2-29 Relationship between *c/p* and Plasticity Index *PI* (UCT)

Tanaka et al., 1994 [18)] examined the strength of clays based on the results of the vane shear test conducted at seven points in the Japanese port and harbor areas where marine clay is thickly deposited. The locations of soil investigation points and soil properties for each point area shown in Figure 2-30 and Table 2-14 respectively. As a result, the following conclusions are drawn.

i) Researchers in Europe and the United States have reported that the value of $s_{u(v)}/\ p_c'$ obtained by normalizing vane shear strength with consolidation yield stress increases with increasing of plasticity index I_P. However, according to the study conducted in this paper, it was found that the value of $s_{u(v)}/\ p_c'$ of the clays in Japan have the value of 0.25 to 0.35 regardless of I_P value as shown in Figure 2-31.

ii) The laboratory vane shear test was conducted with the sample in the sampling tube, therefore the strength of such tests are not affected by disturbance after pushing out of samples.

iii) The strength obtained by the above laboratory vane shear test were almost consistent with the in-situ vane shear strength.

iv) The ratio between the vane shear strength and the $q_u/2$ obtained from the unconfined compression test is somewhat influenced by plasticity index I_P, but its value is 0.9 to 1.3, and the vane shear strength is slightly larger as shown in Figure 2-32.

v) Applying the method of correcting $s_{u(v)}$ by the correction factor proposed by Bjerrum, 1954 [50)] to Japanese clay considerably underestimates the shear strength of the clay with large I_P as shown in Figure 2-33.

Table 2-14 Soil Properties of Investigation Points

Investigation Point	Depth (m)	Soil Type	w_n (%)	Ip	IL	OCR	s_u[1] (kPa)
久里浜（KRM）	0−30	clay	50−100	40−75	0.6−0.8	1.2−1.4[2]	20−70
八郎潟（HCG）	0−12 12−25 25−42	clay clay clay	150−210 105−150 50−100	100−150 75−110 80−85	0.7−0.9 0.8−0.9 0.6−0.8	1.0−1.2[3] 1.0−1.3[3] 1.2[2]	15−20 15−30 30−40
出雲（IZM）	0−9 9−32	sand clay	70−110	65−110	0.5−0.8	1.0−1.2[3]	20−45
桑名（KWN）	0−10 10−24	sand clay	45−75	30−60	0.5−0.8	1.3−1.6[3]	50−85
東扇島（OGS）	0−5 5−20	sand clay	50−100	25−80	0.7−1.0	1.8−2.0[3]	15−60
東雲（SNM）	0−14 14−25	sand clay	50−75	20−40	0.6−0.9	1.4[3]	55−100
玉野（TMN）	0−14	clay	30−100	30−60	0.9−1.3	1.5−2.5[3]	15−30

Remarks
1) In-situ Vane Test
2) Standard Consolidation Test
3) Constant Rate Consolidation Test

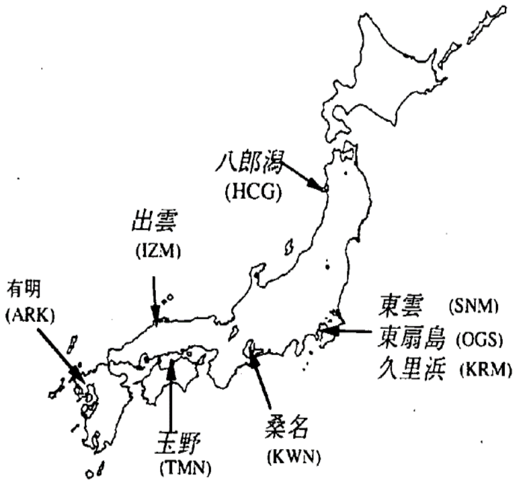


Figure 2-30 Location of Soil Investigation Points

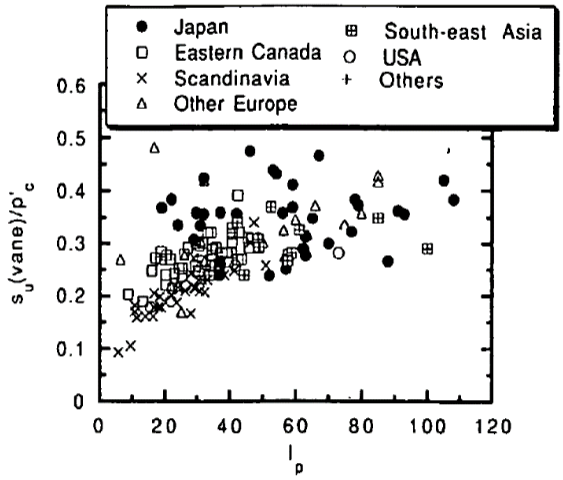


Figure 2-31 Strength ratio from the data by Leroueil et al. and the data of this study

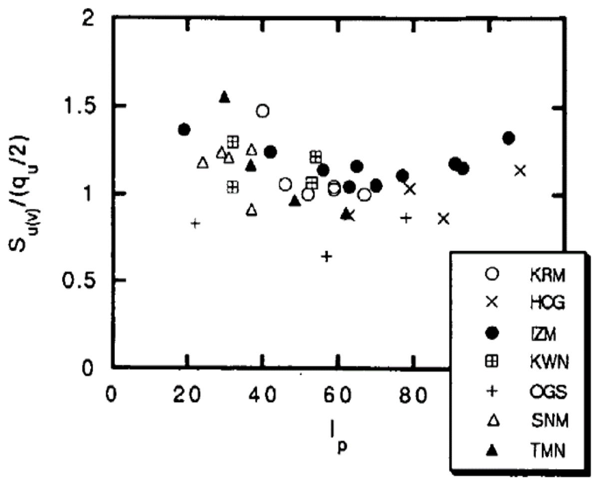


Figure 2-32 Comparison between Vane shear strength and $q_u/2$

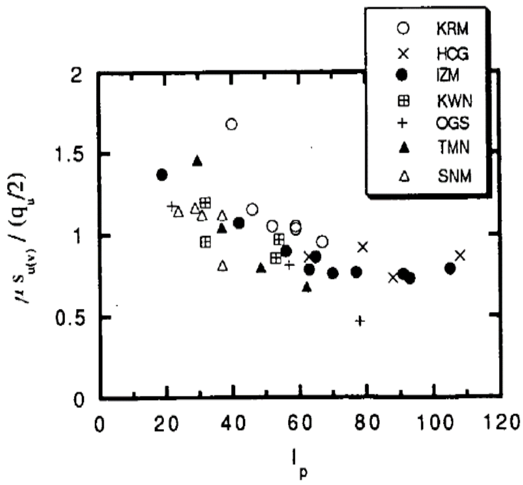


Figure 2-33 Comparison between Bjerrum method and q_u method

vi) Mesri, 1975 [45] has set the shear strength ratio of young alluvial clay to 0.22 regardless of I_P, but in actual construction projects in Japan, it adopts 1/4 or 1/3 value.

vii) It is believed that this difference was caused by the fact that Mesri led the shear strength ratio based on the relationship of soils strongly affected by glaciers such as in Northern Europe and North America.

Fujikawa and Takayama, 1990 [15] also conducts a study based on soil investigation and laboratory test results on the shear strength ratio of Ariake marine clay. Representative soil properties are shown in Figure 2-34. The results of their study are as follows.

i) c_u/p' value can be regarded that it has some relations of plasticity index I_P, liquidity index I_L, sensitivity ratio S_t, and it was shown that it cannot be regarded as a function of I_P only. That is, it cannot be determined only by the type of clay because the e-log p relationship is different due to the difference in loading method and loading speed and the sensitivity ratio which differs depending on the elapsed time after consolidation. Therefore, it is due to that p' (or e) ~ c_u relation is not uniquely determined.

ii) c_u/p' by natural consolidation can be regarded as a function of I_P (or w_L) as the first approximation regardless of the difference in the main clay mineral, and it increases with increasing of I_P (or w_L).

iii) On the other hand, c_u/p' obtained by laboratory tests tends to converge to a constant value while decreasing with increasing of I_P (or w_L), but in ordinary Ariake clay ($30 < I_P < 90$), c_u/p' is almost constant as $c_u/p' = 0.3 \sim 0.4$ as shown in Figure 2-35.

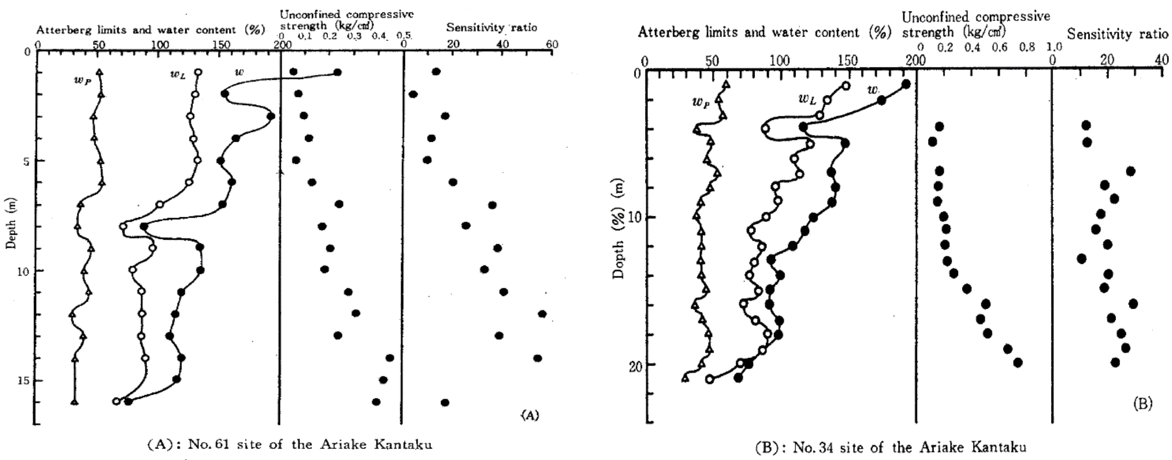


Figure 2-34 Representative soil properties of Ariake marine clay

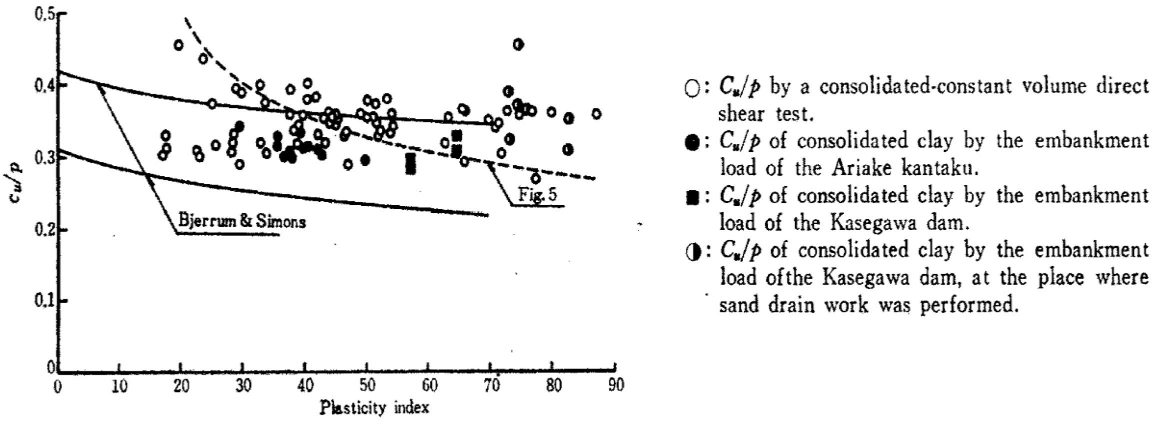
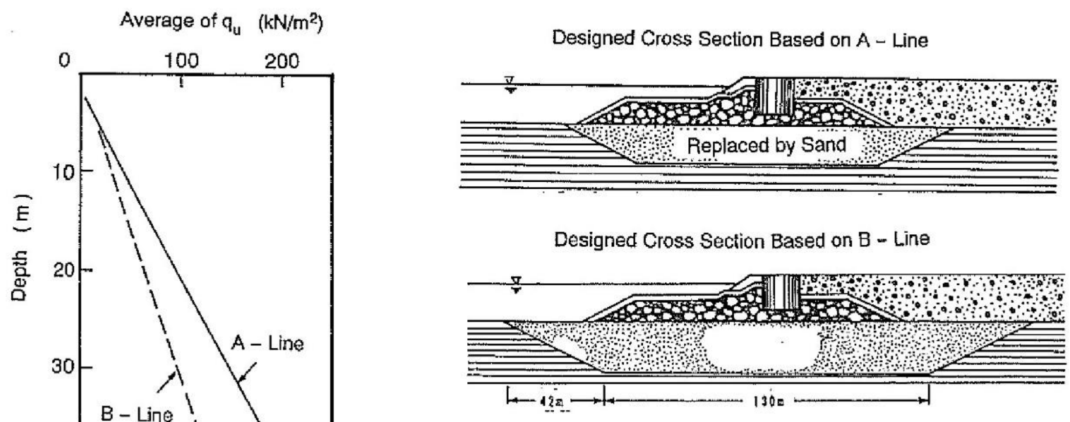


Figure 2-35 Relation between c_u/p' and plasticity index I_P for Ariake marine clay

Tsuchida and Tanaka, 1995 [17)] reviewed the shear strength determination method (q_u method) based on the current unconfined compressive strength and clarified the background and problems of the current shear strength determination method (q_u method). They clarified the relation between the strength obtained by the q_u method and the strength obtained by other methods for determining the shear strength and showed the applicability of other shear strength determination methods to marine clay in Japan. Furthermore, they proposed a practical method to solve the problem of q_u method and clarified the effectiveness of their proposed method by showing application examples with importance of q_u quality (Refer to Figure 2-36 and Figure 2-37).



Strength	Volume of Dredged Soil (m³/m)	Cost for Ground Improvement (thousands yen / m)
A-line	2,000	4,200
B-line	3,600	7,400

Figure 2-36 Cross Sections of Revetments Designed for the Two q_u-depth relations (Case-1)

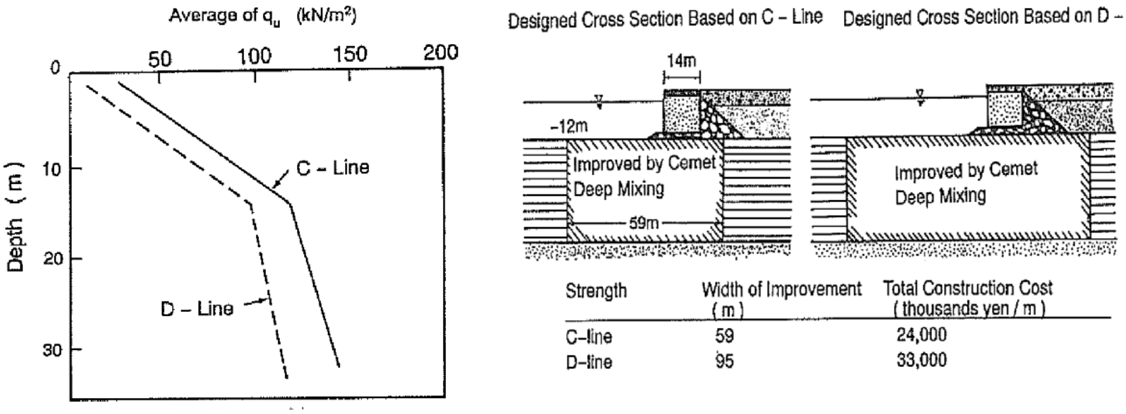


Figure 2-37 Cross Sections of Revetments Designed for the Two q_u-depth relations (Case-2)

2.4 Coefficient of Secondary Consolidation

When considering the long-term settlement, we always come across the issue how to set the coefficient of secondary consolidation, and as for this issue, too, there are many past studies from long time ago.

Oohara, Matsuda and Aboshi, 1985 [24)] took up the preloading method as one of the construction methods to suppress the residual settlement caused by the secondary consolidation that occurs after the construction of the structure. Then they clarified that the coefficient of secondary consolidation is uniquely determined by the effective over-consolidation ratio (σ'/σ_f, σ': the effective stress σ' at the time of unloading of preload, σ_f : the final load). The relation between the coefficient of secondary consolidation and the effective over-consolidation ratio was found for some kinds of clay, and as a result, the following conclusions are derived.

i) In the pre-consolidation method, if the effective over-consolidation ratio is 1.0 or less, the settlement suppression effect cannot be expected sufficiently as shown in Figure 2-38.
ii) The coefficient of secondary consolidation of pre-consolidated clay is smaller as the effective over-consolidation ratio is larger, and this tendency is prominent for the clay having large plasticity index (I_P> 30 to 40) as shown in Figure 2-39.
iii) In order to sufficiently expect the effect of suppressing the residual settlement due to the secondary consolidation after the construction of the structure by the pre-consolidation method, the size of the preload and the consolidation degree at the time

of surcharge removal shall be set to obtain the effective over-consolidation ratio about 1.8 as shown in Figure 2-38 and Figure 2-39.

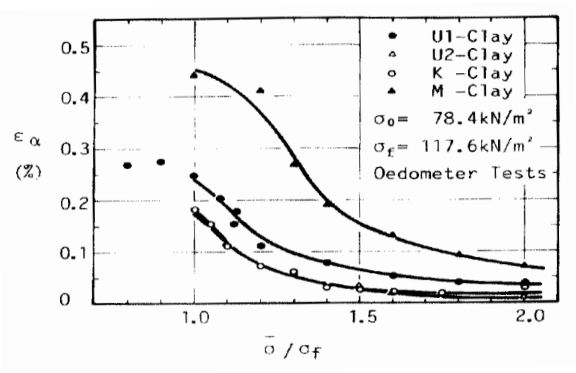


Figure 2-38 Relationships between ε_α and σ'/σ_f

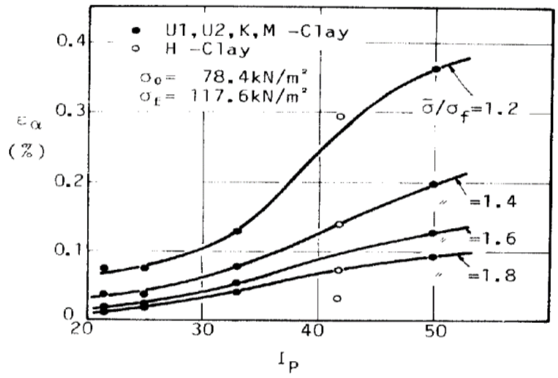


Figure 2-39 Relationships between ε_α and I_P

In addition, Kanayama and Hiyama et al., 2001 [20] carried out consolidation tests by step loading for remolded clays which have different main clay minerals and undisturbed Ariake clay in order to grasp the characteristics of the coefficient of secondary consolidation $C_{\alpha\varepsilon}$. Physical properties of remolded clays and undisturbed Ariake clays are shown in Table 2-15 and Table 2-16 respectively.

As a result, the following conclusions are obtained.

Table 2-15 Physical properties of remolded clays

Sample	ρ_s (g/cm^3)	w_L (%)	w_p (%)	w_0 (%)	I_{L0}	Salt Concent. (%)
Bentonite	2.594	269.9	35.4	81.3	0.20	0
	—	93.6	40.9	79.8	0.74	3.76
Ariake Clay	2.642	110.3	45.8	96.8	0.79	0.03
	—	130.3	46.9	94.8	0.57	3.80
Cericite	2.835	55.8	28.5	47.9	0.71	0
	—	55.3	30.0	43.8	0.55	3.73
Kaorinite	2.650	71.5	27.7	70.1	0.83	0
	—	67.5	27.3	68.0	1.19	3.99
Aso Loam	2.707	115.6	51.6	106.8	0.86	0
	—	118.7	52.4	104.3	0.78	3.80

* w_0: Initial moisture content at Consolidation Test
IL0: Initial moisture content at w = w0

Table 2-16 Physical properties of undisturbed Ariake clays

Sample No.	Depth (m)	ρ_s (g/cm^3)	Grading (%) Clay	Silt	Sand	w_L (%)	w_p (%)	w_n (%)	Salt Concent. (%)
T-1	2.41	2.657	61.0	32.1	6.9	116.3	46.2	156.7	2.54
T-2	4.41	2.658	52.0	44.7	3.3	95.0	41.1	124.9	2.33
T-3	6.43	2.642	58.0	41.0	1.0	105.4	43.4	131.5	1.90
T-4	8.42	2.639	60.0	37.5	2.5	100.2	45.4	125.6	1.68
T-5	10.42	2.626	57.0	38.8	4.2	89.2	41.3	111.5	1.47
T-6	12.41	2.650	62.0	36.2	1.8	92.6	37.9	107.3	1.43
T-7	14.42	2.663	47.0	50.8	2.2	72.2	35.2	84.3	1.49
T-8	16.41	2.675	56.0	43.2	0.8	81.4	35.9	94.1	1.33
T-9	18.34	2.681	59.0	40.4	0.6	85.7	37.0	96.2	1.62
T-10	20.40	2.693	55.0	44.3	0.7	86.8	33.6	83.2	1.38
T-11	22.36	2.671	61.0	37.4	1.6	82.5	32.8	70.8	1.49
No.54	9.85	2.676	49.4	42.3	8.3	89.9	39.6	105.3	–
K-24	24.10	2.657	45.6	36.5	17.9	81.7	35.6	88.4	–
A-13	13.80	2.655	49.9	48.7	1.4	79.1	37.8	86.4	–

(A) Remoled clay

i) The coefficient of secondary consolidation of kaolin, Aso loam, sericite were almost constant with respect to consolidation pressure, but relationship between the secondary consolidation and consolidation pressure of bentonite and Ariake clay became convex upward curve, it did not become constant as shown in Figure 2-40.

ii) The ratio between $C_{\alpha\varepsilon}$ and C_c, $C_{\alpha\varepsilon}/C_c$ was nearly almost constant with decreasing I_{LS} (I_{LS}: Liquidity Index at the beginning of straight line of S~log t curve) in case of kaolin. In the case of Aso loam and sericite, $C_{\alpha\varepsilon}/C_c$ showed a sharp decrease with decreasing I_{LS} at the initial consolidation stage, but after that, $C_{\alpha\varepsilon}/C_c$ remained nearly constant against decrease of I_{LS}. On the other hand, $C_{\alpha\varepsilon}/C_c$ of bentonite and Ariake clay decreased almost linearly with decrease of I_{LS} as shown in Figure 2-41.

iii) The coefficient of secondary consolidation of remolded clay does not have a certain tendency but varied depending on the main clay mineral of the clay. The range of $C_{\alpha\varepsilon}/C_c$ of the remolded clay was approximately 0.01 to 0.07 as shown in Figure 2-41.

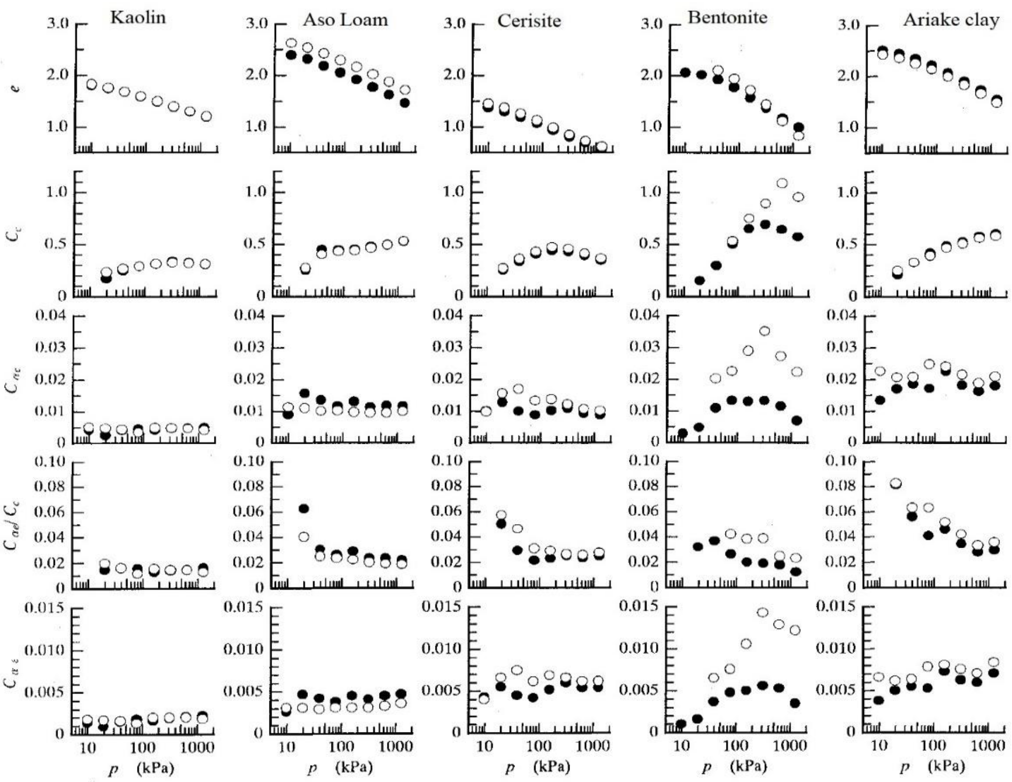


Figure 2-40 Consolidation test results and coefficient of secondary consolidation for remolded clay

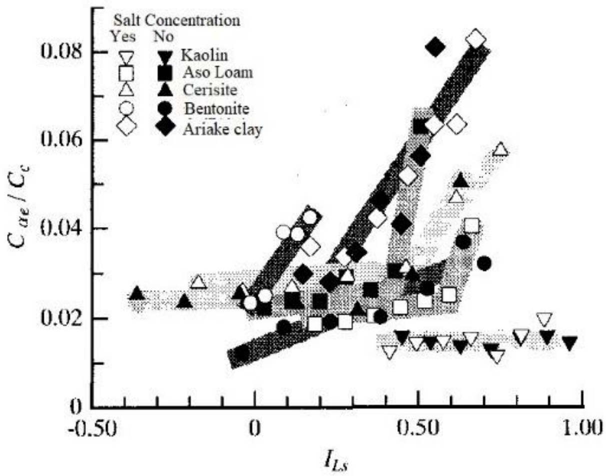


Figure 2-41 Relationship between $C_{\alpha\varepsilon}/C_c$ and I_{LS} for remolded clays

(B) Undisturbed Ariake clay

i) The coefficient of secondary consolidation of the undisturbed Ariake clay showed its maximum value at consolidation yield stress p_c or slightly larger p and tends to decrease when p increases as shown in Figure 2-42.

ii) $C_{\alpha\varepsilon}/C_c$ of the undisturbed Ariake clay was almost constant for consolidation pressure higher than p_c. The range of $C_{\alpha\varepsilon}/C_c$ of Ariake clay was between 0.03 and 0.05 as shown in Figure 2-42, which was within the range pointed out by Ladd et al., 1977 [46)].

iii) It was found that a linear relationship was established between the coefficient of secondary consolidation $C_{\alpha\varepsilon}$ and the void ratio e_s at the time of the secondary consolidation started, and the coefficient of secondary consolidation decreased with decreasing e_s as shown in Figure 2-43 and Figure 2-44.

iv) The drainage condition and the load increase ratio had little influence on the coefficient of secondary consolidation and secondary consolidation properties such as $C_{\alpha\varepsilon}/C_c$.

v) As a result of the long-term consolidation tests of about 10^5 minutes, the coefficient of secondary consolidation decreases with the elapsed time from about 10^2 to 10^3 minutes, but it was confirmed that it became almost constant as shown in Figure 2-45.

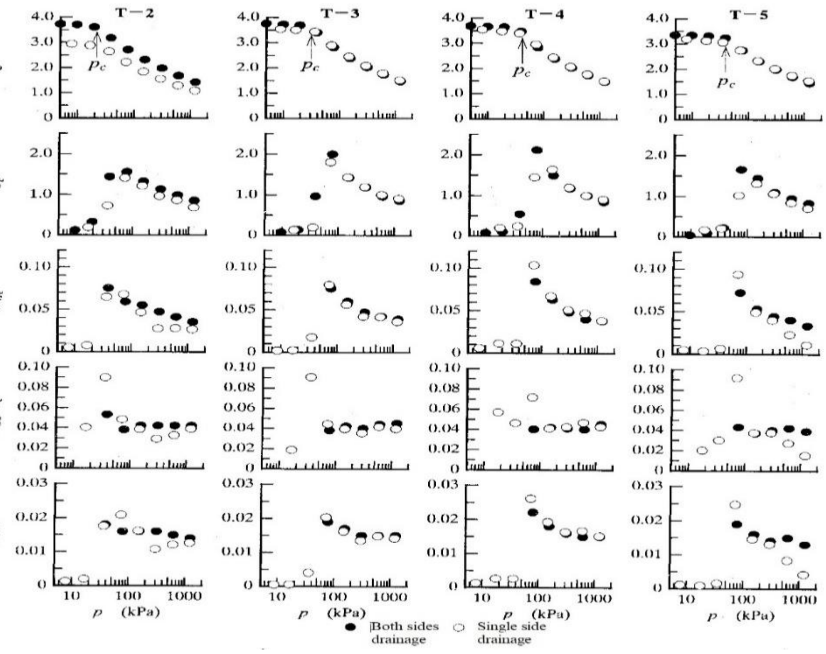


Figure 2-42 Relationship between $C_{\alpha\varepsilon}/C_c$ and I_{LS} for remolded clays

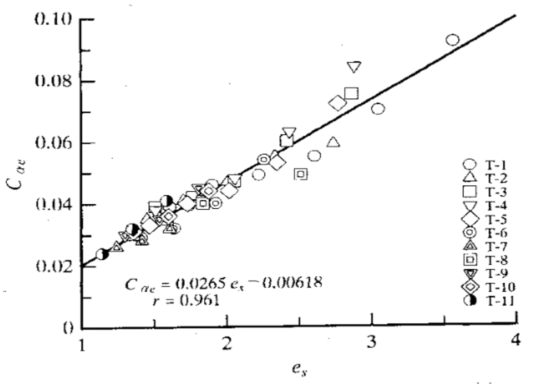
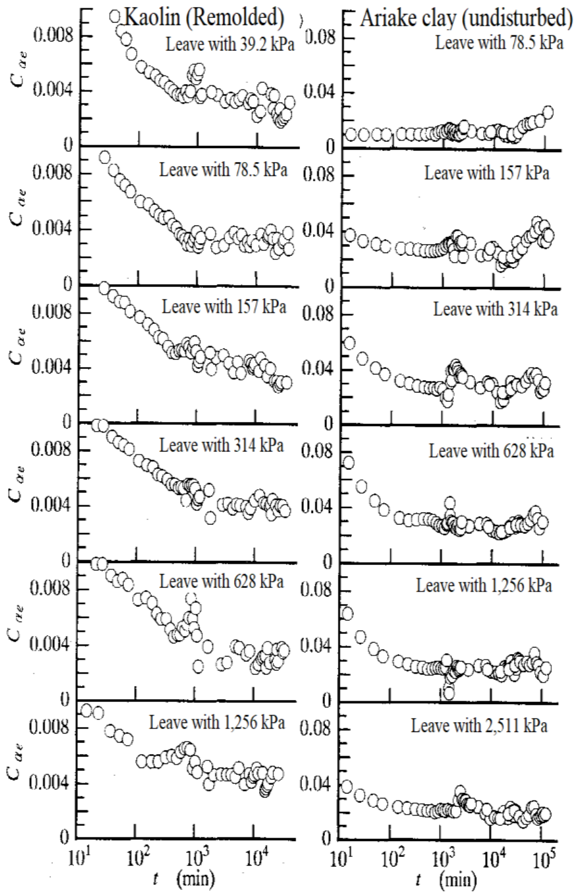


Figure 2-43 Relationship between $C_{\alpha\varepsilon}$ and e_0 for undisturbed Ariake clay



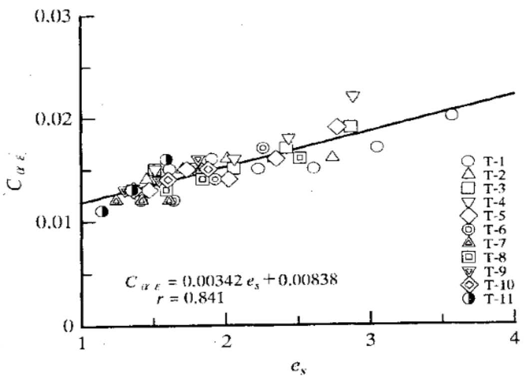


Figure 2-45 Relationship between $C_{\alpha\varepsilon}$ and e_0 for undisturbed Ariake clay

Figure 2-44 Relationship between $C_{\alpha\varepsilon}$ and elapsed time t

Furthermore, Aboshi, 1995 [22] shows the long term progress of the secondary consolidation settlement based on the long-term settlement monitoring results with different diameter of test ring as shown in Table 2-17 and Figure 2-46. According to this monitoring results, the coefficient of secondary consolidation is becoming gradually smaller with very slow speed as shown in Figure 2-47.

Table 2-17 Several specimen sizes for long-term settlement monitoring

Specimen Size

Size Ratio	1	2.4	10	20	50
D cm	6	14.4	60	120	300
H cm	2	4.8	20	40	100

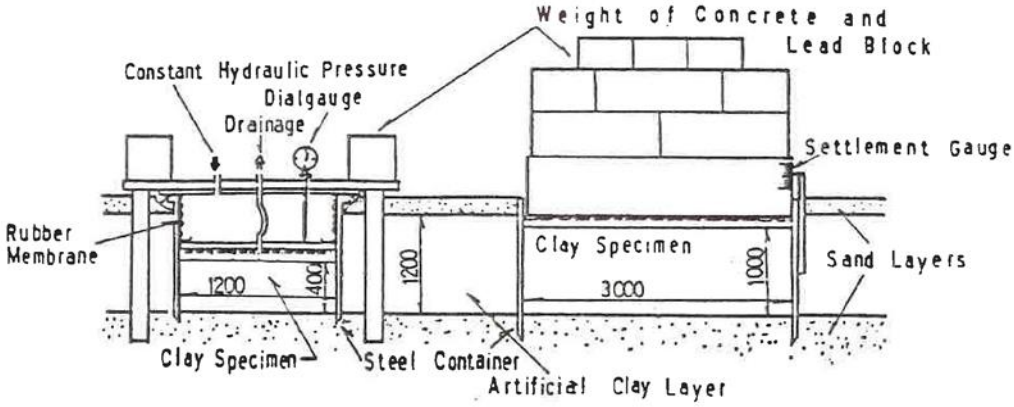


Figure 2-46 Large scale consolidation test

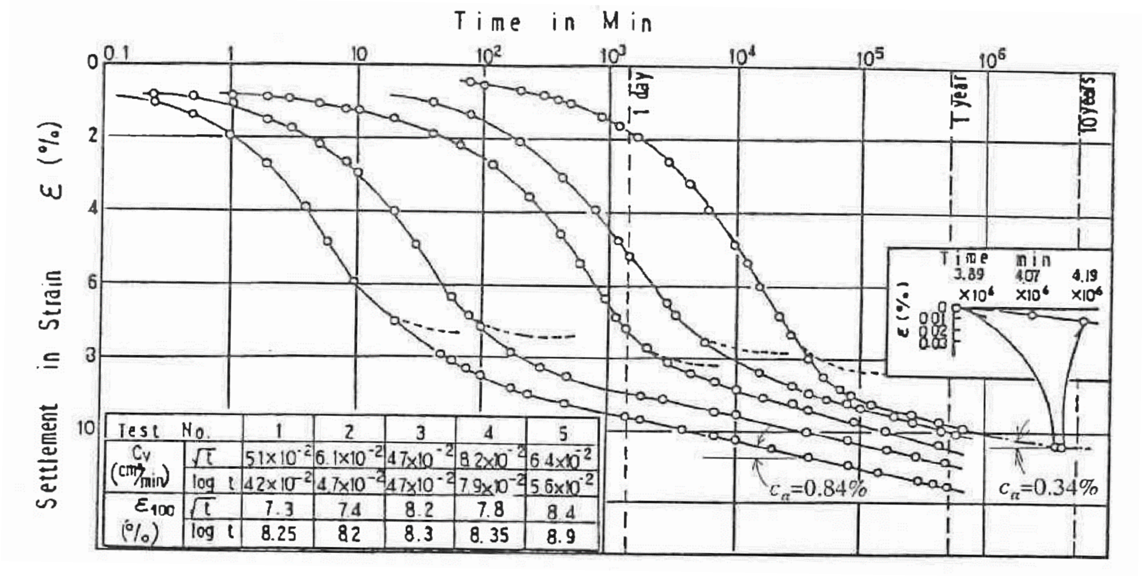


Figure 2-47 Comparison of settlement curves monitored for long term with different specimen sizes

In addition, Kumamoto and Tsuchida et al., 2016 [21] studied the calculation methods for prediction of secondary consolidation settlement using actual monitored data for 30 years at Western Development of Hiroshima City and for 20 years at Hiroshima Eastern Water Treatment Center. Representative ground conditions at Hiroshima Western Development Project site are shown in Figure 2-48. As a result, the following conclusions are drawn.

i) As for the Western Development of Hiroshima City project, the area can be classified into two areas where ground was improved by sand drain method and non-improved. It has already 30 years passed, and those settlement rates are 0.27 cm/year in improved area and 0.53 cm/year in non-improved area. The settlements of the ground were almost calm as shown in Figure 2-49.

ii) There are some discussions that the coefficient of secondary consolidation decreases with the elapsed time, but the coefficient of secondary consolidation has remained constant over a long period of time for 30 to 40 years above as shown in Figure 2-50.

iii) The coefficients of secondary consolidation defined by the void ratio e of the clay layer calculated from the actual monitored data of Western Development of Hiroshima City project were distributed between 0.03 and 0.09, and the most frequent values was 0.06 as shown in Figure 2-51.

iv) The relationship between the compression index C_c and the coefficient of secondary consolidation $C_{\alpha\varepsilon}$ was obtained from the standard consolidation test results of the clay samples collected in Hiroshima Port close to the project site as shown in Figure 2-52. Using the relationship obtained, the $C_{\alpha\varepsilon}$ values estimated from the compression index C_c for clays of Western Development Hiroshima City project site almost agreed with the values obtained from the monitored settlement there as shown in Figure 2-53.

v) The coefficient of secondary consolidation $C_{\alpha\varepsilon}$ obtained by multiplying the coefficient of secondary consolidation estimated from the compression index C_c obtained from the consolidation tests before construction by the settlement reduction factor used in the design of the low-replacement Sand Compaction Pile (SCP) method almost correspond with the coefficient of secondary consolidation obtained from the actual monitored data.

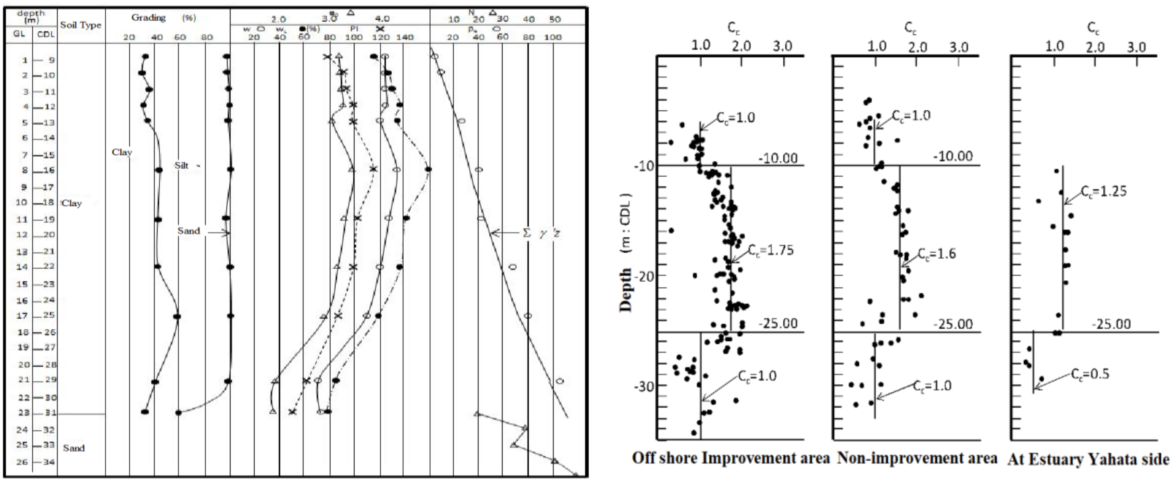


Figure 2-48 Representative ground conditions at Hiroshima Western Development Project Site

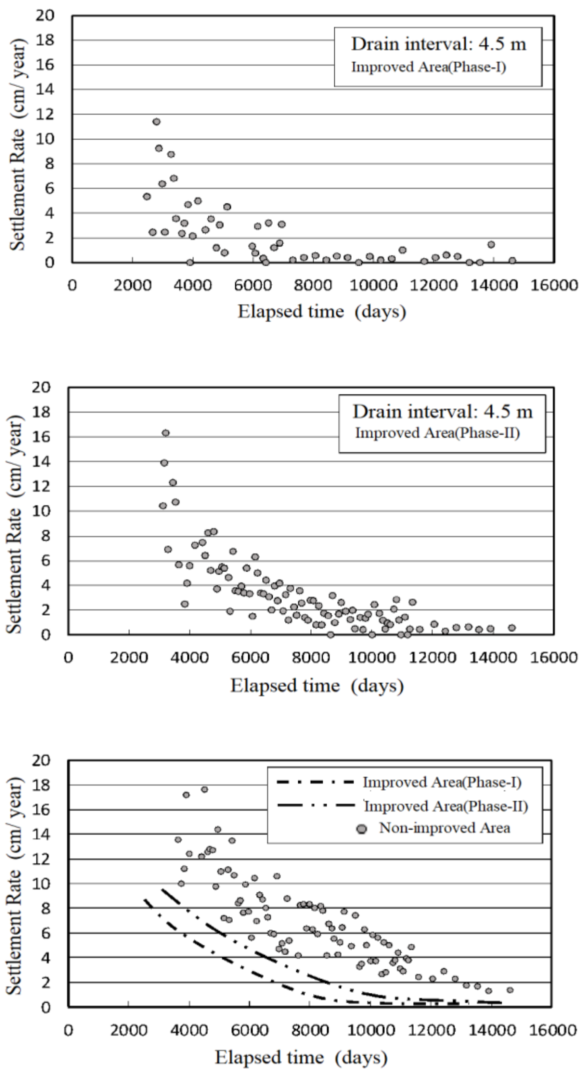


Figure 2-49 Settlement rate ~ Elapsed Time t

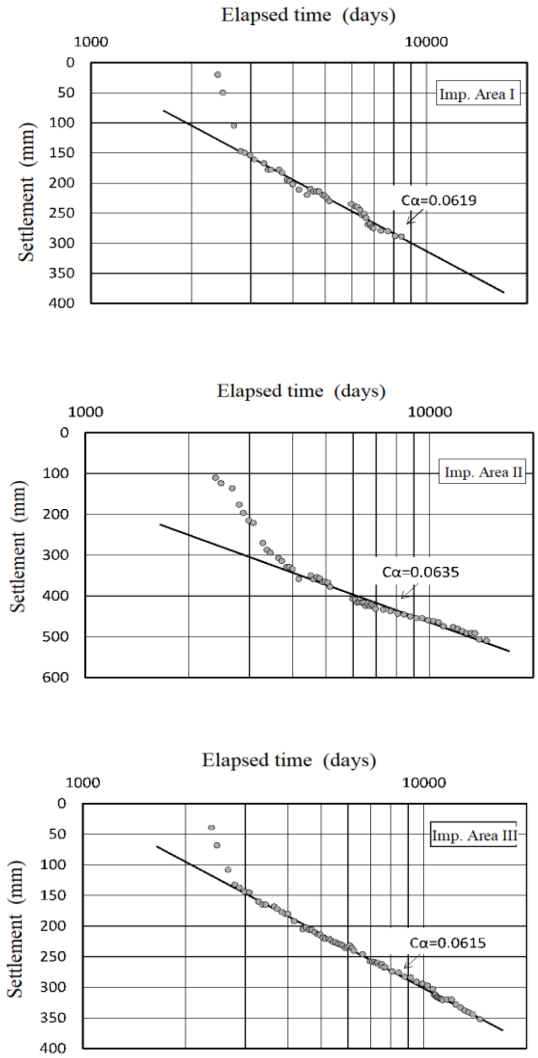


Figure 2-50 Settlement rate ~ Elapsed Time t

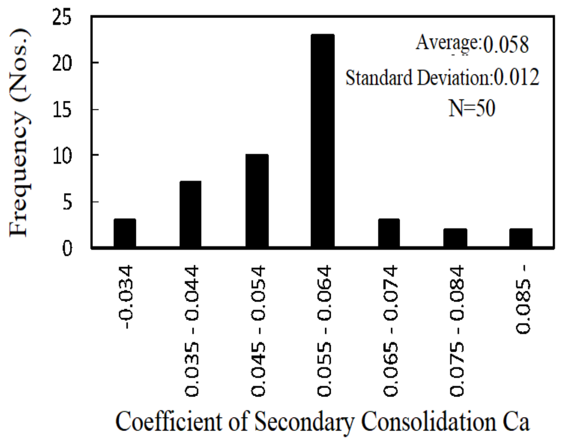


Figure 2-51 Histogram of Coefficient of Secondary Consolidation obtained from long-term monitored data

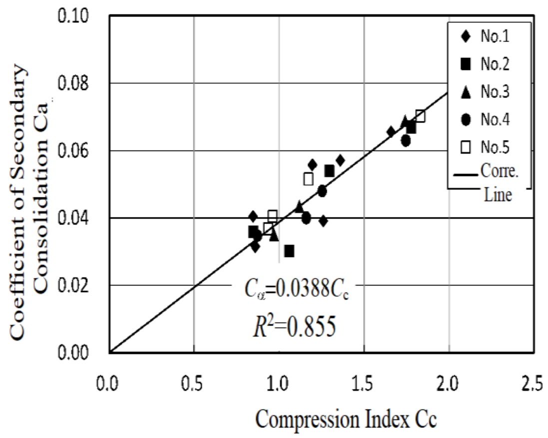


Figure 2-52 Relationship between Coefficient of Secondary Consolidation C_α and Compression Index C_c

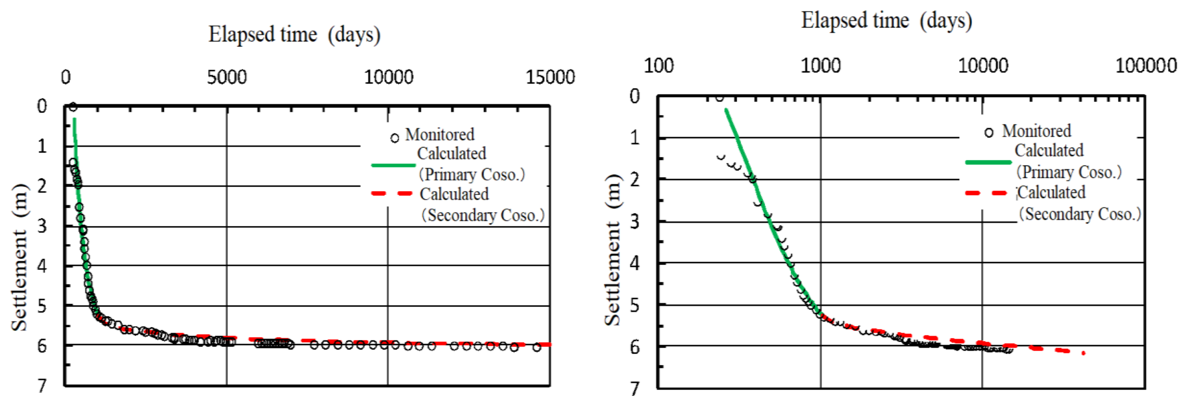


Figure 2-53 Comparison between calculated and monitored settlement

Shirako and Tonosaki et al., 2002 [23)] studied the long-term settlement and secondary consolidation of soft ground at the road and residential land preparation sites. They studied the relationship between the long-term settlement and natural moisture content w_n at the road embankment and the residential land preparation sites on soft ground, then they showed that the settlement at road embankment site appears greater. The main reason of this is considered to be that the long-term settlement at the road embankment included the settlement caused by shear deformation. Therefore, larger values than the ordinary coefficient of secondary consolidation $C_{\alpha\varepsilon}$ estimated by the natural moisture content w_n will be derived by adding the shear deformation amount on it.

2.5 Relations between Clay Minerals and Physical Properties

Although there are not so many past studies on the relationship between clay mineral and physical properties, there is a study by Kamon and Kawamae, 1997 [28] that discussed the correlation between physical property parameters of diluvial marine clay and clay minerals.

They estimated the distribution of clay minerals in the horizontal and vertical directions at seabed of the Osaka Bay by conducting X-ray analysis for mineral identification and quantification. In addition, the correlation between clay mineral composition and physical properties is widely studied. As a result, the following conclusions are drawn.

i) Physical properties and clay content were found to have a significant correlation. Liquid limit w_L was considered to be largely dependent on the content of fine particulate smectite among clay minerals, but rather the liquid limit w_L tended to increase as the content of medium sized illite increased. As for the plasticity index I_P, the correlation with the content of illite was stronger than smectite as well as liquid limit w_L. By estimating the cation exchange capacity of soil based on the content of clay mineral, the change of physical properties could be qualitatively explained.

ii) There was no clear effects of clay mineral on strength of clay.

iii) Clay content was correlated to some extent with compression index C_c as shown in Figure 2-54. As a result of examining each clay mineral and compression index C_c, it was found that correlation with illite is relatively strong by considering the sedimentary environment. Regarding the coefficient of consolidation c_v and clay content CC, although not so clear, there was a tendency for the coefficient of consolidation c_v to decrease as the clay content CC increased as shown in Figure 2-55. In addition, when clay minerals were individually examined, influence by illite was the largest to the coefficient of consolidation c_v.

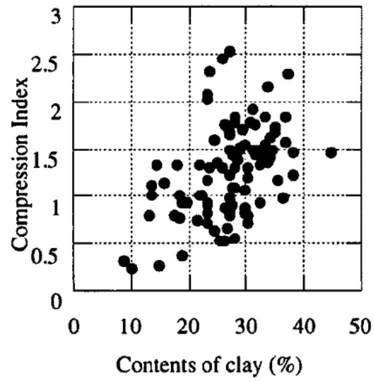


Figure 2-54 C_c ~ CC Relation

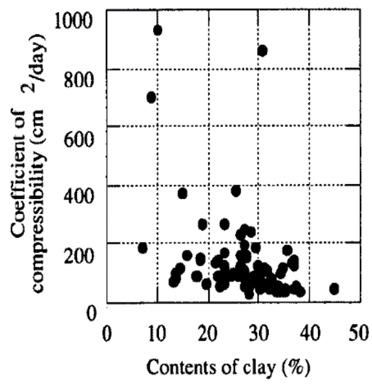


Figure 2-55 c_v ~ CC Relation

Also regarding the smectite among clay minerals, Kitagawa, 2004 [27)] described the characteristics and properties of smectite's chemical structure as follows.

Smectite (montmorillonite) is a 2 to 1 type clay mineral having a structure in which an aluminum octahedral layer is sandwiched in a tetrahedral layer of silicic acid. In the case of smectite, since aluminum is isomorphous substituted by magnesium or divalent iron in the aluminum octahedral layer, negatively charged ions are generated there. Therefore, the distance to the crystal lattice layer tube is long, the retention power of the neutralized cations is weak, then water tends to enter between the layers, and as a result, the layer lattice becomes easy to expand. Smectite has a larger cation exchange capacity and specific surface area than kaolinite.

Maeda and Souma, 1974 [26)] studied on the relationship between the specific surface area of clay particles and the liquid limit w_L. Their contents of the study are as follows.

With clays having a large specific surface area, it is known that clay having a large liquid limit (w_L) and a small specific surface area has a relatively small inter-particle force as compared with clay having a large specific surface area, so that the liquid limit is small. Conversely, particles with a large specific surface area have large inter-particle forces, excluding physicochemical properties such as ionic load and solution concentration of adsorbed salts, the size of the liquid limit depends on the size of the specific surface area.

From specific surface area, what is known is considered referring to the following Table 2-18 and Table 2-19.

Table 2-18 Liquid Limit of Clay (Warketinn Birrell White, 2000 [26)])

Sample Type	Liqud LImit (%)
Kaolinite —Na	52
—Ca	73
Illite —Na	61
—Ca	90
Montmorillonite —Na	700
—Ca	177
Allophane	
— Raw Soil	231
— Air dry soil	85
Na— Montmorillonite	
— Water	950
—0.01 N NaCl	870
—1.0 N NaCl	350
Ca— Montmorillonite	
— Water	360
—1.0 N $CaCl_2$	310
Kaolinite pH 4	
— Water	54
—0.01 N $CaCl_2$	46
—1.0 N $CaCl_2$	39
Na— Kaolinite pH 10	
— Water	36
—0.01 N NaCl	34
—1.0 N NaCl	40

Table 2-19 Size of clay particle and Specific surface area (Yong Warkenth, 2000) [26)]

Clay Mineral	Approximate Thickness (Å)	Maximum Specific Surface Area (m^2/g)	Volume change observed
Montmorillonite	20	800	Big
Illite	200	80	Normal
Kaolinite	1,000	15	Small

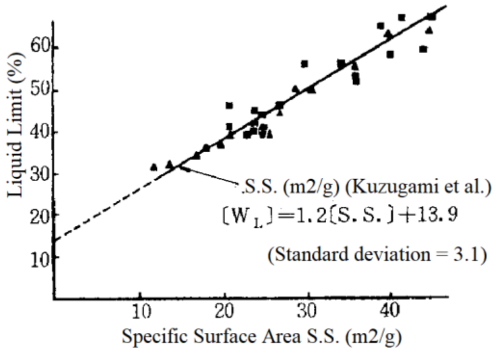


Figure 2-56 w_L ~ S.S. Relation [26)]

Meanwhile, a correlation formula between Specific surface S.S. and Liquid Limit w_L, w_L (%) = 1.2 S.S. (m^2/g) +13.9 was indicated as shown in Figure 2-56.

On the other hand, Yoshimura, 2009 [25)] described the sequential action and clay mineral, the action that clay changes to another kind of clay mineral is a diagenesis and described the change of clay mineral by the diagenesis of clastic sediment. In general, what kind of clay mineral produced is said to be defined by the surrounding environmental conditions, temperature and chemical chemistry and quantity of solution (rock involved in reaction to water ratio) and microbial activity. Since many changes occur near the surface of the ground, earth pressure is regarded as having little concern. Therefore, clay minerals related to the physical properties of clay are generated by a diagenesis occurring in the process from clay depositing to rock formation, and at the same time as the composition of the sediment itself, the water quality and properties give maximum influence to chemical properties of clay minerals (Refer to Figure 2-57).

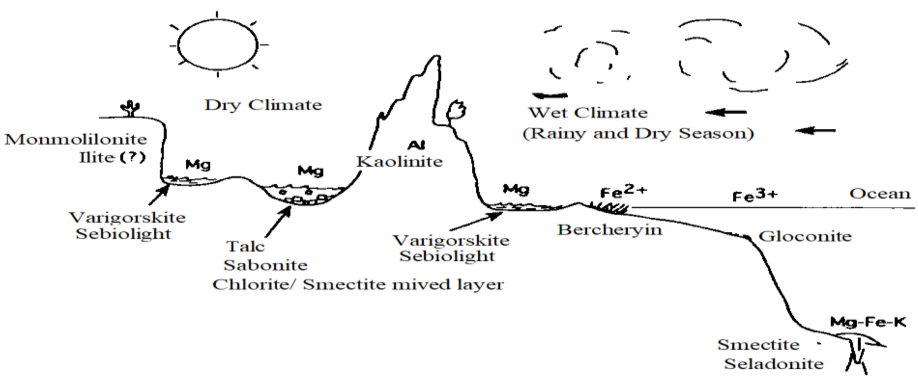


Figure 2-57 Formation environment of spontaneous clay mineral of Fe. Mg near ground surface [25)]

2.6 Effects of Sample Disturbance due to Different Sampling Methods

Tanaka, 2003 [34)] researched the influence of soil investigation methods on physical property evaluation of soil distributed in coastal areas of Japan.

He examined the position of Japanese sampling technology in the world and he studied the test methods and the in-situ tests which do not easily affect the disturbance on soil samples. In addition, focusing on the diatoms contained in the clays in Japan, phenomena which cannot be explained by the concept of usual effective stress such as the cementation and the age effect of clays are quantitatively analyzed using the microscopic structure of soil and the diatom content. As a result, the following conclusions were obtained.

i) Fixed piston type thin wall sampler was developed by the American sampling committee, taking advantage of the piston sampler devised in Scandinavia and the thin wall tube made in the United States. This sampler was introduced in Japan in 1952 and since then many improvements have been made, it has become a sampler with current function. In Japan, the method of sampling using this sampler is strictly determined according to the standards of the Japanese Geotechnical Society (JGS).
ii) In the world, there is a sampler that matches the ground of the country, and sampling methods are set according to sampling technology and laboratory test technique of the country. That is, the sampling method can be decided by the deposition environment of soil and its strength. From this, it is assumed that the undrained shear strength of the ground differs from region to region. For interpretation of shear strength abroad, it is necessary to clarify sampling methods and laboratory test methods.
iii) No matter how carefully takes samples, the effects of stress relief cannot be avoided. For this reason, in-situ tests such as Field Vane Shear Test (FVT), Cone Penetrometer Test (CPT) and Dilatometer Test (DMT) were developed. In the in-situ test, the test method is relatively simple and reproducible, and it is not affected by stress relief, but because drainage conditions at the time of the test are unknown, sufficient consideration is required for the application.
iv) As the diatom content increased, physical properties and mechanical properties changed greatly. Since the presence of diatoms alters the engineering properties of soils, it became clear that the presence of diatom is a factor to determine the regional

characteristics of the soils in the world as shown in Figure 2-58 and Table 2-22. Physical Property test results for those clays are shown in Table 2-20 to Table 2-22.

Table 2-20 Physical property test results of diatomaceous

Sample	Particle Density (g/cm^3)	Sand (%)	Silt (%)	Clay (%)	Liquid Limit (%)	Plastic Limit (%)
Diatom	2.294	0	64.5	35.5	NP	NP

Table 2-21 Physical property test results of clays before mixing with diatomaceous

Sample	Particle Density (g/cm^3)	Sand (%)	Silt (%)	Clay (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Activity
Kaolin Clay	2.775	0	20	80	69	35	34	0.50
Singapore Clay	2.692	1	29	70	85	31	54	1.00
Bangkok Clay	2.730	2	21	77	89	26	62	1.03

Table 2-22 Physical property test results of clays mixing with diatomaceous soil

Kaolin : Diatom K D	Particle Density (g/cm^3)	Sand (%)	Silt (%)	Clay (%)	Liquid Limit (%)	Plasticity Index (%)
K : D=100 : 0	2.775	0	20	80	69	35
K : D=75 : 25	2.664	0	38	62	83	48
K : D=50 : 50	2.557	0	54	46	101	68
K : D=25 : 75	2.472	0	63	37	112	88
K : D=0 : 100	2.374	1	77	22	NP	NP

Singapore clay : Diatom S D	Particle Density (g/cm^3)	Sand (%)	Silt (%)	Clay (%)	Liquid Limit (%)	Plasticity Index (%)
S : D=100 : 0	2.692	1	29	70	85	31
S : D=75 : 25	2.567	2	34	64	96	47
S : D=50 : 50	2.471	2	41	57	NP	NP
S : D=25 : 75	2.384	1	56	43	NP	NP
S : D=0 : 100	2.374	1	77	22	NP	NP

Bangkok clay : Diatom B D	Particle Density (g/cm^3)	Sand (%)	Silt (%)	Clay (%)	Liquid Limit (%)	Plasticity Index (%)
B : D=100 : 0	2.730	2	21	77	89	26
B : D=75 : 25	2.600	3	33	64	109	42
B : D=50 : 50	2.496	2	44	54	NP	NP
B : D=25 : 75	2.386	1	53	46	NP	NP
B : D=0 : 100	2.374	1	77	22	NP	NP

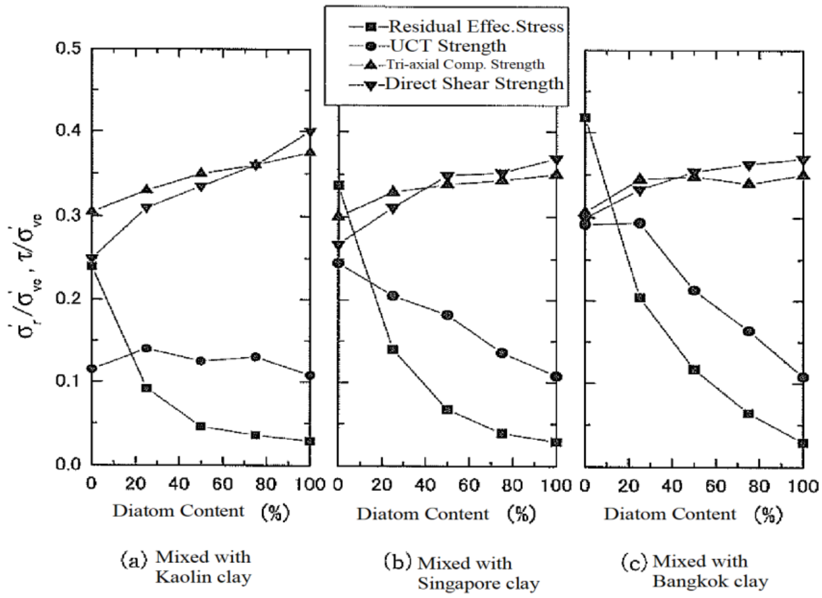


Figure 2-58 Relation between Diatom content and shear strength

v) In the ground containing a large number of diatoms, since the residual effective stress is not retained at the original position, the undrained shear strength obtained from the unconfined compression test may be underestimated, and it is necessary that undrained shear strength by unconfined compression test need to be compared and studied with ones from ones by the direct shear test under confined pressure or triaxial compression test. In order to carry out significant design for ground containing a large number of diatoms, it is essential to clarify the diatom content and to select appropriate test methods and comprehensively evaluate the shear test results with consideration as shown in Figure 2-58.

vi) Shear strength greatly differs depending on the boring method and sampling method. When comparing the shear strength of soil, it is necessary to clarify the boring method and the sampling method. The rotary boring is less likely to disturb the ground than the displacement boring and the wash boring, so that it is possible to collect a good quality sample. The boring method and sampling method in Japan can take good sample samples compared to the other samplers in the world, so even if it is internationalized it will not need to change that way significantly as shown in Figure 2-59 and Table 2-23.

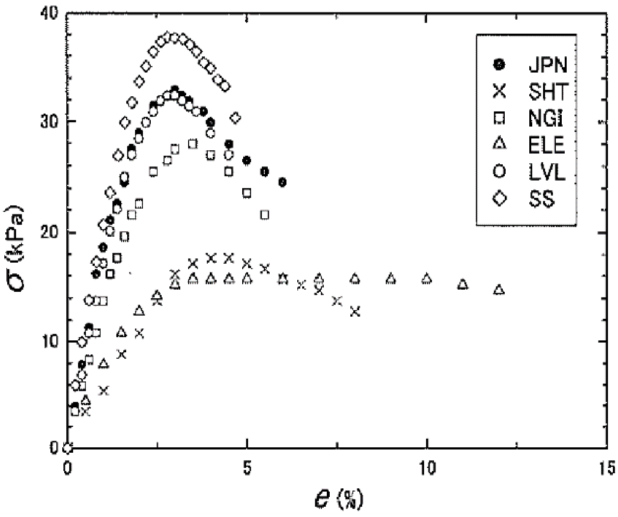


Figure 2-59 Stress ~ strain curve of unconfined

Table 2-23 Specifications of each sample used for the research

Sampler type	Inside diameter (mm)	Sampler length (mm)	Sample length (mm)	Thickness (mm)	Area ratio2) (%)	Inside clearance ratio3) (%)	Piston	Developer
JPN	75	1000	800	1.5	7.5	0	yes	Japanese Geo-technical Society
SHT	72	610	500	1.65	8.6	1	no	ASTMD1587
NGI	54	768	650	13	42	0.5	yes	Norwagean Geo-technical Institute
ELE	101	500	330	1.7	6.4	0	yes	Engineering Lab. Equipment, UK
LVL	208	660	600	4	7.3	0	no	Lavel University, Canada
SS	2501)	–	350	–	–	–	no	Sherbrooke University, Canada

1) : sample diameter
2) : $(D_o^2 - D_e^2)D_e^2 \times 100$ where, D_o : outer diameter of sampler.De : tip diameter of sampler
3) : $(D_i - D_e)/D_e \times 100$ where, D_i : inner diameter of sampler

vii) Since the quality of the sample in the sampler depends on the position from the cutting edge, in the mechanical property tests such as the unconfined compression test etc., it is better to use the specimen close to the cutting edge as much as possible as shown in Figure 2-60 and Figure 2-61. Generally, the direction of extracting a sample from a sampling tube is opposite to that in sampling. For this reason, the distribution of the shear strength in the sampler varies, specimens close to the cutting edge should be used for the mechanical tests. In addition, it was found that it is necessary to conduct the laboratory tests as soon as possible after sampling.

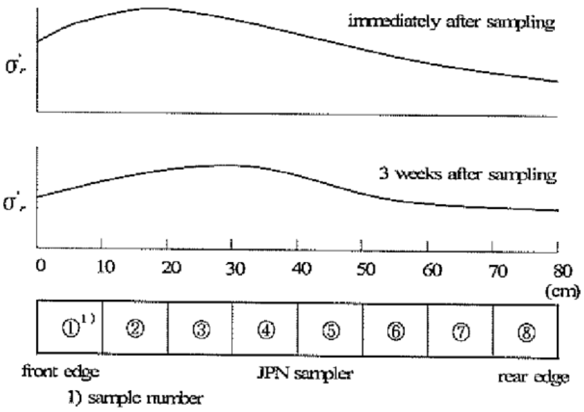


Figure 2-60 Conceptual figure for effective residual stress $\sigma_r{'}$

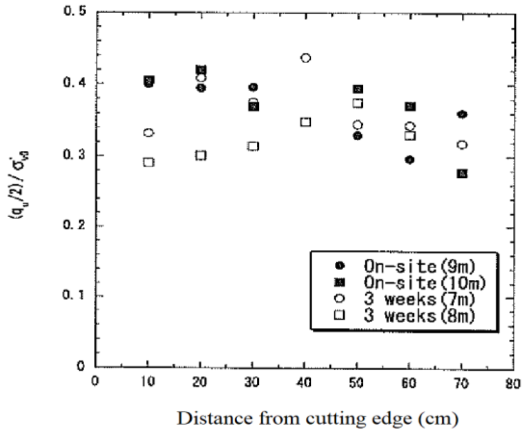


Figure 2-61 Relation between $q_u/2$ and distance from cutting edge of sample tube

Tsuchida and Kobayashi et al., 1988 [30] analyzed the soil investigation cases conducted by the different investigators on the same ground to clarify the factors of variation of unconfined compressive strength. In order to investigate the influence of cracks which were not taken into consideration before as a factor of variation, a series of laboratory tests were conducted. As a result of these analysis, the following conclusions are drawn.

i) As a result of analyzing the soil investigation cases conducted by different investigators on the almost uniform seabed ground in Tokyo Bay and Osaka Bay, a difference was recognized in the unconfined compressive strength q_u which is thought to be caused by the disturbance at the time of sampling. This indicates that q_u may underestimate the strength of the ground in some cases depending on the quality of sampling as shown in Figure 2-62.
ii) According to the conventional concept of disturbance of samples, the disturbance at the time of sample collection was said to appear remarkably in the coefficient of deformation E_{50}. Focusing on the relation between E_{50} and q_u in this case, although the conventional idea is applicable to the case in Tokyo Bay, it was not so much applicable to the case in Osaka Bay due to different type of stress ~ strain curves as shown in Figure 2-63.
iii) From the unconfined compression test results of artificially cracked clay sample, when cracks are present in the specimen, the E_{50} does not decrease as much as the specimen remolded or sheared, its decrease ratio is almost same value with one of q_u. Even in the

failure strain ε_f at the time of failure, in the case of cracks, it showed a clearly different tendency from the case of disturbance by remolding or shearing.

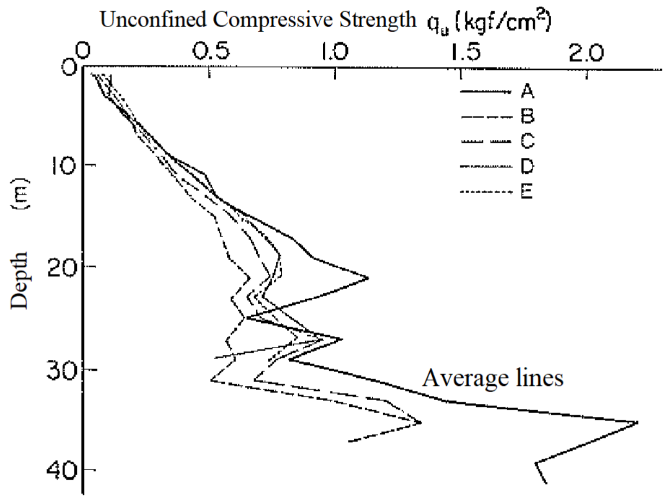


Figure 2-62 Comparison of average q_u with depth obtained by different companies at the same site

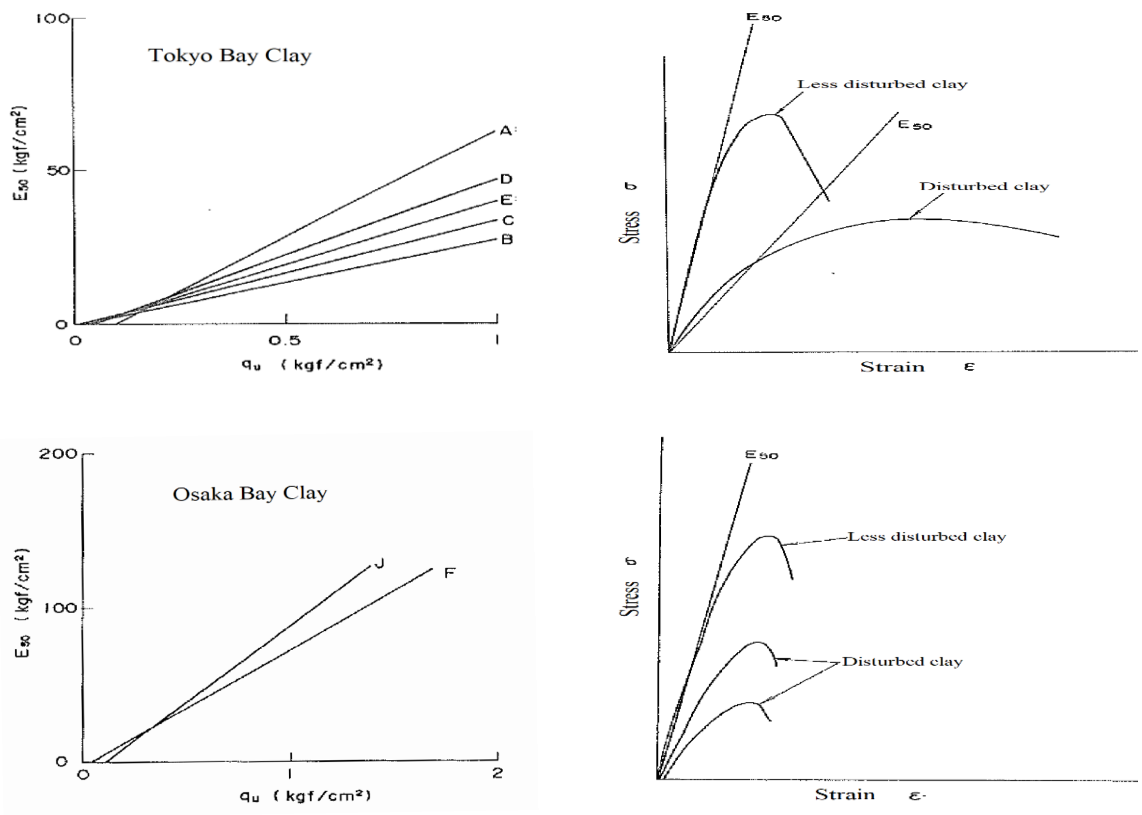


Figure 2-63 Different stress ~ strain curves between Tokyo Bay and Osaka Bay clays in unconfined compression test

iv) The disturbance of clay sample can be divided into "remold type" and "crack type". In both cases, the relationship between the decrease in strength due to disturbance and the decrease in coefficient of deformation is distinctly different. This can be confirmed by considering the results of the X-ray transmission test of the sampler reported by Ishii et al., 1987 [47)].

v) If there is a crack inside the specimen, it is possible to obtain strength close to the original one by triaxial UU test. In addition, it is possible to obtain the strength and stress ~ strain relation without the influence of the crack by triaxial Ck_0U test as shown in Figure 2-64.

vi) For saturated clay, it is considered that the design using the strength of triaxial UU test instead of q_u may be carried out in the current design method.

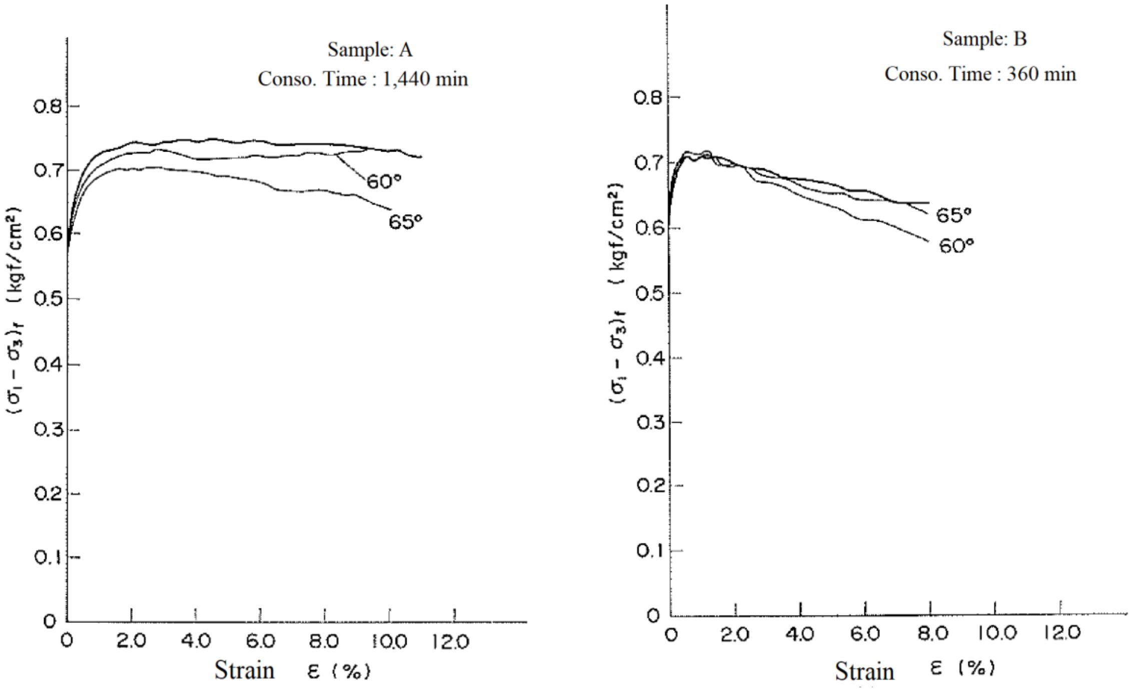


Figure 2-64 Comparison between stress ~ strain curves of samples with and without cracks by Triaxial Test (CK_0U)

Tsuchida, Noguchi and Watabe, 2017 [33)] reported that, in order to evaluate the strength of the marine clay layer at the site of Tokyo International Airport D-runway Construction Project, in addition to unconfined compressive strength, the strength evaluation by recompression method using triaxial compression test, unconfined compressive strength evaluated using combined simple CU strength (combined method) and strength evaluation by triaxial UU test were conducted. As a result, the following conclusions are drawn.

i) In the case of low plasticity clay having a liquid limit less than 40%, the unconfined compressive strength remarkably decreased due to disturbance regardless of sampling depths, but the quality of samples was judged to be good in case of 40% or more of the liquid limit.
ii) In addition, the shear strength obtained from triaxial UU test on the specimen deeper than 20 m was judged to be somewhat better quality, it was considered that it is due to improvement of strength decrease by the disturbance of the crack type found in the unconfined compressive strength as shown in Figure 2-65.

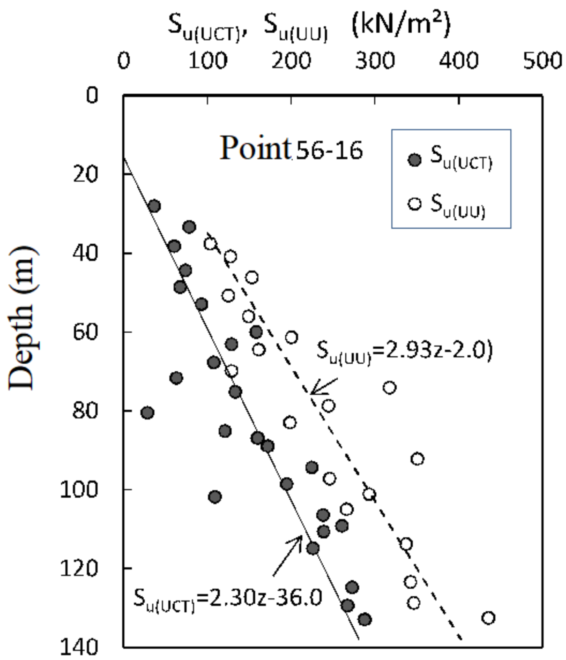


Figure 2-65 Relation between maximum deviation stress of UU test and depth (Diluvial clay at off Senshu, Osaka Bay)

In addition, Shogaki and Kaneko, 1994 [31)] studied the influence of sample disturbance on shear strength and consolidation parameters using soil disturbing equipment as shown in Figure 2-67.

Statistical Properties such as mean value and coefficient of variation of undrained shear strength and consolidation parameters of disturbed soils were investigated through laboratory testing. A series of unconfined compression tests and standard consolidation tests were performed on natural deposit of Kuwana clay. Soil profile of the ground and index properties of samples are shown in Figure 2-66 and Table 2-24, respectively. Stress ~ strain curves of unconfined compression test and relationship between q_u-ratio and cross sectional area ratio R_a, rerationship between E and σ/q_u are shown in Figure 2-68, Figure 2-69 and Figure 2-70 respectively.

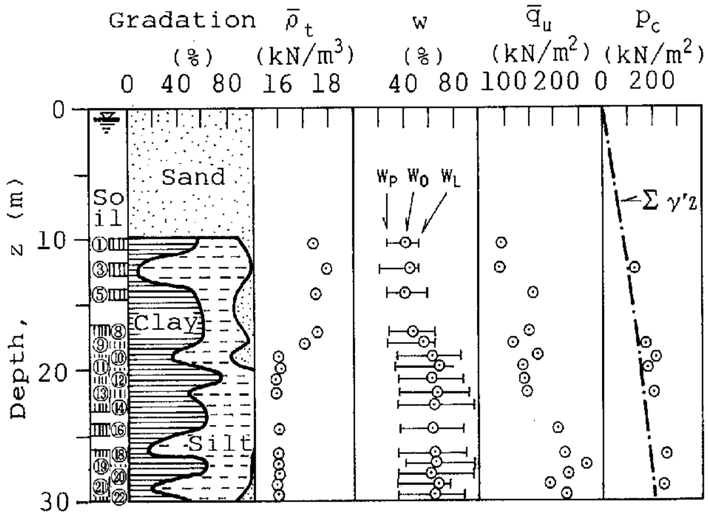
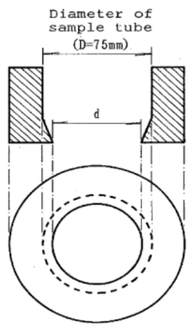


Figure 2-66 Soil Profile of Kuwana Coastal Area

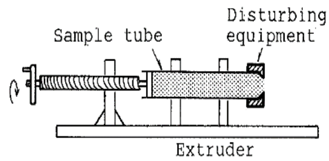
Table 2-24 Index properties of samples of Kuwana Clay

Soil	T-3	T-9	T-10	T-11	T-13	T-18	T-21
Depth (-m)	12	18	19	20	22	26	29
Sand (%)	1.2	5.8	18.5	1.9	0.5	2.4	3.8
Silt (%)	93.6	38.0	47.7	46.4	53.0	81.5	79.0
Clay (%)	5.2	56.2	33.8	51.7	46.5	16.1	17.2
w_L (%)	50.6	64.2	87.1	79.5	95.2	91.5	78.3
I_p (%)	25.8	36.0	50.8	44.1	57.2	53.6	42.9



Sample	d(cm)	A_E (cm²)	R_a
A	7.50	44.18	1.000
B	7.12	39.86	0.902
C	6.72	35.47	0.803
D	6.33	31.44	0.712
E	Remolded soil		

a) Disturbing equipment



b) Procedure for disturbing

Figure 2-67 Equipment and procedure for disturbing soil samples

As a result, the following conclusions are drawn.

i) The values of consolidation yield stress, p_c, and compression index, C_c, become larger than the actual values for a range of disturbed samples to the undisturbed sample, q_u-ratio, being greater than 0.8, and vice versa, for a range of q_u-ratio being smaller than 0.8 as shown in Figure 2-71.

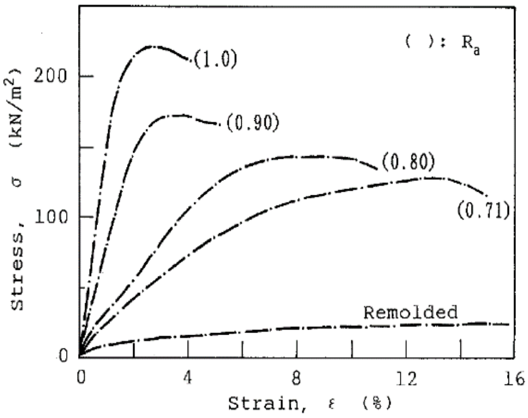


Figure 2-68 Stress ~ Strain Curves (T-18)

ii) The effect of sample disturbance on coefficient of consolidation, c_v, coefficient of volume compressibility m_v and coefficient of permeability k are different for the range of consolidation pressure p, up to p_c. Therefore, the q_u-ratio is affected only by these consolidation parameters, and is independent of I_p, p, and q_u values as shown in Figure 2-71 and Figure 2-72.

iii) For the normally consolidated region, c_v, m_v and k values of remolded soil are about 70%, 40%, and 80 % smaller than those of the undisturbed soil, respectively as shown in Figure 2-73 and Figure 2-74.

iv) The relationship between consolidation parameters and sample disturbance is given as a function of the q_u-ratio.

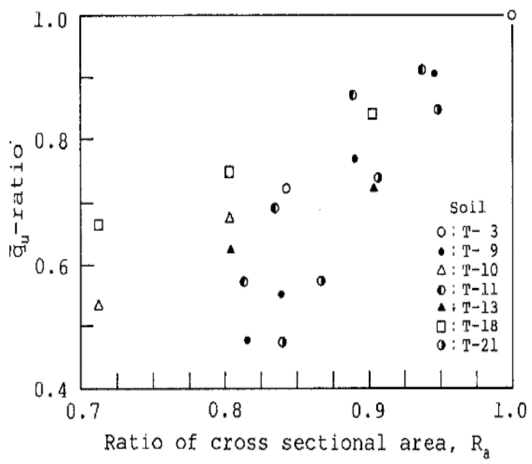


Figure 2-69 Relationship between q_u-ratio and cross sectional area ratio R_a (T-18)

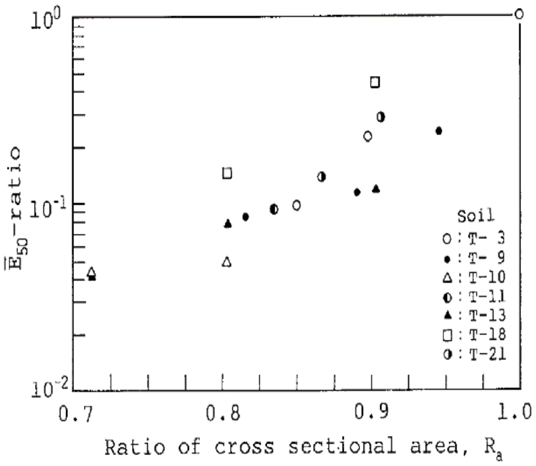


Figure 2-70 Relationship between E and σ/q_u (T-18)

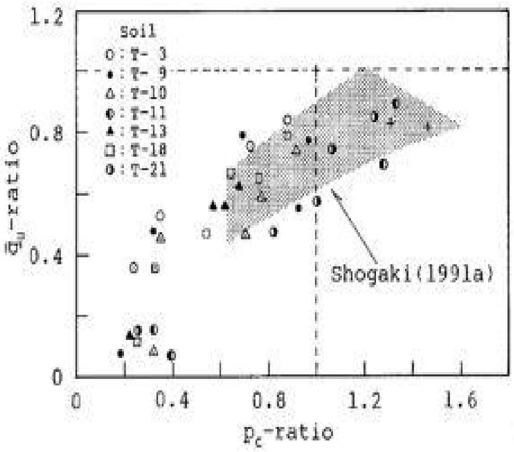


Figure 2-71 Relationship between q_u-ratio and p_c-ratio

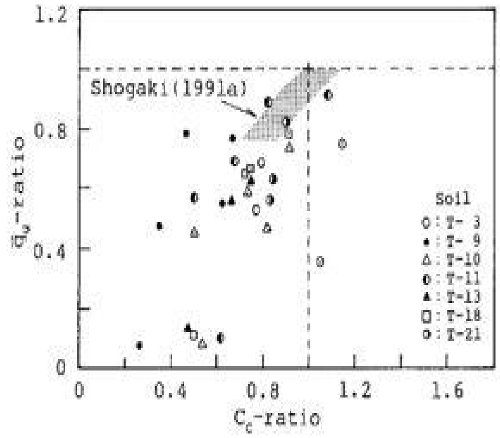
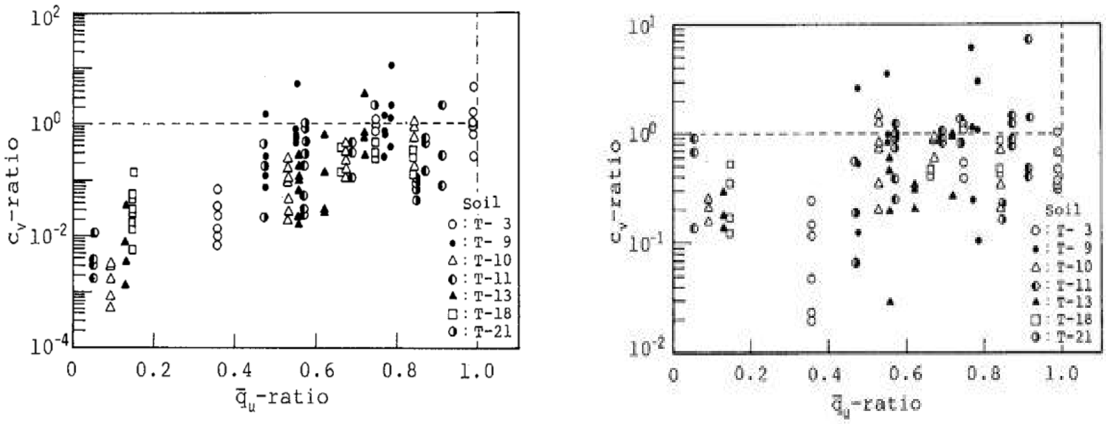


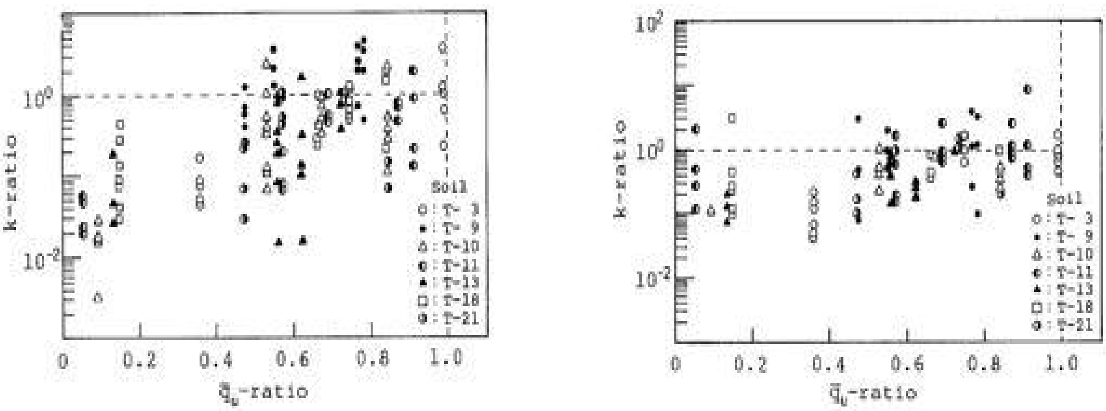
Figure 2-72 Relationship between q_u-ratio and C_c-ratio



a) Overconsolidated region

b) Normally consolidated region

Figure 2-73 Relationship between c_v-ratio and q_u-ratio



a) Overconsolidated region

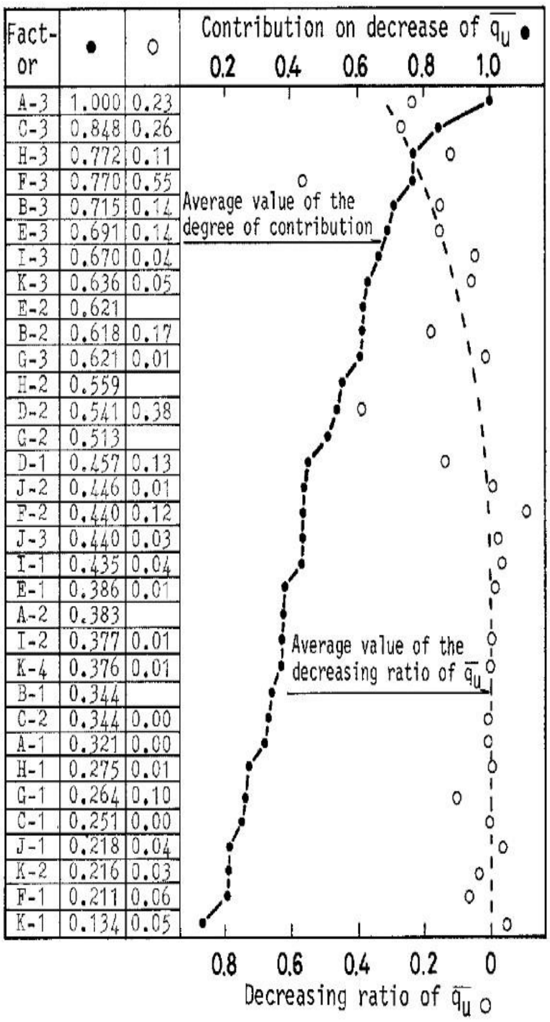
b) Normally consolidated region

Figure 2-74 Relationship between k-ratio and q_u-ratio

According to Matsuo and Shogaki, 1986 [48)] as a factor affecting q_u-value, it is sorted out as shown in the following Table 2-25 and Figure 2-75.

Table 2-25 Notations of factors influencing q_u value give in questionnaires (after Matsuo and Shogaki, 1986)

A	1	Excessive compression of soils due to penetration of sampler without drilling hole
	2	Swelling of soils due to insufficient pressure of drilling mud
	3	Rapid rise of pump pressure during drilling
B	1	Unexpected movement of soil sampling along the wall of sampling tube
	2	Deformation of soil samples due to insufficient rigidity of sampling tube
	3	Excessive compression of soil samples to 10% more than normal due to choked up drain hole
C	1	Compression of soils at the bottom of borehole due to detached drill rod
	2	Drilling mud between stationary piston and soil samples
	3	Deformation of cutting edge
D	1	Excessive compression of soil samples to 5% more than usual due to insufficient locking of stationary piston
	2	Sampling with the use of free piston sampler
E	1	Irregular penetration of sampling tube
	2	Sample disturbance due to the existence of randomly distributed shells
	3	Excess penetration of sampler 10% more than normal
F	1	Rotation of sampler before withdrawal
	2	Drilling of casing before withdrawal of sampler
	3	Dropping of the sampler with soil samples to the bottom of borehole
G	1	Deformation of sampling tube during with drawal of piston
	2	Suction at the top of soil sample due to with drawal of piston
	3	Vibration and shocks due to machinery
H	1	Soil testing after three days with insufficient sealing of sample
	2	Movement of soil samples during sealing of the soil sampler
	3	Fall of soil sampler during the sealing operation
I	1	Deformation of sampling tube due to temperature variations
	2	Deformation of sampling tube caused by high temperature
	3	Vibration during transportation
J	1	Friction between soil and sampler during extrusion
	2	Horizontal extrusion of soil samples
	3	Excess fastening of sampling tube during extrusion
K	1	Soil testing seven to ten days after sampling
	2	Two specimens of 35mm diameter from a sample of 75 mm diameter
	3	Trimming of soil samples by unskillful worker
	4	5%/min strain rate during compression



Fact-or	●	○
A-3	1.000	0.23
C-3	0.848	0.26
H-3	0.772	0.11
F-3	0.770	0.55
B-3	0.715	0.14
E-3	0.691	0.14
I-3	0.670	0.04
K-3	0.636	0.05
E-2	0.621	
B-2	0.618	0.17
G-3	0.621	0.01
H-2	0.559	
D-2	0.541	0.38
G-2	0.513	
D-1	0.457	0.13
J-2	0.446	0.01
F-2	0.440	0.12
J-3	0.440	0.03
I-1	0.435	0.04
E-1	0.386	0.01
A-2	0.383	
I-2	0.377	0.01
K-4	0.376	0.01
B-1	0.344	
C-2	0.344	0.00
A-1	0.321	0.00
H-1	0.275	0.01
G-1	0.264	0.10
C-1	0.251	0.00
J-1	0.218	0.04
K-2	0.216	0.03
F-1	0.211	0.06
K-1	0.134	0.05

Figure 2-75 Contribution on decrease and decreasing ratio of q_u for factors (after Matsuo and Shogaki, 1988)

Tanaka and Sharma et al., 1996 [29)] conducted comparative studies on the effect of disturbance caused by sampling of undisturbed samples by different samplers.

Sample quality was studied at two different sites in Japan where thick marine Holocene clay is deposited. Six different types of samplers used were ; the Japanese standard thin wall sampler with and without piston, the Laval sampler, the Shelby tube, NGI 54mm and ELE 100mm piston (ELE100) samplers. Sample quality was evaluated by the unconfined compression test, which is a standard test for evaluating undrained shear strength in Japan.

The sample quality obtained by the Japanese piston sampler is the same as that obtained by the Laval sampler. The Shelby tube and the ELE 100 sampler, however, both provided poor quality samples.

Three types of laboratory tests were performed to evaluate the influence of sample disturbance on undrained shear strength: laboratory vane shear tests and recompression techniques using triaxial and direct shear testing equipment. It was observed that these methods are quite useful to evaluated shear strength even for poor quality samples.

The location of test sites and properties of clay deposits at Ariake site and Kinkai sites are shown in Figure 2-76 and Figure 2-77 respectively.

The conclusions of the above research are as follows.

i) Sample quality was evaluated by the unconfined compressive test which is used extensively in Japan to measure the undrained shear strength for cohesive soils. Previous investigation conducted at Bothkennar indicated that the Laval and the Sherbrooke samplers have better performance than conventional tube samplers for obtaining good quality samples. It was found, however, that sample quality obtained by the Japanese piston sampler is equivalent to that obtained by the Laval Sampler. Samples collected by the Shelby tube and the ELE100 sampler, however, showed lower strength and larger failure strain as shown in Figure 2-78 and Table 2-26.
ii) For geotechnical engineering practice, it is important to establish methods whereby the proper strength can be measured without the influence of sample quality. One of these methods is recompression technique in which the specimen is subjected to confining stress to compensate for the reduction of the in situ effective stress caused in the sampling process as shown in Figure 2-79. There has been a controversy that such a recompression technique overestimates the shear strength of a poor quality sample due to void ratio reduction after consolidation at certain confining pressure.
iii) Two recompression techniques were examined, i.e., Hanzawa's, 1992 [38] and Tsuchida's, 1991 [50] proposals. It is true that volumetric change for poor quality sample obtained by the Shelby tube and the ELE100 sampler is considerably greater than that indicated by the Laval and the Japanese samplers as shown in Figure 2-80. The shear strength measured by these methods in not, however, affected by the volumetric strain caused during the recompression. It can be concluded therefore that the recompression technique is quite useful to evaluate shear strength for poor quality sample.

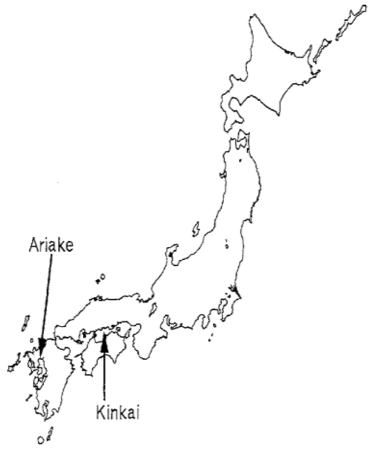


Figure 2-76 Location of test sites

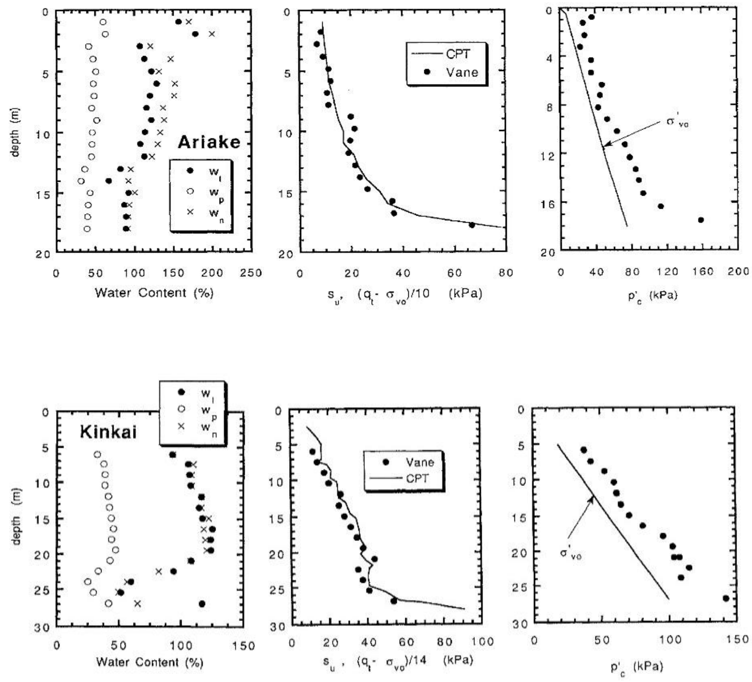


Figure 2-77 Properties of clay deposit at Ariake site and Kinkai site

Table 2-26 Sampler Dimensions

Name of sampler	Inside diameter (mm)	Sampler length (mm)	Thickness (mm)	Area ratio (%)	Piston
JPN(P)	75	1000	1.5	7.5	Yes
JPN(O)	75	1000	1.5	7.5	No
Laval	208	660	4.0	7.3	No
Shelby	72	610	1.65	8.6	No
NGI54	54	768	13.0	54.4	Yes
ELE100	101	500	1.7	6.4	Yes

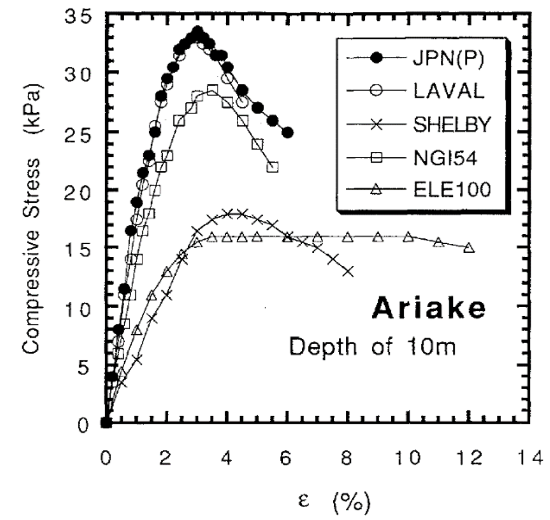


Figure 2-78 Stress and strain curves from unconfined compression test

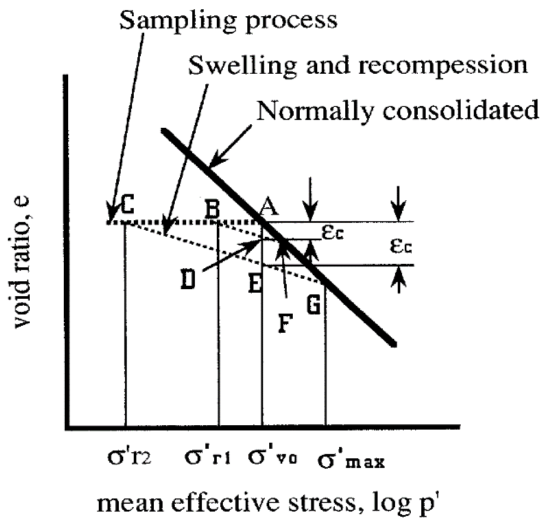


Figure 2-79 Reduction of effective stress due to sampling

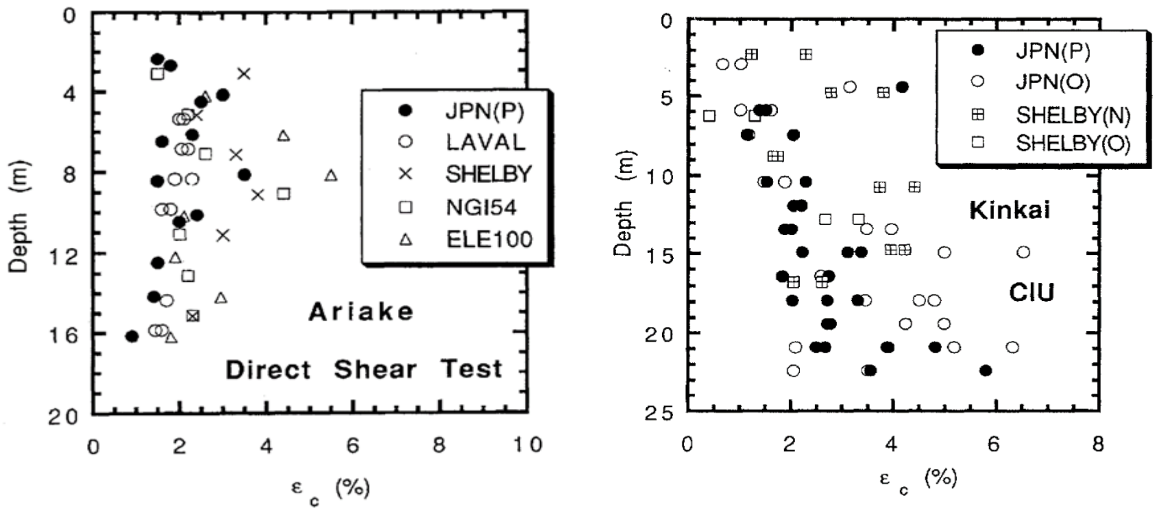


Figure 2-80 Volumetric change ε_c after consolidation for Ariake clay and Kinkai clay

Tsuchida, 2000 [32)] studied about the evaluation of shear strength considering the quality of soft clay samples.

In Japan, the undrained strength of soft clay deposit is determined by half of the average unconfined compressive strength q_u. One problem with the q_u method is that the validity of the method depends largely on the quality of the undisturbed soil sample, while there exists no accepted method to evaluate the sample quality.

The various methods for determining the undrained strength of soft clay were reviewed and compared, focusing on the effectiveness of the unconfined compression strength q_u method. The conclusions are summarized as follows:

i) The undrained strength of soft clay deposits is determined by half of the average unconfined compressive strength q_u in Japan. An important problem to be solved in the q_u method is that the validity of the method highly depends on the quality of the undisturbed soil sample, while there exists no accepted method to evaluate the sample quality.

ii) The strength from the q_u method, S_u (q_u), are compared with those from the two proposed methods, SHANSEP and Recompression. For most undisturbed samples taken with a fixed piston sampler, the strengths obtained by the q_u method were almost the same as S_u (M.B.), which was obtained by the Modified Bjerrum's method (recompression method). However, when the samples were disturbed for some reasons, S_u (q_u) became smaller than S_u (MB). The strength of the SHANSEP method were usually smaller than those of the q_u method as shown in Figure 2-81.

iii) The q_u method seems to be valid for most Japanese marine clays, if the undisturbed samples are collected with a fixed piston sampler of Japanese Industrial Standard. For samples taken with Shelby-tube, the q_u method will seriously underestimate the undrained strength as shown in Figure 2-82.

iv) A new practical method named the "Advanced q_u method" is proposed. In this method, 3 unconfined compression tests and a Simple CU test are performed. By comparing the average of q_u/2 with the strength of Simple CU test, the quality of the sample and the strength to be used for the design are determined. The effectiveness of the new method was ascertained in field tests as shown in Figure 2-83.

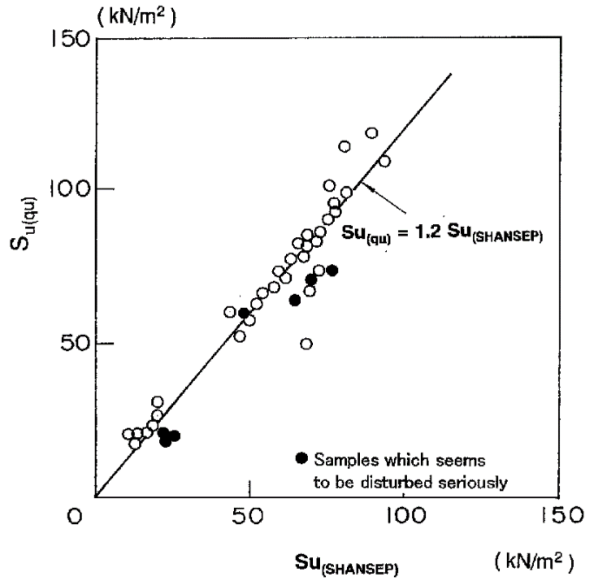


Figure 2-81 Comparison between $S_{u(qu)}$ and $S_{u(SHANSEP)}$

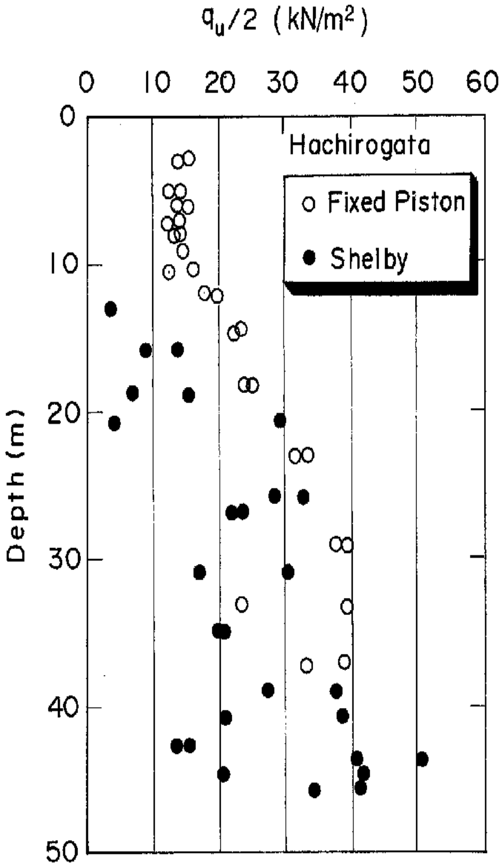


Figure 2-82 Values of q_u taken by Fixed Piston sampler and Shelby-tube sampler

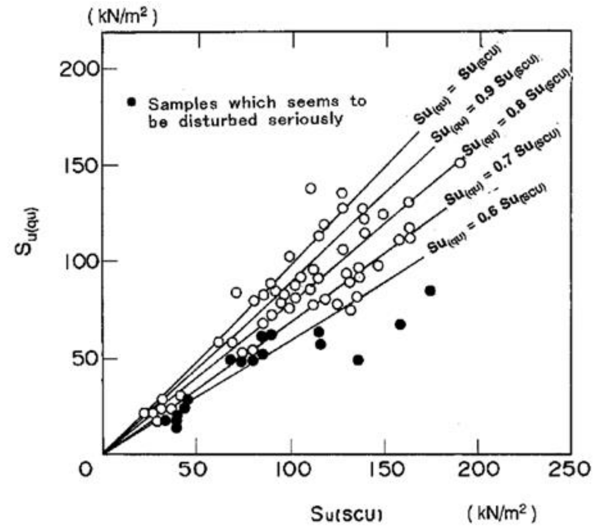


Figure 2-83 Comparison between $S_{u(qu)}$ and $S_{u(SCU)}$

Ladd, C.C. and DeGroot, D.J., 2004 described comprehensive opinions and recommendations about "Recommended Practice for Soft Ground Site Characterization" in Arthur Casagrande Lecture. In that paper, they mentioned the sources of disturbance and procedures to minimize the amount of disturbance. Potential sources of sample disturbance via hypothetical stress path during the process of obtaining a tube sample for laboratory testing are illustrated as shown in Figure 2-84. Then, they gave several recommendations to minimize the amount of disturbance from the drilling borehole to the transportation and storage of samples as follows.

To prevent excessive disturbance before sampling, it has to be sure that borehole remains filled with drilling mud having a weight.

Use minimum outside tube diameter D_0=76mm, tube well thickness such that $D_0/t >$ 45with sharp cutting edge, and inside clearance ratio near zero (certainly less than 0.5%). Use new tubes made of brass, stainless steel or coated (galvanized or epoxy) steel to help minimize corrosion.

Tube sample should be obtained with a stationary (fixed) piston samplers both to control the amount of soil entering the tube and to better retain the soil upon extraction. Piston samplers usually yield far better recovery and sample quality than push sampler. After advancing the tube, allow time for the clay to partially bond to the tube, then slowly rotate the tube two revolutions to shear the soil, and finally slowly withdraw the sample.

Leave the in the tubes and pack for shipping (if necessary) following the guidelines set forth in ASTM D4220. The cost of tubes is far less than money wasted by running expensive consolidation and strength tests on disturbed soil.

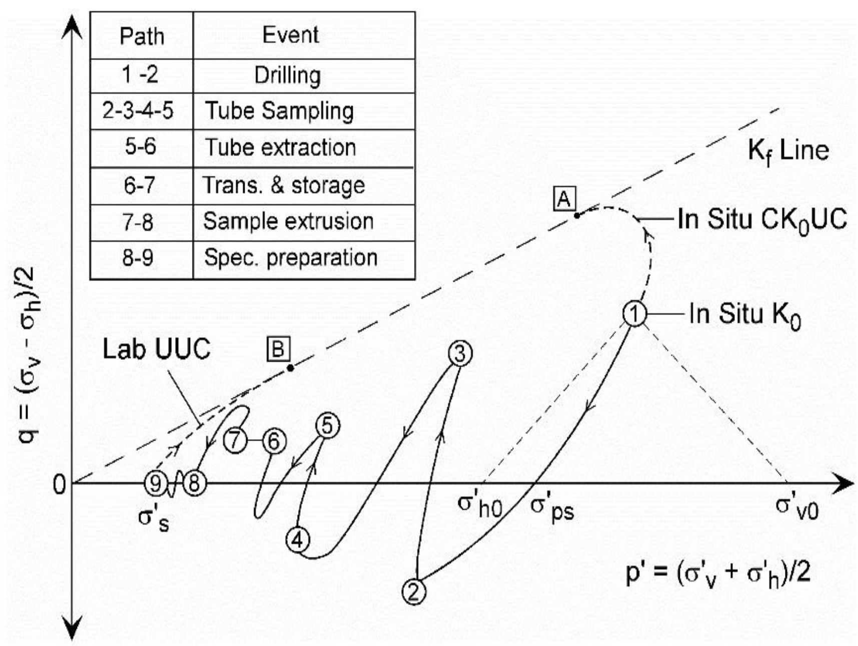


Figure 2-84 Hypothetical Stress Path during Tube Sampling and Specimen Preparation of Centerline Element of Low OCR Clay (after Ladd and Lambe 1963 [51)], Baligh et al. 1987 [52)])

2.7 Conclusion of this Chapter

In this chapter, I have classified and compiled past researches and studies on physical properties and mechanical properties of clays. Past researches and studies related to this study were classified into the following five categories.

(1) Correlations among the Physical and the Mechanical Properties
(2) Shear Strength and Shear Strength Ratio of Cohesive Soils
(3) Coefficient of Secondary Consolidation
(4) Relations between Clay Minerals and Physical Properties
(5) Effects of Sample Disturbance due to Different Sampling Methods

Physical properties and mechanical properties of cohesive soils and their relationships are greatly affected by the soil's regional character, the difference in the environment and processes in which the clay is formed, the particle size distribution, the liquid limit of clay, and the content of clay minerals etc., it varies from region to region. Therefore, the correlation equations of physical properties and mechanical properties related to cohesive soils, which have been proposed in the past studies and researches, are not always applicable to soils depending on the locality of soils, and it was found that it was necessary to obtain the relationships among the physical properties and the mechanical properties for each area/ region.

In this study, Yangon area in Myanmar was focused, and clarified the physical properties and mechanical properties and their relationships of cohesive soils in Yangon.

Reference

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3. Effects of sample disturbance due to different sampling methods with regard to the quality of undisturbed samples of cohesive soils in Yangon

3.1 Outline of Study

Soil has varieties of properties and its localities as well, therefore it cannot always be standardized the geotechnical testing and soil investigation methods. Sampling method is also not unified to one method. Because influence onto sample quality is quite different depending on soil localities.

There are several methods for taking undisturbed soil samples. All the undisturbed samples used for this research were taken by fixed piston sampler (hydraulic pressure type, refer to Figure 3-1). However, in Myanmar, Shelby tube sampler which is an open drive sampler is often used for undisturbed soil sampling because Shelby tube sampler (Refer to Figure 3-2) is recognized as one of the authorized undisturbed sampling methods in ASTM [1)].

In other researches by Tanaka, 2003 [2)], Tanaka et al., 1996 [3)], and Tsuchida, 2000 [4)], it is reported that there are obvious differences between sample qualities taken by fixed piston sampler and Shelby tube sampler and, as for the unconfined compressive strength, strength of samples taken by Shelby tube sampler shows about half of the strength of samples taken by fixed piston sampler. Thus, influence onto the quality of undisturbed samples due to the sampling methods is very important and should be paid attention to it.

Therefore, to grasp the extent of the changes in the sample quality due to the usage of different type of samplers for clays in Yangon, samples were actually taken at three locations with a fixed piston sampler and a Shelby tube sampler, each sampler was used to take samples following the same sampling procedure, and laboratory tests such as physical property tests, unconfined compression tests and standard consolidation tests were conducted. These samples were taken from soft clay layers having N-value of 4 or less.

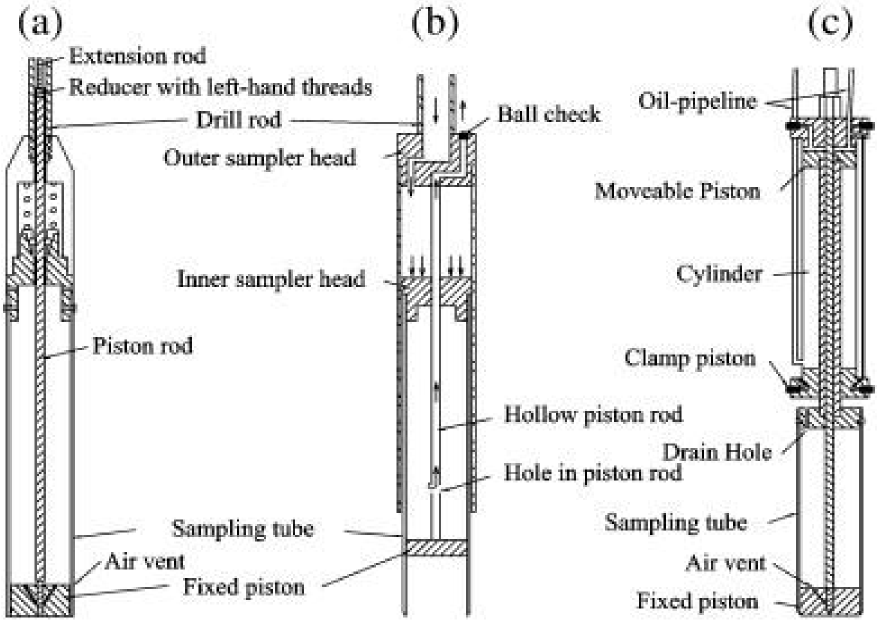


Figure 3-1 Fixed Piston Sampler (Hydraulic Pressure Type) [1), 5)]

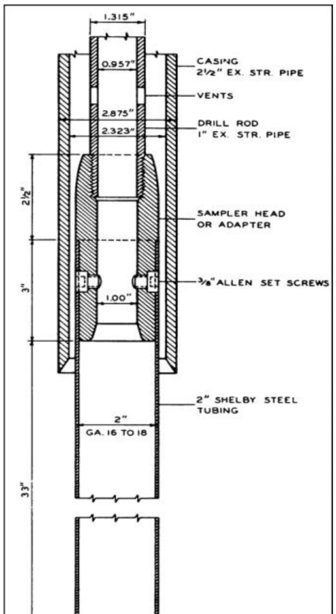


Figure 3-2 Shelby Tube Sampler [1)]

3.2 Samples to be compared

As shown in Figure 3-3, samples were taken at three locations, A, B and C in Yangon and surrounding areas. For the comparison purpose, Thi Vai clay along Mekong River which has different natural moisture content from Yangon clay was also compared each other. The clay samples taken for this comparison purpose were as follows.

(A) West suburb of Yangon (Twantay Township)

(B) Eastern Yangon (North Dagon Township)

(C) Southern suburb of Yangon (Thilawa in Thanlyin Township)

(D) Thi Vai clay (Hochiminh, Vietnam) [6)]

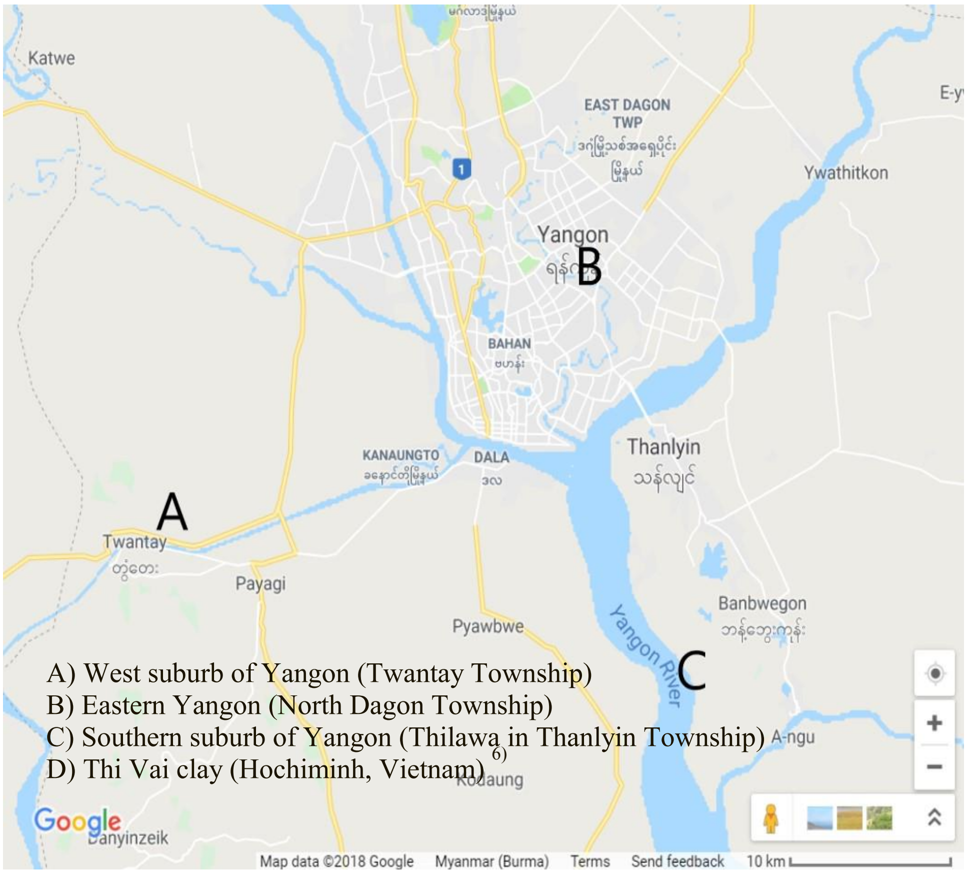


Figure 3-3 Locations of Samples for Comparison of Sample Quality between Fixed Piston Sampler and Shelby Tube Sampler (Source: retouched to Google Map)

3.3 Physical properties of each clay sample

Physical properties of each clay sample are as shown in Figure 3-4. As for the samples in Yangon and its suburbs, there are no significant differences in wet unit weight ranging from 18 to 20 kN/m^3, natural moisture content ranging from 20 to 40 %, liquid limit ranging from 30 to 60 % and plastic limit ranging from 20 to 30 %. On the other hand, Thi Vai clay has its wet unit weight ranging from 13 to 16 kN/m^3 and natural moisture content ranging from 60 to 110%, which is much different from the values of clays in Yangon and its suburb area. It can be said that Thi Vai clay is highly structured and has high compressibility compared to Yangon and its suburb clays.

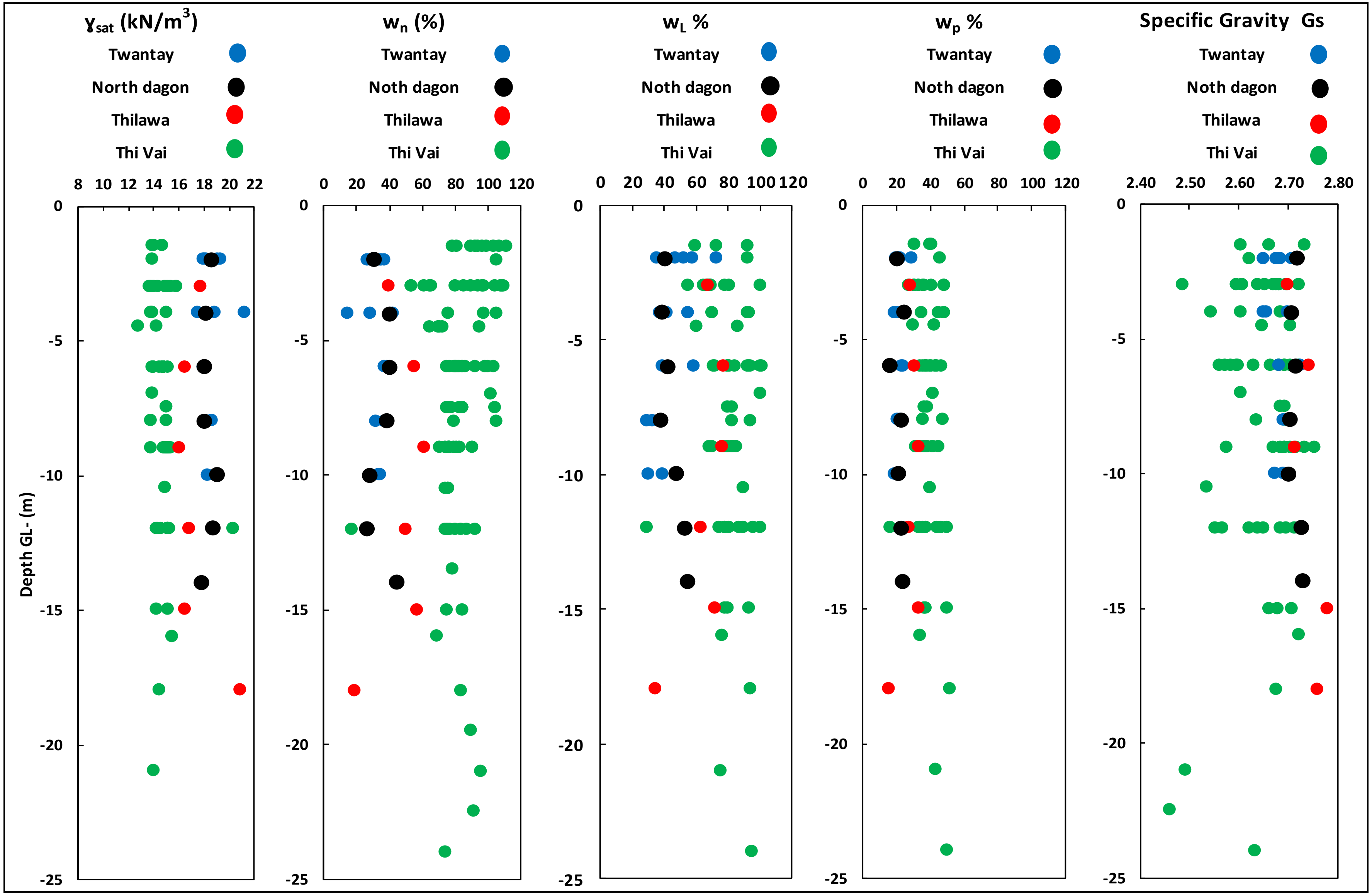


Figure 3-4 Physical Properties with depth of each clay samples (Twantay, North Dagon, Thilawa and Thi Vai clay)

3.4 Influence onto Sample Qualities between Fixed Piston Sampler and Shelby Tube Sampler

There are several methods for taking undisturbed soil samples. All the undisturbed samples used for this study were taken with a fixed piston sampler (hydraulic pressure type). However, in Myanmar, a Shelby tube sampler, which is an open drive sampler often used for undisturbed soil sampling, was used because the Shelby tube sampler is recognized as one of the authorized undisturbed sampling methods by the ASTM (ASTM, 2019) [1)].

The quality of soil samples is an important factor for the statistical study of soil data. A lot of studies have been presented on the effects of sample disturbance in soft clays (Ladd et al., 1963 [7)], Tanaka, 2003 [2)], Tanaka et al., 1996 [3)], Tsuchida, 2000 [4)]). According to research results by Tanaka and Tanaka et al.,1992 [9)], there are obvious differences between the quality of samples taken by the fixed piston sampler and Shelby tube sampler, and for unconfined compressive strength, the strength of samples taken by Shelby tube sampler show about half the strength of samples taken by fixed piston sampler. It was also reported that the sample disturbances affected the e-log p relationship and the compression index C_c, obtained by oedometer test (Shogaki, et al., 1994 [8)]).

Figure 3-5 shows the unconfined compressive strength q_u, failure strain ε_f, compression index C_c and Consolidation yield stress p_c with Depth (GL-:m). As shown in Figure 3-5, among the samples in Yangon and its suburb areas, there was not much difference on unconfined compressive strength q_u and failure strain ε_f due to the difference on sampling method between fixed piston and Shelby tube samplers. Failure strains ε_f are distributed in the range of 2 to 10% in both samples.

On the other hand, as for Thi Vai clay having high structure state, a clear difference from Yangon and its suburb's clays was observed between the unconfined compressive strength and the failure strain due to the difference of the samplers, such as ε_f = 1 to 5 % in the fixed piston sampler, $\varepsilon_f = 4$ to 9%. It can be said that high quality samples with less disturbance taken by the fixed piston sampler.

This difference between Yangon clays and Thi Vai clay [6)] is thought to be due to the difference in sensitivity ratio of both clays. It can be considered that Thi Vai clay having high structure and high compressibility is very sensitive compared to Yangon and its suburb clays. Furthermore, if it is focused on the Thilawa clay comparing with Thi Vai clay, difference of sample quality between fixed piston sampler and Shelby tube sampler come to be significantly clearer as follows.

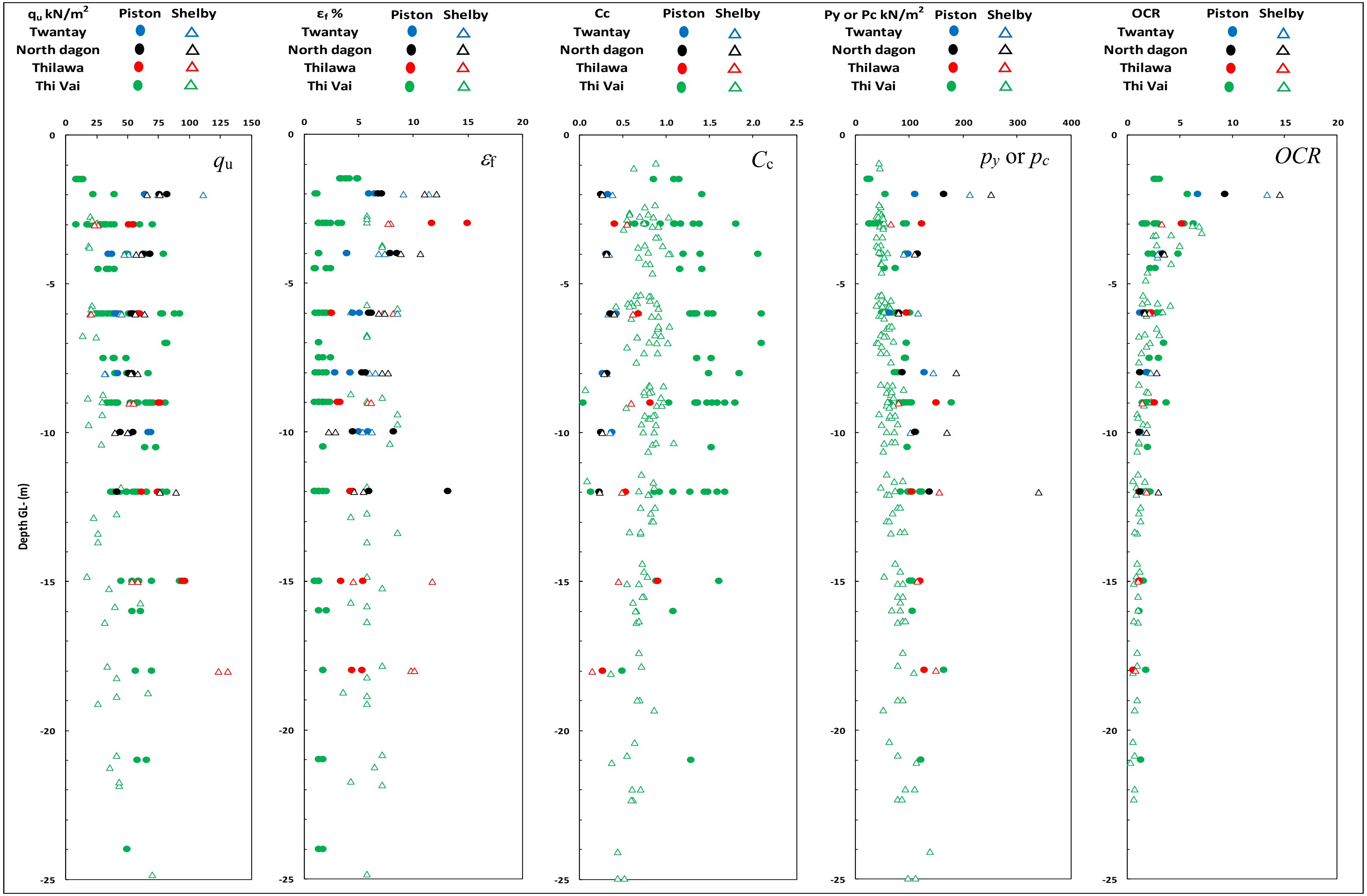


Figure 3-5 Result of Mechanical Property Tests of each clay samples (Twantay, North Dagon, Thilawa and Thi Vai clay)

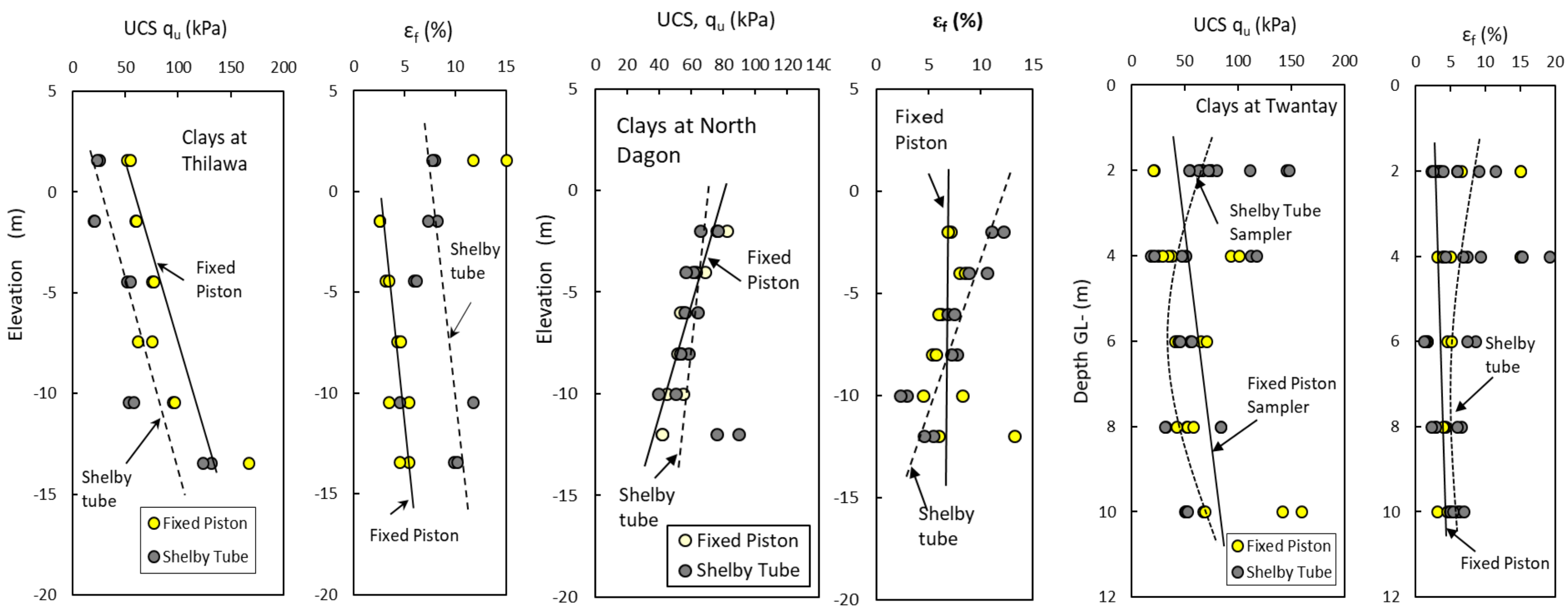
3.4.1 Unconfined Compressive Strength q_u and Failure Strain ε_f

To clarify the effect on sample quality due to different sampler types, comparison of unconfined compressive strengths between undisturbed samples taken with a fixed piston sampler and a Shelby tube sampler was carried out using clays from the Thilawa in Thanlyin Sub-area with 30 % to 60 % natural water content (w_n), and clays from North Dagon in the Eastern Sub-area with a w_n of 30% to 40%.

Figure 3-6 shows the comparison in unconfined compressive strength and the failure strain with depth between samples taken by the fixed piston sampler and Shelby tube sampler for clays in Thilawa, North Dagon and Twantay in Yangon and its suburbs, respectively. As shown in Figure 3-6, a clear difference in unconfined compressive strength was found between the samples taken with fixed piston and those with the Shelby tube sampler in the Thilawa. The Thilawa clay showed 50-100% greater strength at strains of less than 5%. However, in the clays from the North Dagon and Twantay, the difference in strength and the failure strain was not so much clear, as shown in Figure 3-6. Figure 3-7 shows the stress-strain curves in the unconfined compression tests of clays at those sites. Comparing the stress-strain curves of those clays, the clays from Thilawa show the peak strength at the axial strain of 3 to 5% and the significant reduction of strength after the failure strain was observed in most of samples, however, clays in North Dagon show an increase in strength with a strain of up to 5 to 8% and the reduction of strength after the failure strain was modest. Clays in Twantay show the peak strength at the axial strain of 3 to 7%. It therefore seems that in the clays in Myanmar the effectiveness of the Shelby tube sampler is dependent on the stress-strain properties of the clays.

The same comparisons were carried out with Thi Vai clay from Hochiminh, Vietnam (Yamada et al., 2013 [6)]), located along the Mekong River with a natural moisture content of 60% to 110%, and with Hachirogata clay from Japan carried out by Tanaka et al., 1992 [9)] as shown in Figure 3-8. The differences in q_u and ε_f obtained from these samples with the two types of sampler are more obvious than in the clays from Thilawa and North Dagon.

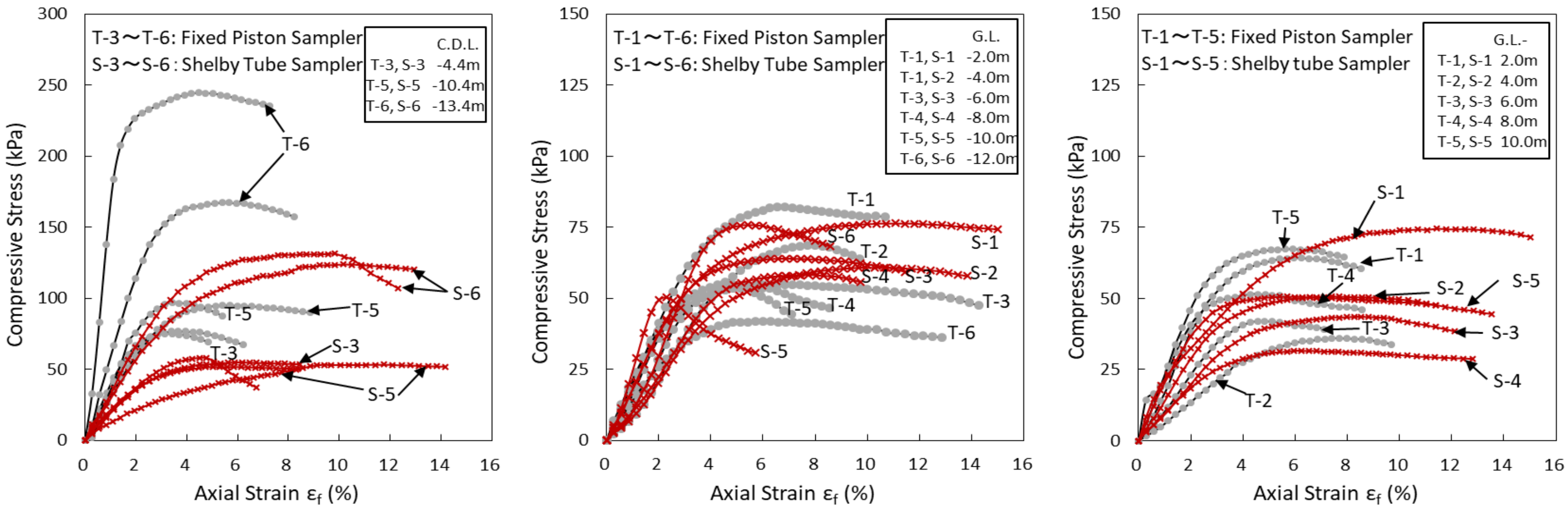
It is thought that Thi Vai clay from alongside the Mekong River in Hochiminh and the Hachirogata clay, with large liquid limits, are very sensitive and have high-grade structures. However, clays from Thilawa and North Dagon have relatively low sensitivity and lower-grade structure, so the influence on unconfined compressive strength by disturbances in the process of sampling and transportation is relatively small.



(a) Thilawa clay at C (b) North Dagon clay at B (c) Twantay clay at A

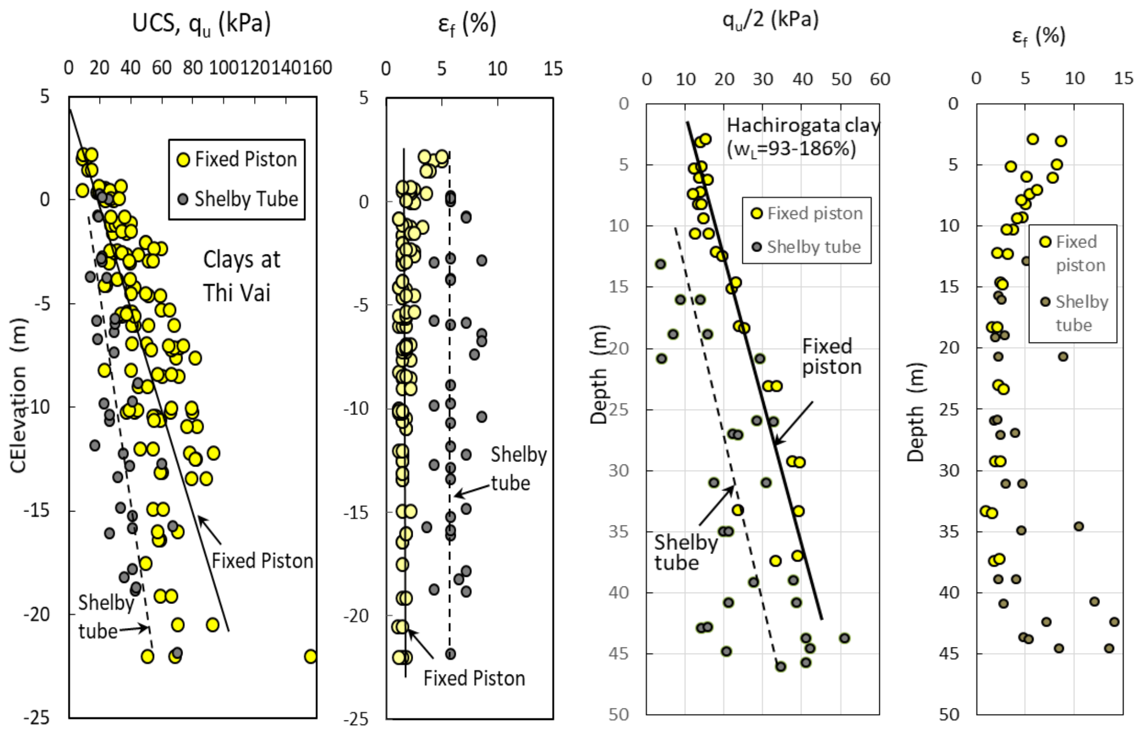
Figure 3-6 Effect to Unconfined Compressive Strength by Fixed Piston Sampler and Shelby Tube Sampler

(Thilawa, North Dagon and Twantay clay)



(a) Thilawa clay (b) North Dagon clay (c) Twantay clay

Figure 3-7 Effect to Stress-Strain Curves of Unconfined Compressive Strength by Fixed Piston Sampler and Shelby Tube Sampler (Thilawa, North Dagon and Twantay clay)



(a) Thi Vai clay (b) Hachirogata clay

Figure 3-8 Effect to Unconfined Compressive Strength by Fixed Piston Sampler and Shelby Tube Sampler (Thi Vai clay in Hochiminh and Hachirogata clay in Japan)

3.4.2 Compression Index C_c

As shown in Figure 3-5, among the samples in Yangon and its suburb areas, there was not much difference on compression index C_c due to the difference between fixed piston and Shelby tube samplers. Compression index C_c is distributed in the range of C_c = 0.2 to 0.5. On the other hand, as for Thi Vai clay which is high-grade structured, a distinct difference between two samplers is observed in the compression index C_c. It is distributed in the range of C_c = 0.5 to 1.0 for samples taken by Shelby tube sampler, whereas C_c is distributed in the range of 1.0 to 2.0 for samples taken by fixed piston sampler. It can be said that a good quality samples with less disturbance is taken by the fixed piston sampler. This difference between Yangon clays and Thi Vai clay is considered to be due to the difference in sensitivity ratio as well as in case of unconfined compression test results.

In addition, Figure 3-9 is a compression curve (ln v ~ ln p curve) shown by specific volume ln v (v = e/ (1+e_0)) and consolidation pressure ln p in order to check whether the structure of clays in Yangon and its suburb areas is highly structured or not. Only one specimen taken with a fixed piston sampler is the curve with a shoulder at the center. This shoulder appears when the structure is judged to be high.

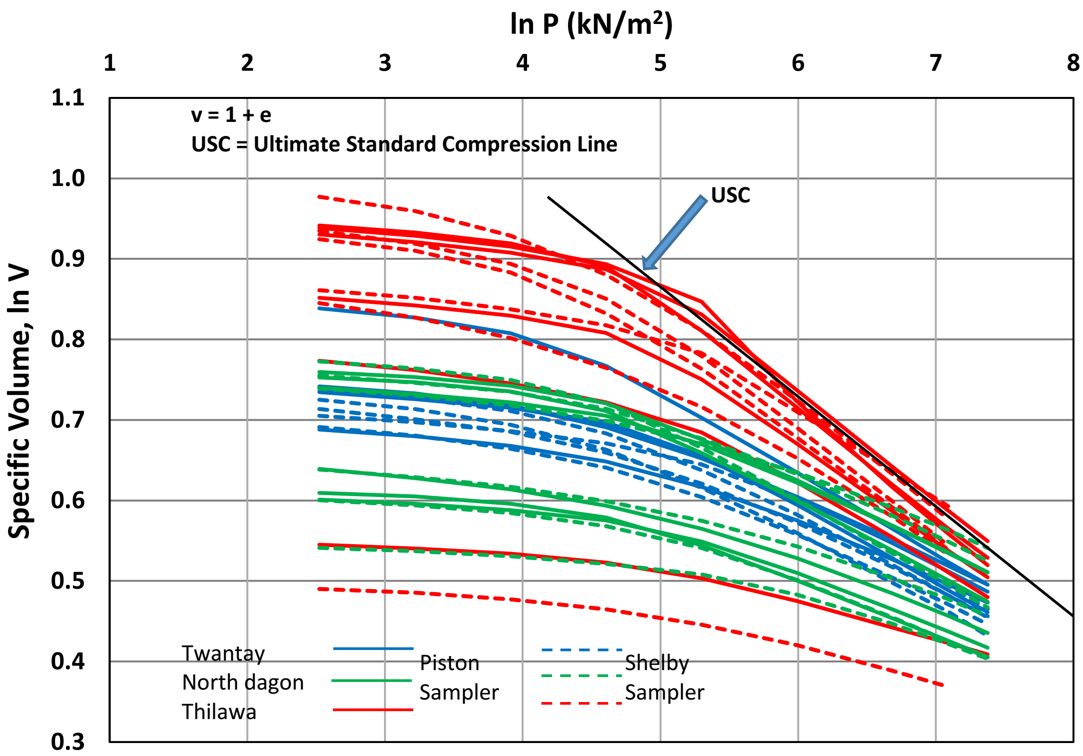
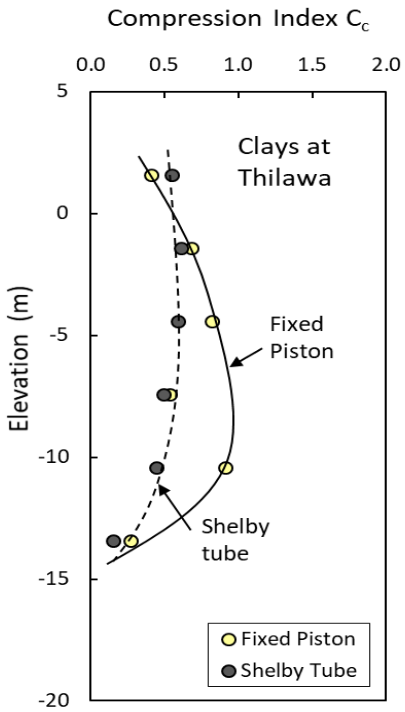


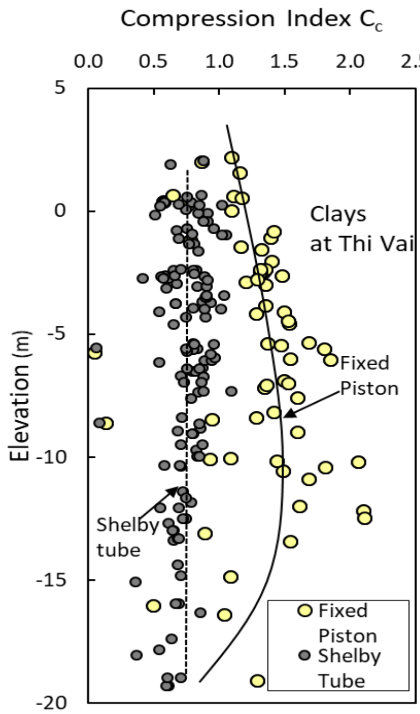
Figure 3-9 ln v ~ ln p curves of samples taken by Fixed Piston Sampler and Shelby Tube Sampler for clays in Yangon and its suburbs (Twantay, North Dagon and Thilawa)

Figure 3-10 shows the effect on the compression Index, C_c of consolidation test results for clays from Thilawa and Thi Vai due to the different sampling methods of fixed piston and Shelby tube samplers. As shown in this figure, it can clearly be seen that the effects from disturbance on the C_c values of samples taken with the Shelby tube sampler is greater than for those taken with a fixed piston sampler. It is generally known that C_c values become smaller because of disturbance to samples (Shogaki et al., 1994 [8)]). Clays in Thi Vai, Hochiminh display more differences in C_c between samples taken with Shelby tube and fixed piston samplers than clays in Thilawa. This tendency is quite similar to the disturbance effect on to unconfined compressive strength.

Therefore, for accurate evaluation of unconfined compressive strength q_u and compression index C_c of clayey soils, fixed piston sampler is better to use for undisturbed soil sampling. It is considered that one of the reasons for the difference of sample disturbance between the fixed piston sampler and Shelby Tube is due to the difference of sample deformations during the pulling-out process of sample tube from the ground as shown in Figure 3-11. In case of using the fixed piston sampler, shear deformation generated in the sample is smaller than the one of Shelby Tube sampler due to the suction effect between the piston and the sample. The whole undisturbed samples used for this research was taken with the fixed piston sampler according to JGS (Japanese Geotechnical Society) Standard, 2004 [5)].



(a) Thilawa clay



(b) Thi Vai clay

Figure 3-10 Effect on Compression Index C_c of Consolidation Test due to sampling methods by Fixed Piston Sampler and Shelby Tube Sampler (Clays at Thilawa and Thi Vai in Hochiminh)

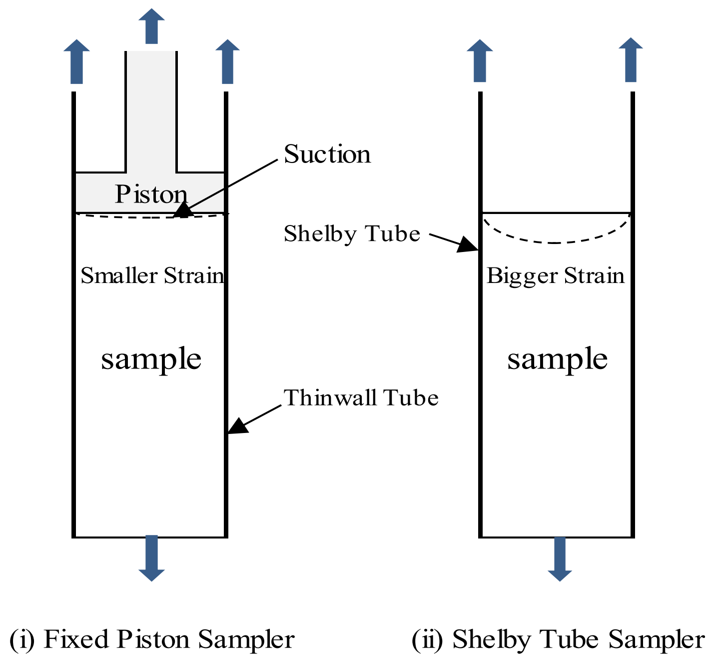


Figure 3-11 A Possible Cause of Sample Disturbance (Strains generated in Undisturbed Samples taken by Fixed Piston and Shelby Tube Samplers)

As mentioned above, there exist some differences in the unconfined compressive strength q_u and the compression index C_c of Thilawa clay due to the sampling methods. However, the differences were dependent on the stress-strain property of clay and were not so large as in the more sensitive Thi Vai and Hachirogata clays. For the accurate evaluation of geotechnical data, engineers should use the fixed piston sampler.

3.5 Conclusion of this Chapter

The conclusion in this chapter which examined the quality of the sample due to the different samplers is as follows.

1) The effects of disturbances by using different types of samplers between fixed piston and Shelby tube sampler are recognized in Yangon clay. However, the differences of degree of disturbance is not so much clear in some of Yangon clays except Thilawa clay.
2) As a result, as for the Thilawa clay, it was recognized clearly that the effect of disturbance was smaller incase of fixed piston sampler than Shelby tube sampler. Therefore, in this study, samples taken only with a fixed piston sampler were used.
3) The effect of the different sampling methods on the mechanical properties of Yangon clay were studied. As for the Thilawa clay in the Thanlyin Sub-area, the strength of samples taken by fixed piston sampler was definitely greater than that taken with the Shelby tube sampler. As for the North Dagon clay in the Eastern Sub-area and Twantay clay in Twantay Township, a difference in strength was not observed clearly between two sampling methods. The stress-strain curves for both clays also showed no significant differences between both samples.
4) There existed some differences in the unconfined compressive strength q_u and the compression index C_c of Yangon clay due to the different sampling methods. However, the differences were dependent on the stress-strain property of clay and were not so large as in the more sensitive Thi Vai clays in Vietnam or Hachirogata clays in Japan.
5) It is necessary to clarify whether to conclude that there is no noticeable difference between Thilawa clay and Yangon clay due to different samplers, the fixed piston and the Shelby tube sampler, with collecting more data.
6) In Thi Vai clay, a remarkable difference in the mechanical test results is recognized depending on the type of sampler. It is thought that this difference is due to the difference in the sensitivity of clay, but it is necessary to examine more what kind of clay properties makes this difference in sensitivity of clay.
7) There exist some differences in the unconfined compressive strength q_u and the compression index C_c of Thilawa clay between samples taken by two sampling methods. It can be recommended that, for the accurate evaluation of geotechnical data, the fixed piston sampler should be used for taking undisturbed samples of cohesive soils.

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4. Comparison and study of geotechnical engineering characteristics of cohesive soils in Yangon

4.1 Outline of Study

Yangon area is located at the border between the Irrawaddy Delta Sub-basin and the Pegu-Yoma Sittaung basin as shown in Figure 4-1 and Figure 4-2. It retrieves that there might be some differences in soil properties due to different conditions of sedimentation at different basins. The borderline between the Irrawaddy Delta Sub-basin and the Pegu-Yoma Sittaung basin can be assumed as shown in Figure 4-2. The western side of the borderline shows Irrawaddy Delta Sub-basin area and eastern side shows Pegu-Yoma Sittaung basin.

Accordingly, in this research, the Yangon area is divided into seven sub-areas, and soil properties for clays obtained from each sub-area are compared. Seven sub-areas are defined based on the Tertiary ridge running from north to south through the center of Yangon area as shown in Figure 4-2 and Figure 4-3.

Based on the above information, Yangon area is divided into 7 sub-areas for comparison purpose as follows.

Irrawaddy Delta sub-basin

1) North West, 2) Western, 3) Downtown,4) Thanlyin (Thilawa)

Pegu-Yoma Sittaung Basin

1) North East, 2) Eastern

Others

1) Central

The borderline between Irrawaddy Delta Sub-basin and Pegu-Yoma Sittaung basin is located along the central ridge of Tertiary Deposit in Yangon area. This ridge area surrounded by the circular railways in Yangon is called here as Central sub-area. West side of this borderline belongs to the Irrawaddy Delta sub-basin and east side belongs to Pegu-Yoma Sittaung basin as well. Under the Irrawaddy Delta sub-basin, the central business district area (CBD area) is called here as Downtown sub-area which is separated as one sub-area due to the high density of buildings and population. Thanlyin (Thilawa) area is separated as one sub-area due to its separated area by Bago and Yangon River as well. Under the Pegu-Yoma Sittaung basin, east side of this borderline is called as Eastern sub-area. However, the northern side from the north end of the circular railways is just separated as North West sub-area for the Irrawaddy Delta sub-basin and North East sub-area for the Pegu-Yoma Sittaung basin as well due to the very small numbers of data in those sub-areas.

At the same time, soil properties of Yangon area are compared with those of typical soft clays in other countries, such as clays in Japanese marine clays [13)], clays in Hochiminh City and the others.

Panoramic view photos taken from Sakura Tower in Downtown Sub-area are shown in Photo 4-1 to 4-2.

Coverage area (data source area) of this study is shown in Figure 4-4.

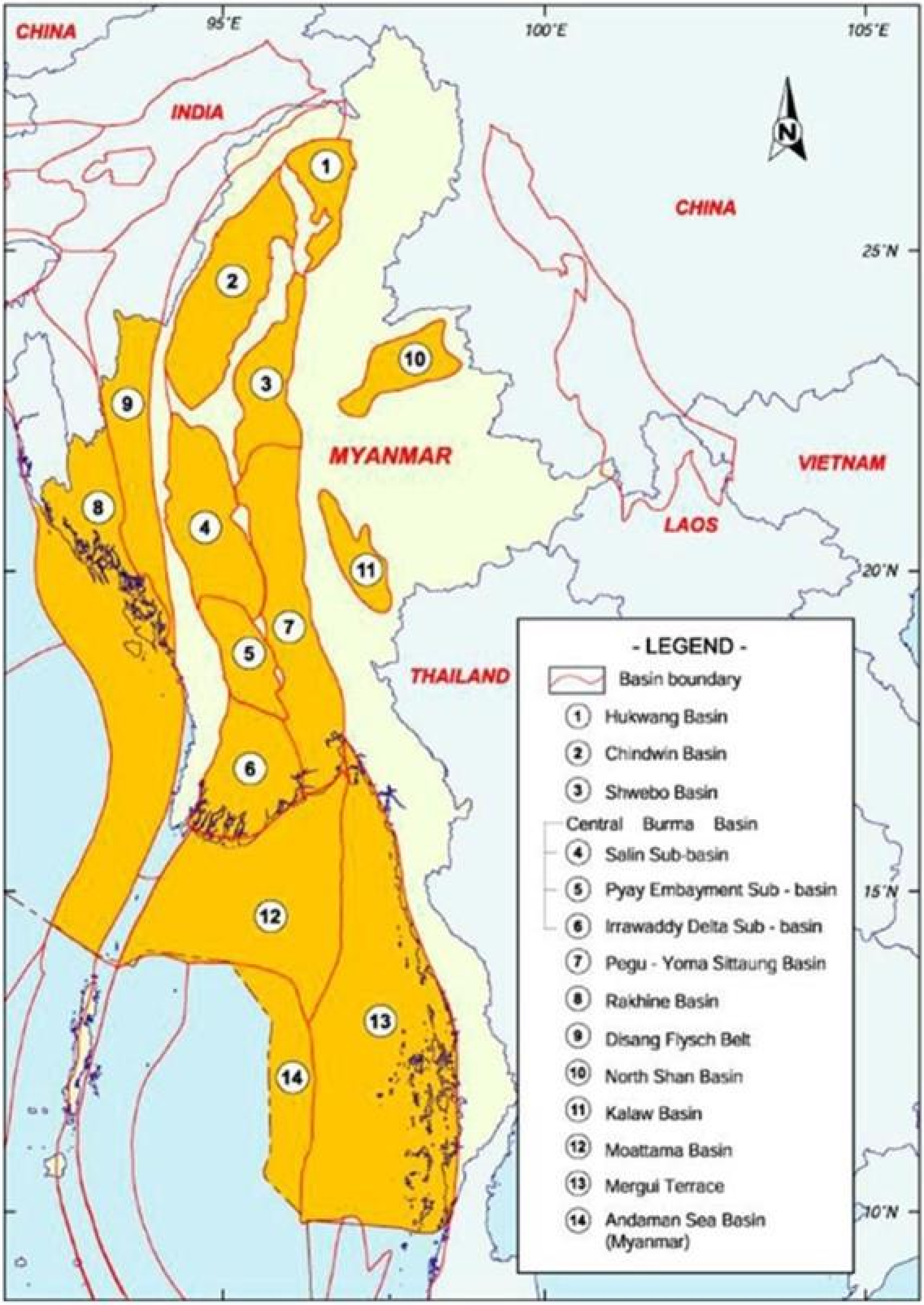


Figure 4-1 Sedimentary Basins in Myanmar [3)]

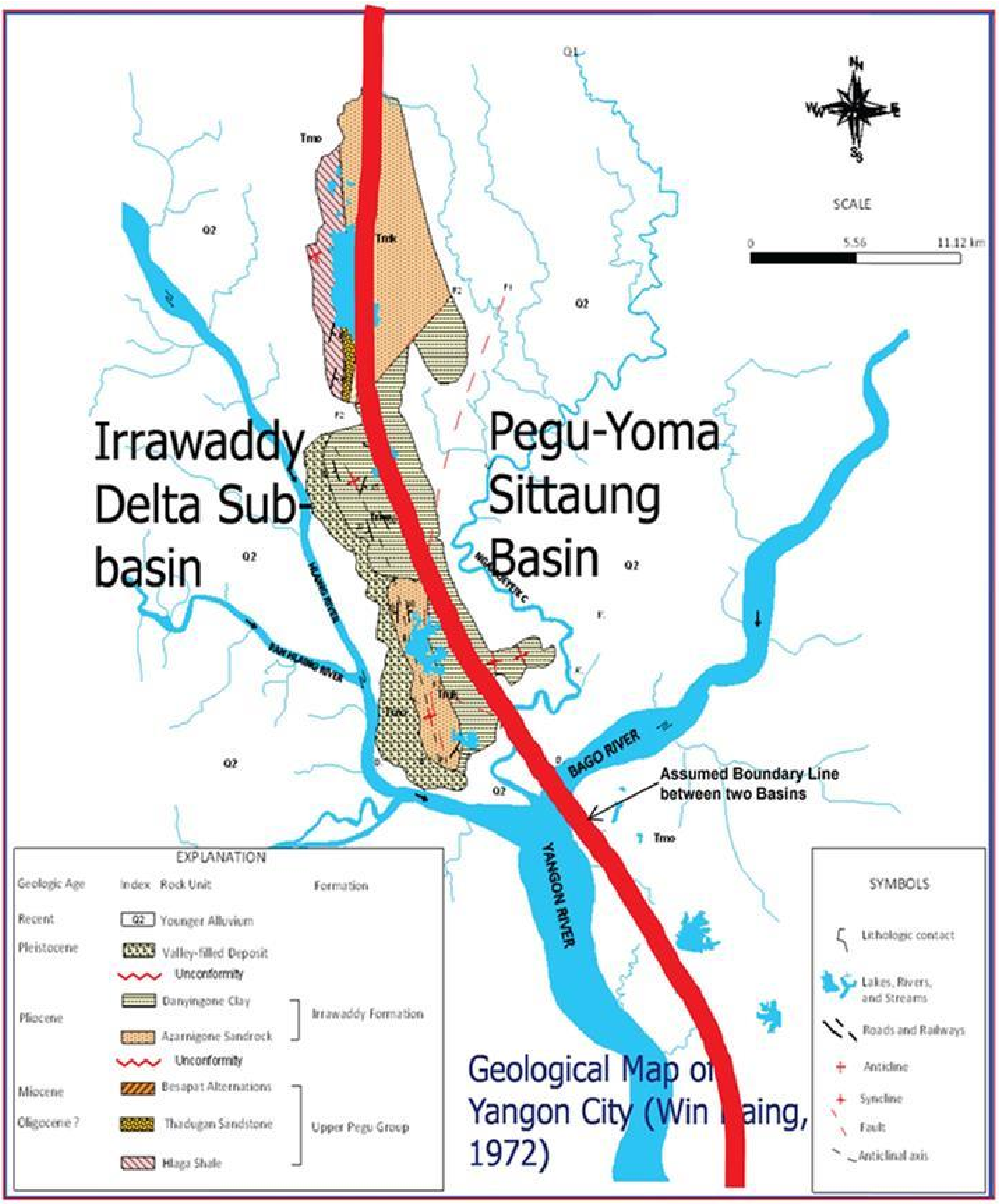


Figure 4-2 Assumed sedimentary borders in Yangon
(Adding border line on geological map [4)])

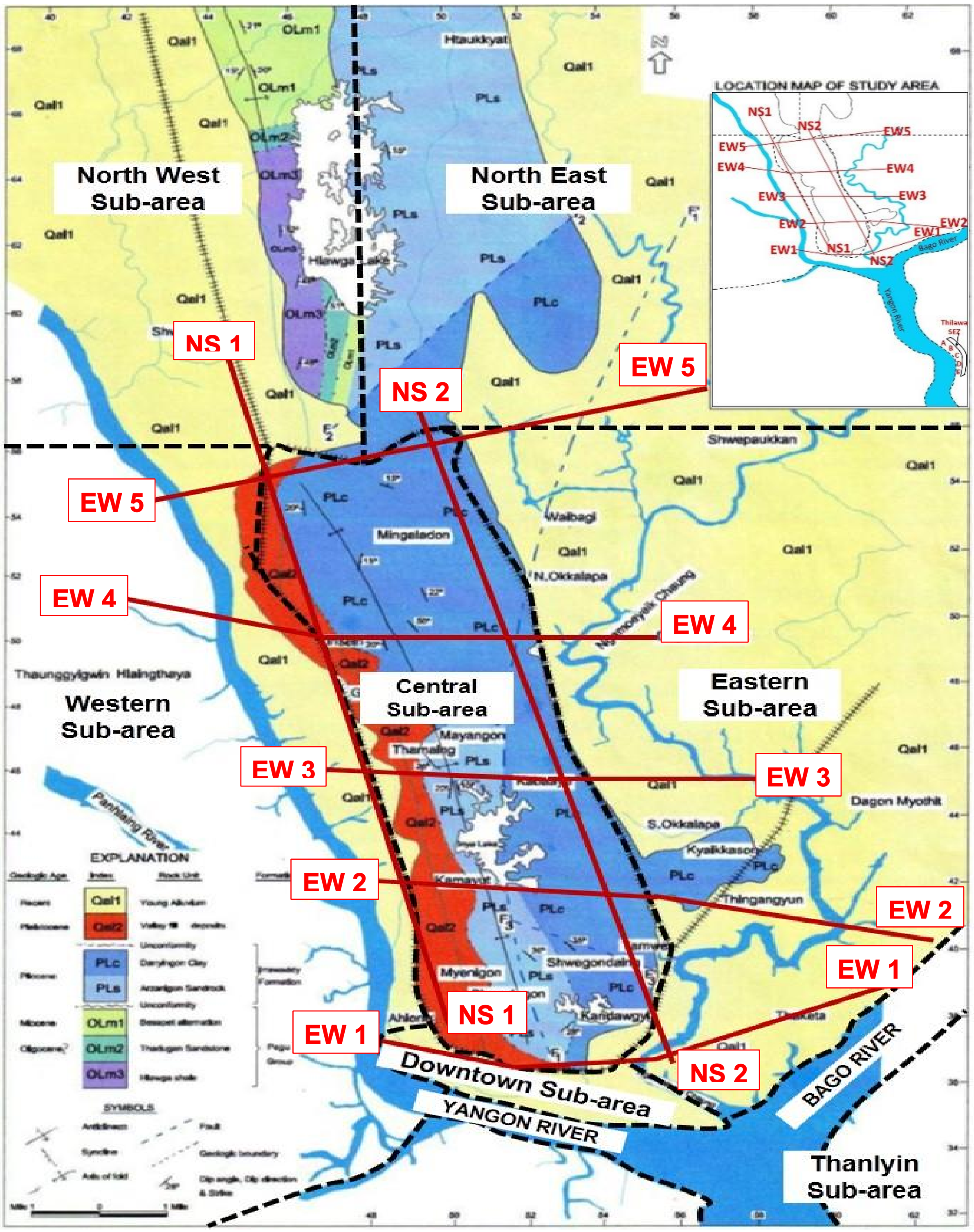


Figure 4-3 Geological Map and Sub-areas in Yangon (Black dash line shows border lines of sub-areas and solid line shows soil profile section lines on geological map, map source: Win Naing (1972) [4], Upgraded by W.R.U.D (1992))

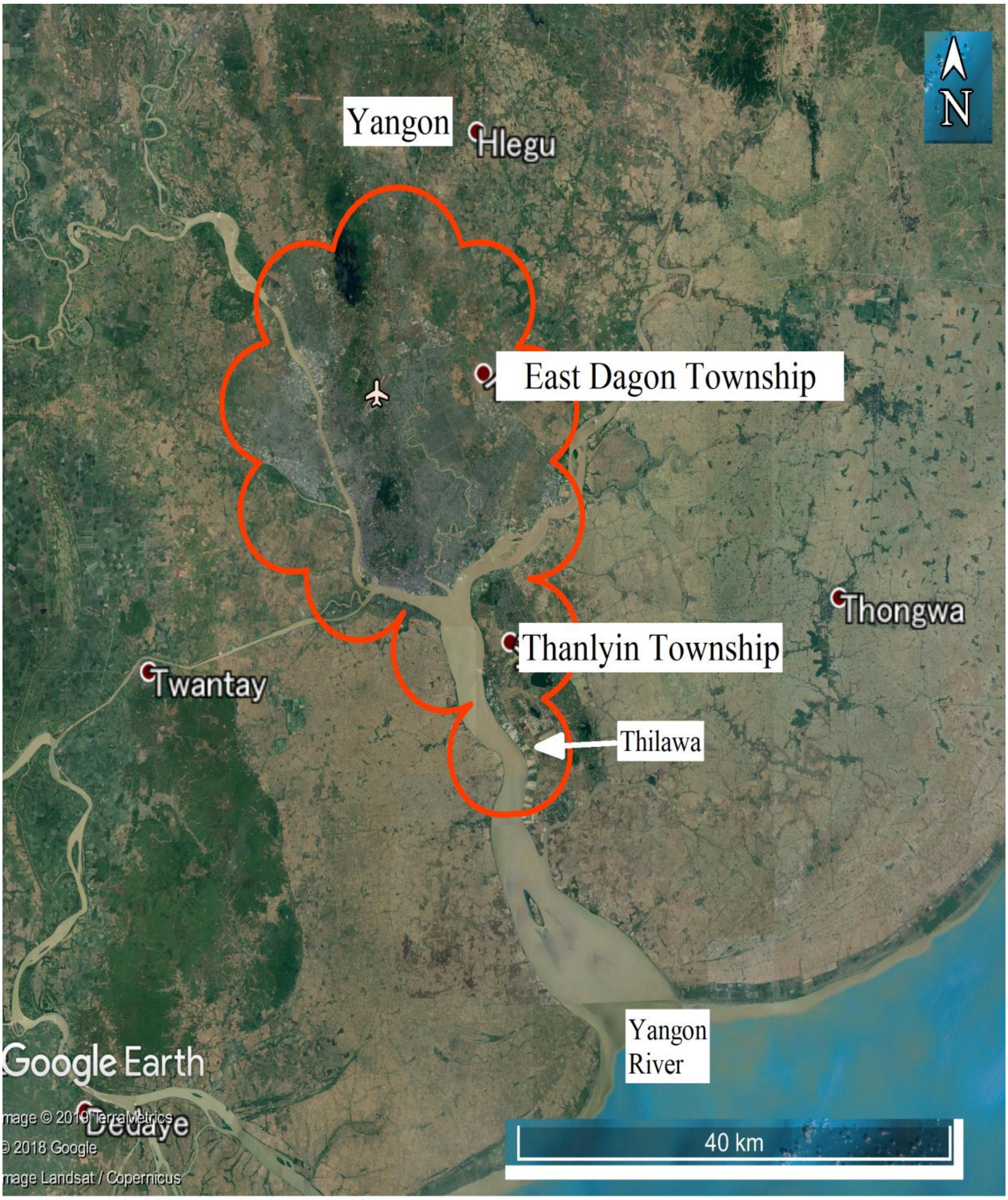
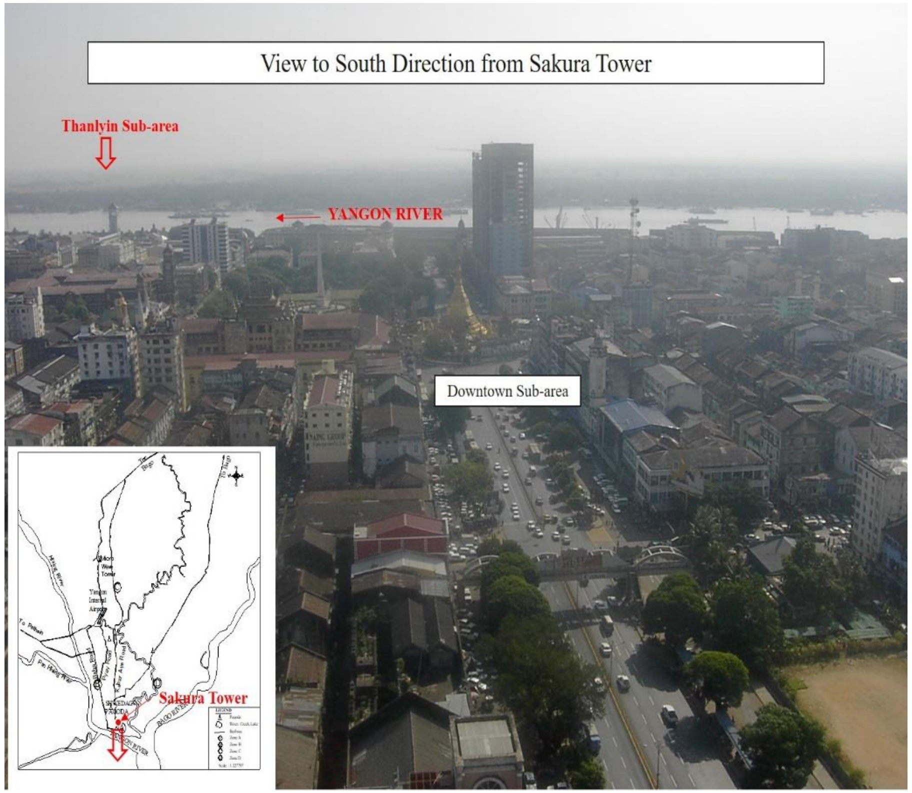


Figure 4-4 Coverage area of this study



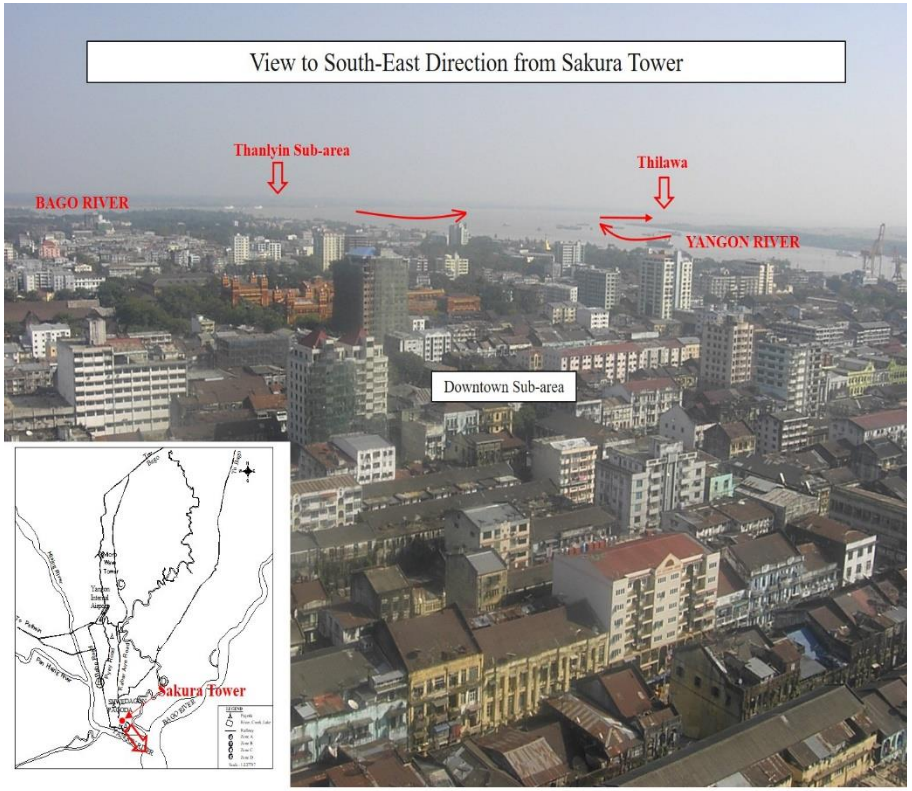
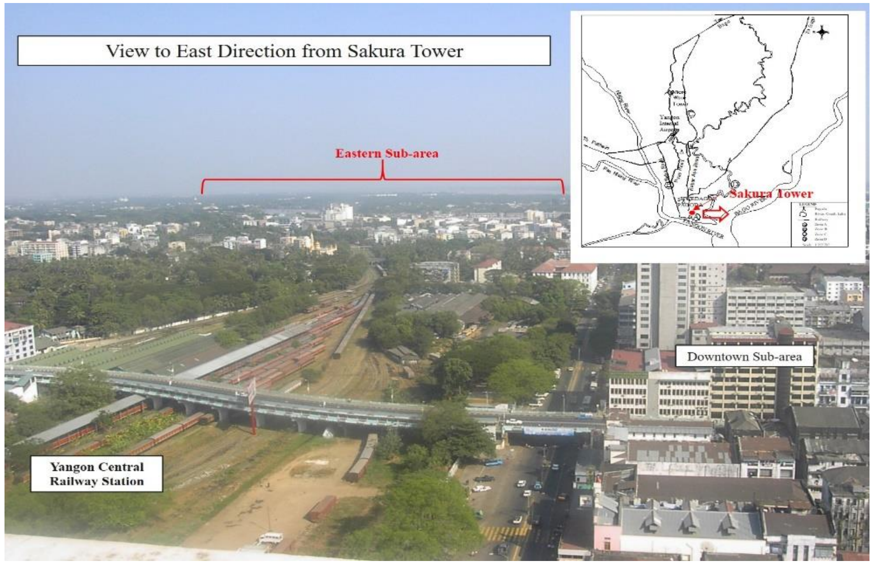
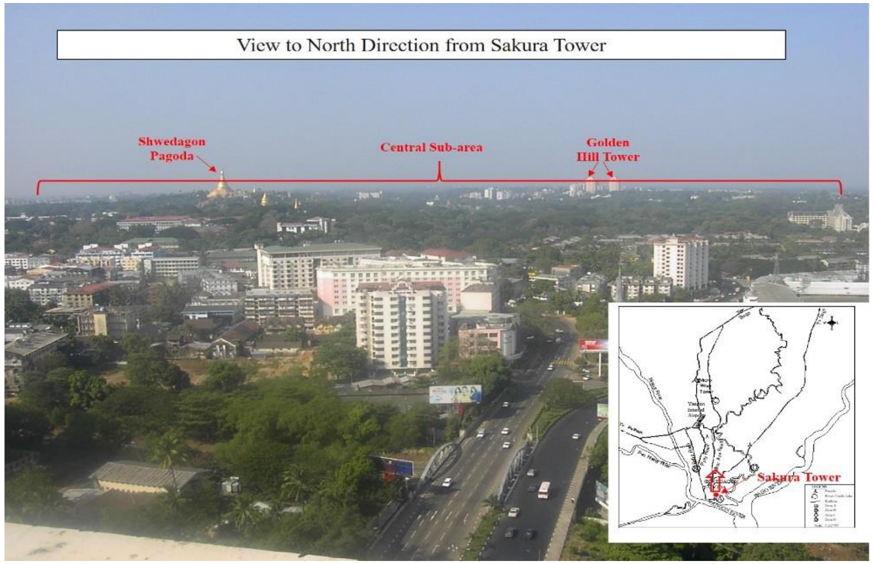


Photo - 4.1 Photos of Panoramic View to South and South-East Direction





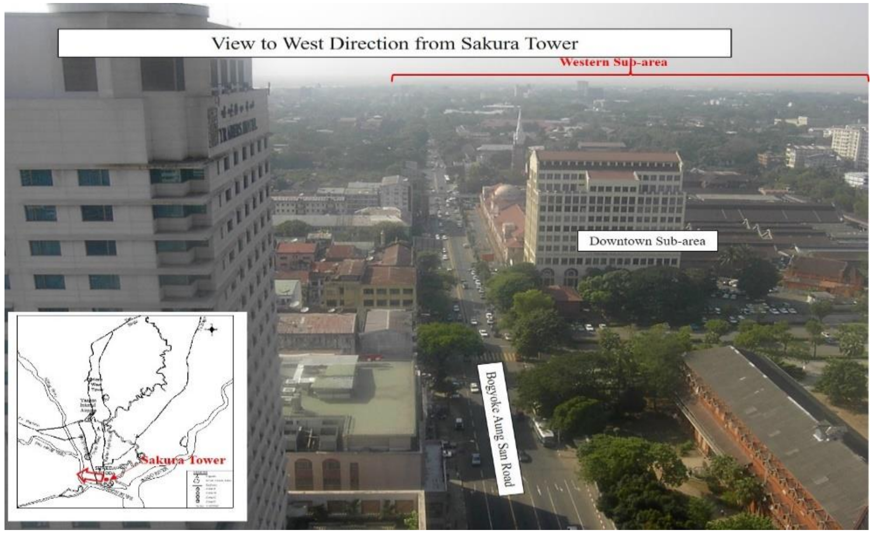


Photo - 4.2 Photos of Panoramic View to East, North and West Direction

4.2 Topography, Geology and Ground Stratification in Yangon Area

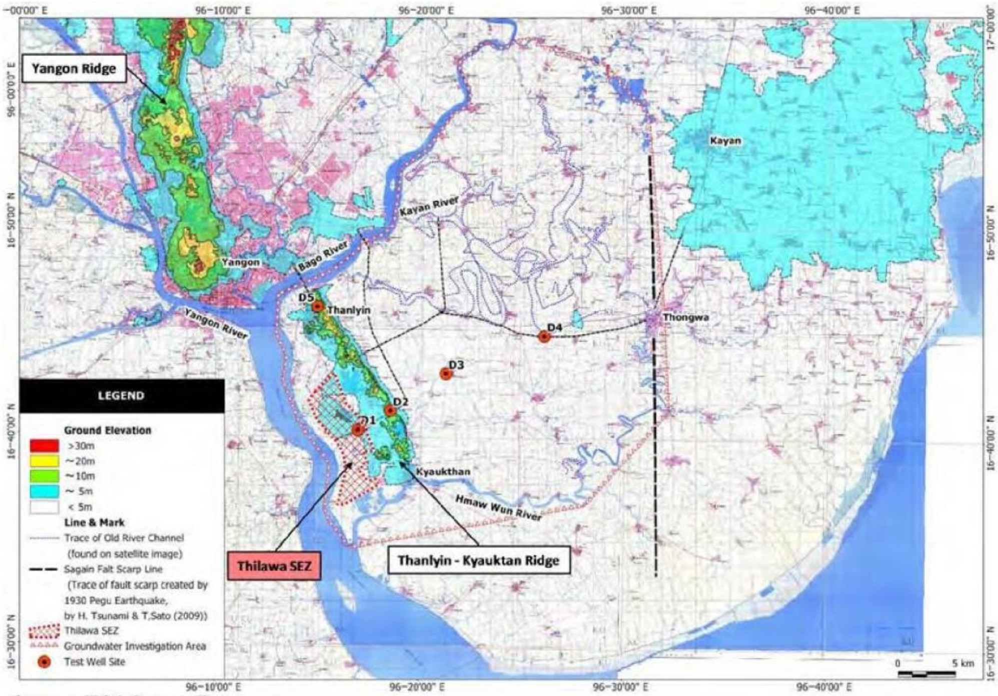
Many researches and surveys on topography, geology and ground stratification have been carried out in Yangon area up to now [1), 3), 4), 5), 6)]. From those past researches, studies and surveys, the topography, geology and ground stratification are described as follows.

4.2.1 Topography of Yangon Area

Yangon-Mingaladon Ridge in Yangon area, which is anticlinal ridge exists in north-north west to south-south east direction at the center of Yangon. The hill height of Shwedagon Pagoda, that is base of pagoda, is about 50 meter above mean sea level. Then Thingangyun Ridge and Thanlyin-Kyauktan Ridge can be seen in south-southeast direction from those continuity.

Topography of Yangon area can be recognized as follows [1), 3), 4), 5), 6)].

1) Hilly area in the Central sub-area can be recognized with ridges. (refer to Figure 4-4 and Figure 4-5).
2) Flat rolling area in the both limb sides of Yangon Mingaladon Ridge and Thanlyin Kyauktan Ridge.
3) Low land and swampy areas which located at southern part of Yangon such as some parts of Thilawa, Dala and along Pazundaung creek, Ngamoeyeik chaung and Pann Hlaing river.
4) Young alluvial deposit area covered at east and west side of Yangon Mingaladon Ridge such as Western sub-area and Eastern sub-area. And they covered Downtown sub-area, Thanlyin sub-area and Dala as well.



Source: JICA Survey Team

Source: Data Collection Survey on Water Resources Potential for Thilawa Special Economic Zone and Adjoining Areas Final Report, September 2014, JICA [6)]

Figure 4-5 Topography and Elevation in and near the Groundwater Investigation Area

4.2.2 Geological Conditions in Yangon Area

Yangon and its surrounding areas including ridges and deltaic low lands and as extensional rolling region of Pegu-Yoma anticlinorium. The area is located in North to South trending sedimentary basin containing a thick Tertiary and Quaternary deposits. Tertiary deposits belong to the Hlawga shale of lower Pegu Group, Thadugan sand stone (lower) and Besapet alternation of upper Pegu Group, and Arzarnigon sand rock (lower) and Danyingon clay (upper) of Irrawaddy Formation (Win Naing, 1972) [4)]. The Quaternary sediments widely distributed at the outskirt of the Yangon, consisting of thick, high plastic, stiff clay underlain by sand and silt. Win Naing (1972) classified generally the Quaternary sediments into valley-filled deposit and the alluvium. The valley-filled deposit includes the Pleistocene older alluvium of a particular type of terrace deposit (Leicester, 1959 and Kyaw Htun, 1996) of unconsolidated gravels, sands and silts and the alluvial is younger age clayey deposit (Aung Lwin, 2012) [5)].

The regional dip is toward the east having a low to moderate dip angle and Western dip slope is very narrow and often covered by the younger Alluvium.

The Yangon-Mingaladon ridge is a long narrow anticlinal ridge of an anticlinal fold plunging towards the north and physiographic evidence of the nose of the anticline is observed at Danyingon. This Yangon-Mingaladon Anticline is an asymmetrical rather than a symmetrical one (Win Naing, 1972) [4].

The Sagaing Fault is a continental transform fault more than 1000 km long that accommodates right-lateral motion between the India and Sunda plates [23]. There are several faults in and surrounding the Yangon area as shown in Figure 4-6. The largest fault in Yangon area is Mingaladon Fault and is observed with its lineament at east side of Yangon International Airport. This fault is considered as a normal fault and its fault plain is estimated to be dipping in a southeast to east direction. Another fault is Danyingon Fault which is understood as a geological boundary between rocks of Upper Pegu Group and Irrawaddy formation in the Hlawga anticline. This fault can be easily observed in the field around Hlawga Lake (Win Naing 1972) [4].

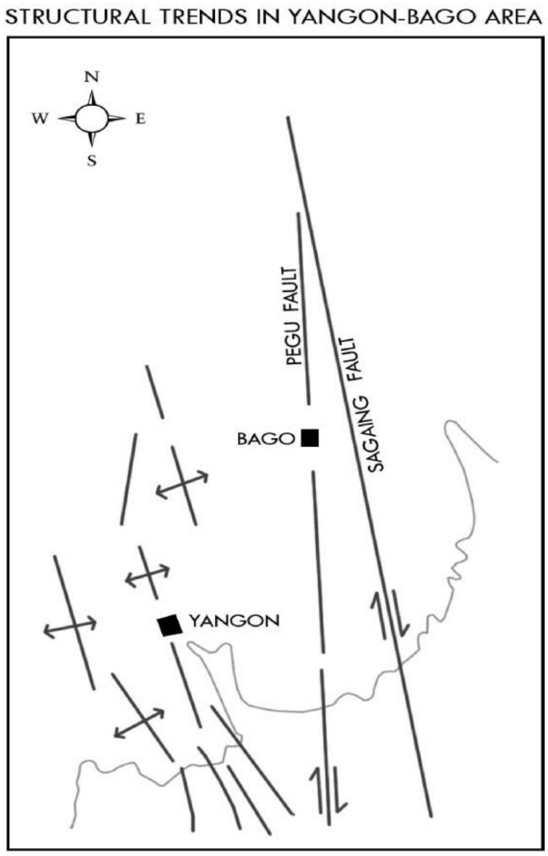


Figure 4-6 Structural trends in Yangon Region (derived from Oil map) [1]

According to Win Naing (1972) [4], the general succession of rocks underlying at the Yangon area is as follows.

Table 4-1 Regional Lithostratigraphic Units of Yangon Area and Pegu-Yoma [4]

System	Series	Yangon Area	Bago Yoma
Quaternary	Recent	Young Alluvium	Alluvium
	Pleistocene	Valley-fill Deposit	
Tertiary	Pliocene	Danyingon Clay	Irrawaddy Formation
		Arzanigon Sandrock	
	Miocene	Besapet Alternation	Obogon Formation
		Thadugan Sandstone	Kyaukkok Formation
	Oligocene (?)	Hlawga Shale	?

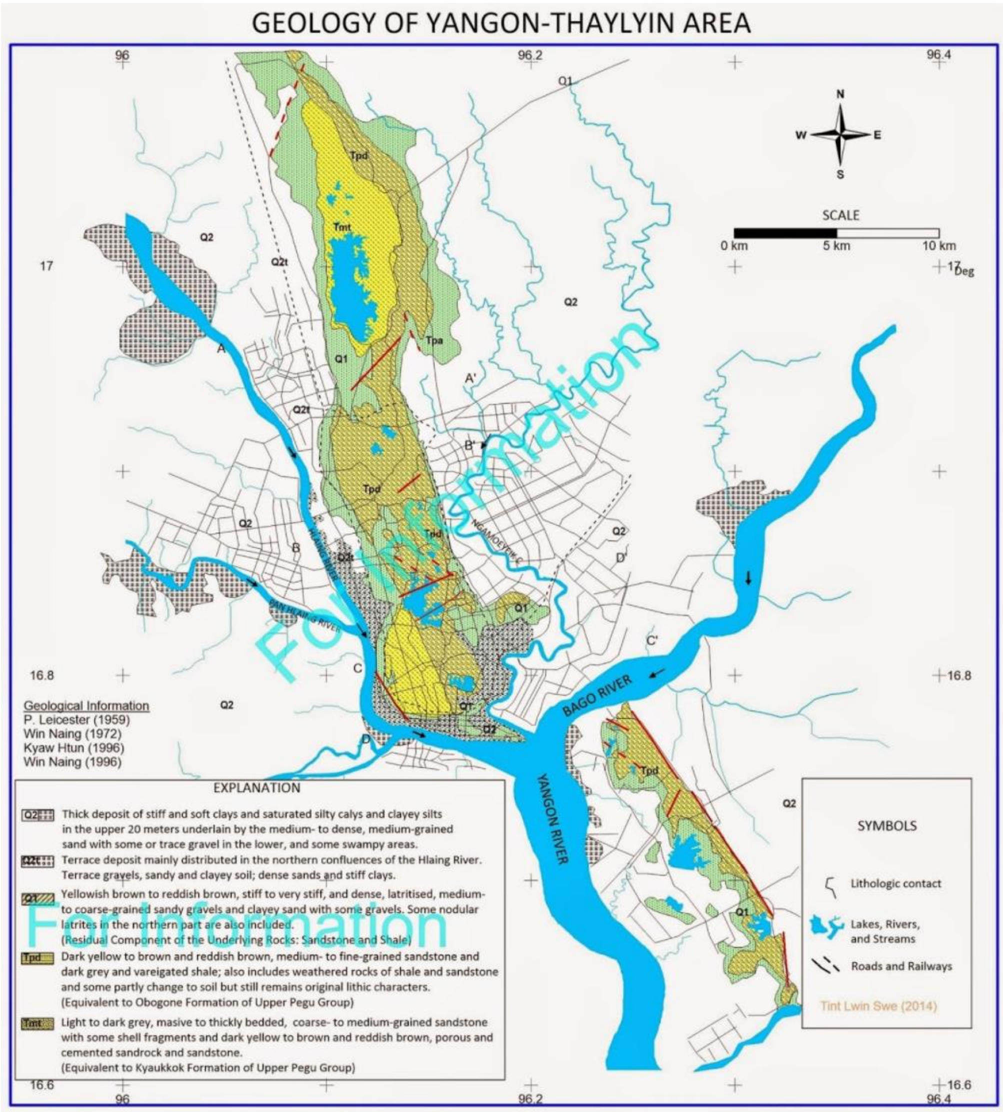


Figure 4-7 Geology of Yangon Thanlyin Area [6]

The Irrawaddy Formation is mainly composed of loose, friable, light yellow sand rocks, frequently iron stained, concretion of hard calcareous sandstone and blue gray sandstone and shales.

Valley Filled Deposit is composed mainly of a thick sequence of loose to dense highly pervious, interbedded sand and fine to coarse gravels, Clay and silt lens occur interbedded with sand and gravel. Arzarnigon Sand rocks unit lies under Danyingon clays and it composes of yellowish gray to blueish gray sand rock, fine to coarse grained sometimes very coarse to gritty with intercalated clays and mudstones and siltstones. Danyingon clays comprises mainly of blue clays, siltstone with interbedded sand rocks. The clay bands show current bedding (Win Naing, 1972 [4]).

The general history of the ground generated in Yangon area can be described as follows (Win Naing, 1972 [4]).

1) The southern part of Pegu-Yoma including Yangon area appears to rise in the form of an anticline at the end of Pliocene period.
2) Valley filled deposit of gravels and gravelly sand was deposited in the area with the lowering of sea level.
3) The thick gravels have been deposited rapidly in the flank of rising anticline at the end of Pliocene time due to the requirement of a cool dry climate.
4) Sea level transgressed again up to 6 meters above present sea level at about 6,000 BP to 7,000 BP, to allow for the deposition of the Younger Alluvium and denudation continued thorough to the present at the end of the Pleistocene (glacial period).

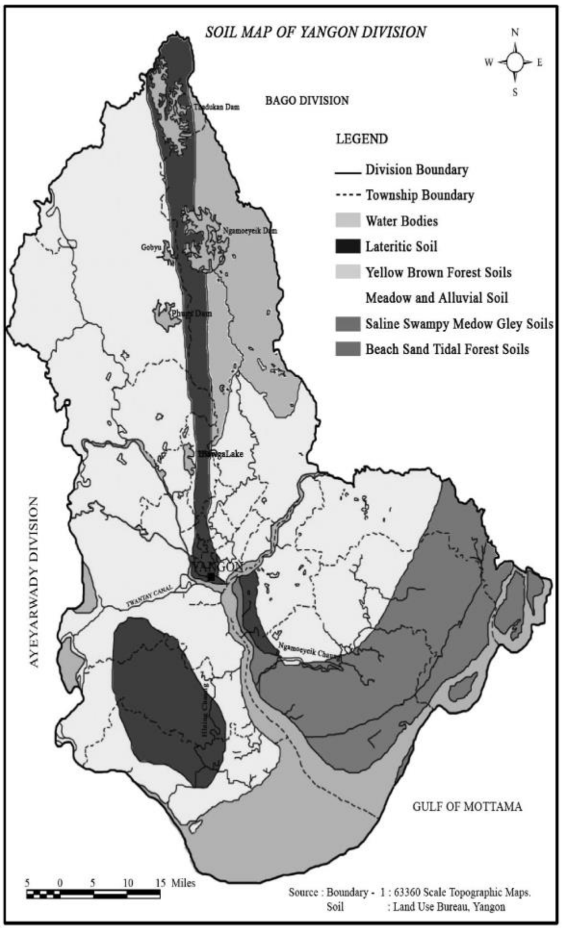


Figure 111 Soil Map of Yangon Region (Hla Hla Aung, 2011) [1]

The geological conditions in Yangon Area have been summarized as follows [1), 5)].

Yangon area is underlain by alluvial deposits (Pleistocene to Recent), the non-marine fluviatile sediments of the Irrawaddy Formation (Pliocene), and the hard, massive sandstones of the Pegu series (early – to late Miocene). The alluvial deposits are composed of gravel, clay, silts, sands, and laterite, which lie upon the eroded surface of the Irrawaddy formation at 3-4.6m above sea level. The central part of Yangon area is occupied by the anticlinal ridge as a 'backbone' at 30 m above the mean sea level and is covered with sand, sand rock, soft sandstone, shale, clays, and the laterite from the Irrawaddy Formation. The hard compact sandstone and shale of the Pegu series can be found at the north-west corner of Hlawga lake with a NNW-SSE strike dipping to the east [4)]. The alluvial deposits are found in the surrounding area of the ridge, whereas lateritic soils can be found along the ridge (Hla Hla Aung, 2011) [1)] (refer to Figure 4-5 [6)] and Figure 4-8 [1)]).

Laminated thinly weathered shale was exposed in Shwegondaing area, when the motor road extension works in 2003 (Aung Lwin, 2012) [5)]. The unconsolidated sandstone was found there when the soil investigation was carried out for Shwegondaing Flyover Project in 2012 [7)] and 2013 [8)].

4.2.3 Ground Stratification in Yangon Area

Figure 4-9 and Figure 4-10 show the soil profiles utilized for engineering purposes in the Yangon Area at the typical cross section marked in Figure 4-3. The ground stratification in the Yangon area changes depending on the distance from the central ridge (the Tertiary deposits) of Yangon. As shown in the East to West sections in Figure 4-9 and Figure 4-10, the North to South sections in Figure 4-11 and the Thanlyin sub-area section separately shown in Figure 4-12 (Yamada et al., 2018 [11)]). The location and ground stratification of each sub-area is described from the ground surface with symbols for soil profiles as follows.

a) North West Sub-area

The North West Sub-area is located at the northern side of the Mingaladon Township and at the west side of Hlawga Lake, as shown in Figure 4-11. At the border between the North West Sub-area and the North East Sub-area, Tertiary (Oligocene to Miocene) clay and sand deposits are found. However soft to firm clay deposits are found around the flat plain area at the western side of the tertiary deposits as shown in Figure 4-10 (Refer to the north end of Section NS-1). Typical soil layers found in this flat plain region are, from the ground surface downwards, as follows;

1) AC-I: Alluvial clay (very soft to soft), 2) AC-II: Alluvial clay (firm to stiff), 3) AS-I: Alluvial sand (loose to medium dense), 4) PlS-I: Tertiary sand deposit (medium dense to dense), 5) PlS-II: Tertiary sand deposit (dense to very dense).

The soft to firm clays studied in this region are usually distributed at shallower depth than G.L. - 25 m.

b) North East Sub-area

The North East Sub-area is located at the eastern side of the North West Sub-area. At the border between the North West Sub-area and the North East Sub-area, younger Tertiary (Pliocene) clay and sand deposits than those of the North West Sub-area can be found, as shown in Figure 4-11. However, soft to firm clay deposits are seen around the flat plain area on the eastern side of these tertiary deposits. As shown at the north end of Section NS-2 of Figure 4-11, the typical soil layers found in the tertiary deposits are, from the ground surface downwards, as follows;

1) AC-I: Alluvial clay (very soft to soft), 2) AC-II: Alluvial clay (firm to stiff), 3) AS-I: Alluvial sand (loose), 4) PlC-I: Tertiary clay deposit (very stiff to hard), 5) PlC-II: Tertiary sand deposit (hard).

The soft to firm clays studied here are usually distributed at depths shallower than G.L. - 25 m.

c) Central Sub-area

The Central Sub-area is located just inside the Yangon circular railway line. This area is relatively hilly and is composed of tertiary (Pliocene) clay and sand deposits. This area sits in a NNW to SSE direction like the backbone of Yangon area. As shown at the center portion of Section EW-2, 3, 4, 5 of Figure 4-9 and Figure 4-10 and most portions of Section NS-1and 2 of Figure 4-11, the typical soil layers seen at Central Sub-area are, from the ground surface downwards as follows;

1) AC-I: Alluvial clay (very soft to soft)/ AC-II: Alluvial clay (firm to stiff), 2) AS-I: Alluvial sand (loose to medium dense)/ AS-II: Alluvial sand (medium dense), 3) PlC-I: Tertiary clay deposit (very stiff to hard)/ PlC-II: Tertiary sand deposit (hard clay).

Soft to firm clays studied here are usually distributed at shallower depths than G.L. -10 m.

Photo - 4.3 Conditions of underground at Central sub-area in Yangon

(Surface 5~6 meter thick layer is composed of yellowish brown colored lateritic clay and below this layer, firm clay layer can be seen. Excavated depth is about 10 meters. There is no water table, only a little seepage water is confirmed within this depth)

d) Western Sub-area

The Western Sub-area is located at the west side of the Central Sub-area at the flat plain/ lowland area as shown in Figure 4-9, Figure 4-10 and Figure 4-11. Soft to firm clay and loose to medium dense sand are distributed in this region. As shown at the west end of Section EW-2,3,4 and 5 of Figure 4-9 and Figure 4-10, the typical soil layers seen at the Western Sub-area are, from the ground surface downwards, as follows;

1) AC-I: Alluvial clay (very soft to soft)/ AC-II: Alluvial clay (firm to stiff), 2) AS-I: Alluvial sand (loose to medium dense), 3) AS-II: Alluvial sand (medium dense), 4) PlS-I: Tertiary sand deposit (medium dense to dense), 5) PlS-II: Tertiary sand deposit (dense to very dense).

The soft to firm clays studied here are usually distributed at depths shallower than G.L. - 25 m.

e) Eastern Sub-area

The Eastern Sub-area is located at the eastern side of the Central Sub-area and the flat plain/ lowland area as shown in Figure 4-9, Figure 4-10 and Figure 4-11. Soft to firm clay and loose to medium dense sand are distributed in this area. As shown at the eastern part of Section EW-1,2,3 and 4 of Figure 4-9 and Figure 4-10, the typical soil layers seen at Eastern Sub-area are, from the ground surface to the depth direction, as follows;

1) AC-I: Alluvial clay (very soft to soft), 2) AS-I: Alluvial sand (loose to medium dense), 3) AC-III: Alluvial clay (firm to stiff), 4) AS-II: Alluvial sand (medium dense)/ PlC-I: Tertiary clay deposit (very stiff to hard).

The soft to firm clays studied here are usually distributed at shallower depths than G.L. - 20 m.

f) Downtown Sub-area

The Downtown Sub-area is located at the south side of the Central Sub-area and the CBD (Central Business District) area, just north of the Yangon River.

In this area soft to firm clays and loose to medium dense sands are distributed. As shown at the west half of Section EW-1 of Figure 4-9(a), the typical soil layers seen at Eastern Sub-area are, from the ground surface downwards, as follows;

1) AC-I: Alluvial clay (very soft to soft), 2) AS-I: Alluvial sand (loose to medium dense), 3) AS-II: Alluvial sand (medium dense), 4) AS-III: Alluvial sand (dense)/ PlC-I: Tertiary clay deposit (very stiff to hard), 5) PlC-II: Tertiary sand deposit (hard clay), PlS-I: Tertiary sand deposit (medium dense to dense), 6) PlS-II Tertiary sand deposit (dense to very dense).

The soft to firm clays studied here are usually distributed at shallower depths than G.L. - 25 m.

g) Thanlyin (Inc. Thilawa Area) Sub-area

The Thanlyin (Inc. Thilawa Area) Sub-area is located at the south side of the Bago River and the east side of Yangon River as shown in Figure 4-12. This area is also divided into the west and east side with Tertiary deposits along the assumed border line between the Irrawaddy Delta Sub-basin and the Pegu-Yoma Sittaung Basin as shown in Figure 4-2. As shown in Figure 4-2, the typical soil layers seen along Yangon river beside the Thilawa SEZ area (refer to Point A to E of Figure 4-12), from the ground surface downwards, as follows;

1) AC-I: Alluvial clay (Clay-I & Clay-II Layer, very soft to soft), 2) AC-II: Alluvial clay (Clay-III Layer, firm to stiff), 3) AC-III: Alluvial Clay (Silty Clay/ Clayey Silt/ Sandy Clay/ Clay- IV Layer, firm to stiff) 4) AS-II: (Silty Sand/ Sand Layer, dense)

The soft to firm clays studied here are usually distributed at depths shallower than G.L. - 25 m.

Typical photos for lateritic clay and soft to firm clays (AC-I and AC-II) are shown in Photo 4.4 to Photo 4.6. The photos of Lateritic clay, AC-I and AC-II are shown below.


Photo - 4.4 Photos of lateritic clay


At Central Sub-area

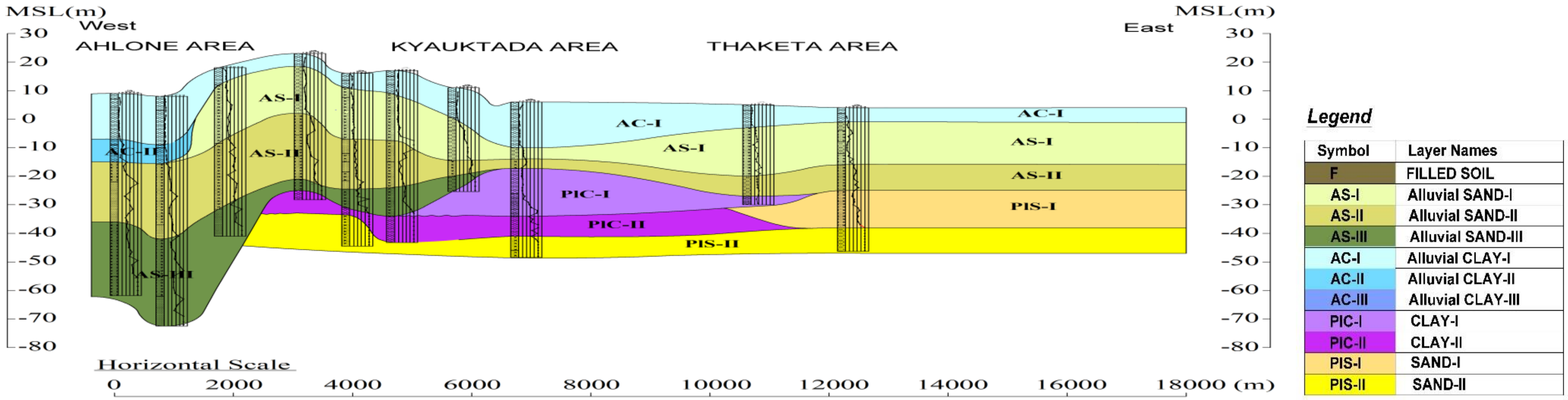
At Other Sub-area

Photo - 4.5 Photos of AC-I clay

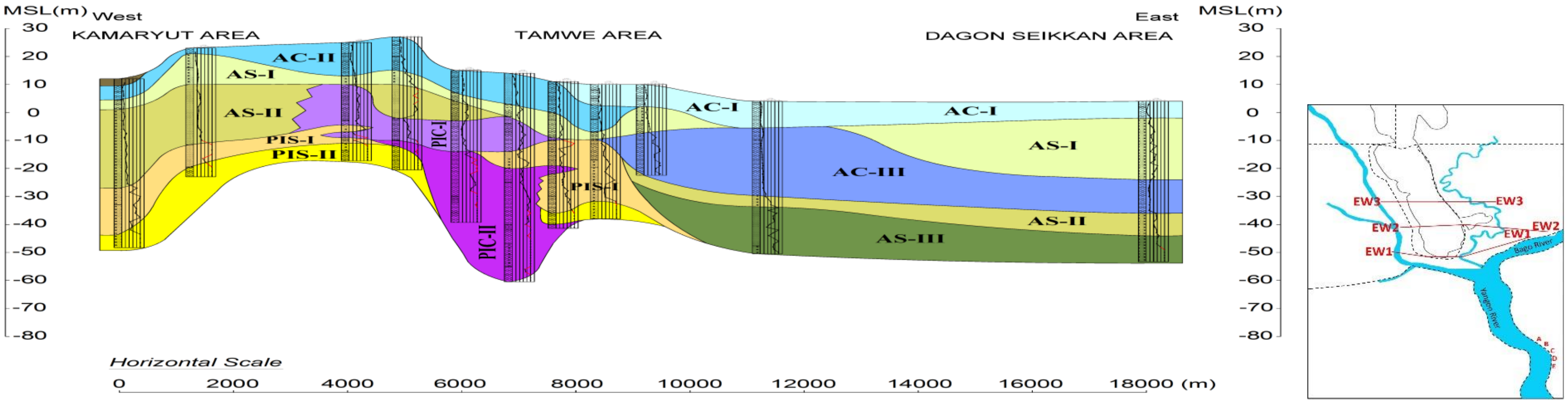

At Central Sub-area

At Other Sub-area

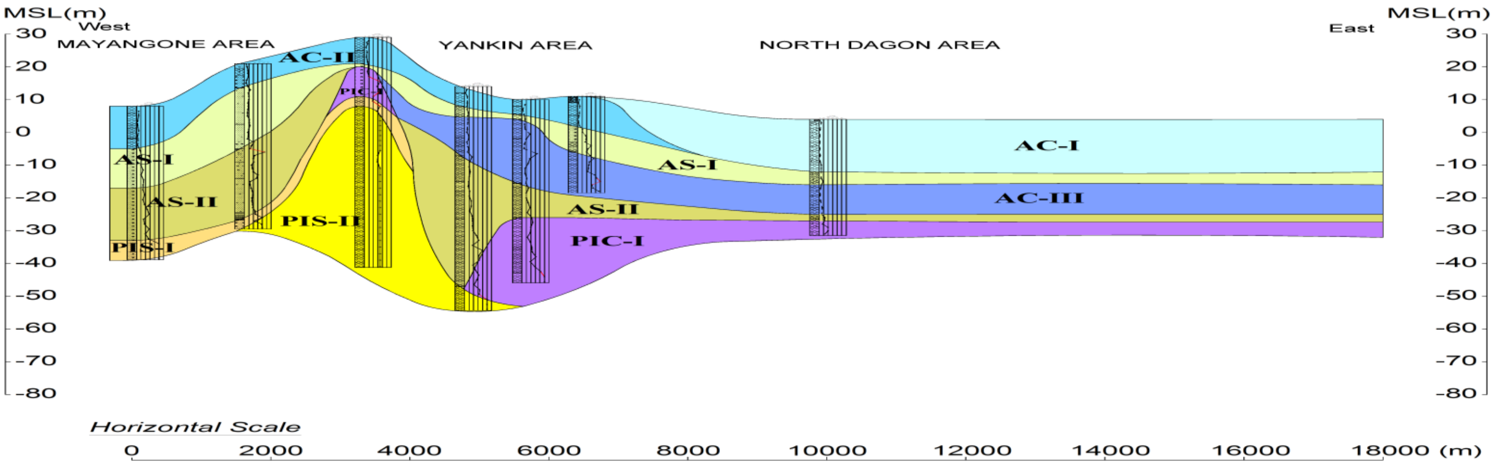
Photo - 4.6 Photos of AC-II clay



(a) Cross Section EW-1



(b) Cross Section EW-2



(c) Cross Section EW-3

Legend

Symbol	Layer Names
F	FILLED SOIL
AS-I	Alluvial SAND-I
AS-II	Alluvial SAND-II
AS-III	Alluvial SAND-III
AC-I	Alluvial CLAY-I
AC-II	Alluvial CLAY-II
AC-III	Alluvial CLAY-III
PIC-I	CLAY-I
PIC-II	CLAY-II
PIS-I	SAND-I
PIS-II	SAND-II

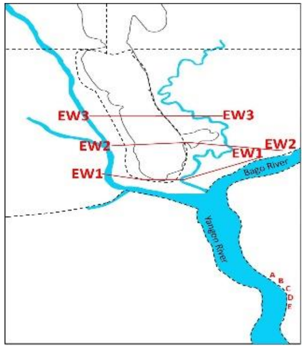
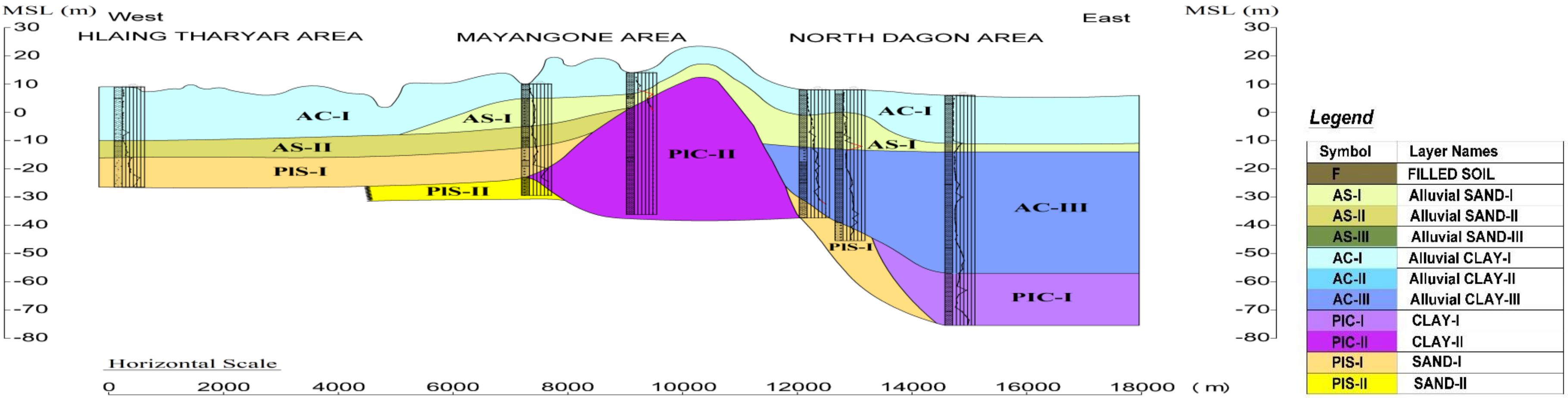
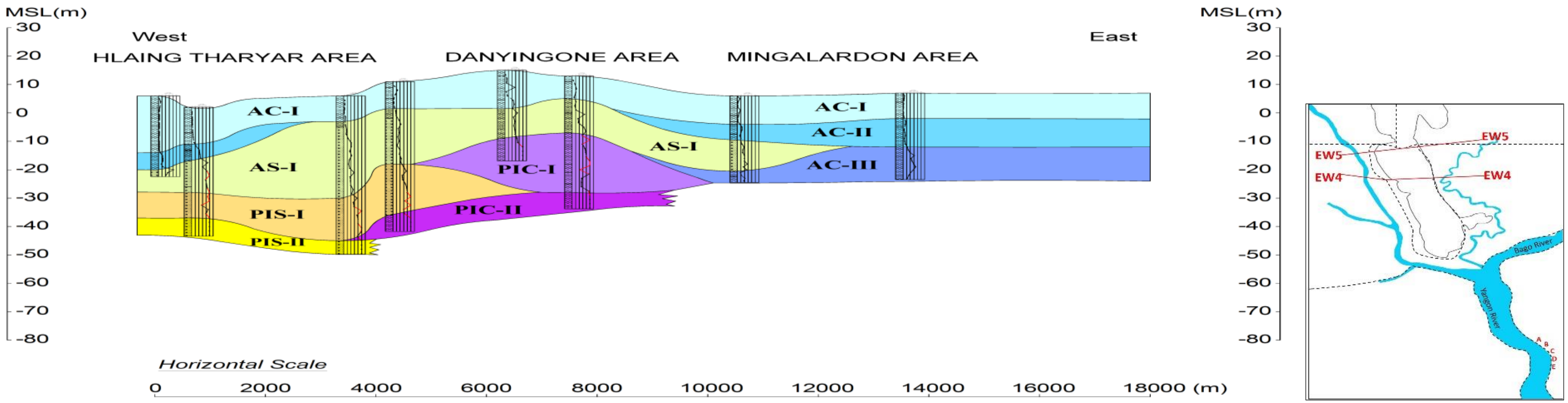


Figure 4-9 East to West Soil Profiles for Engineering Purpose in Yangon Area (EW-1 to EW-3)

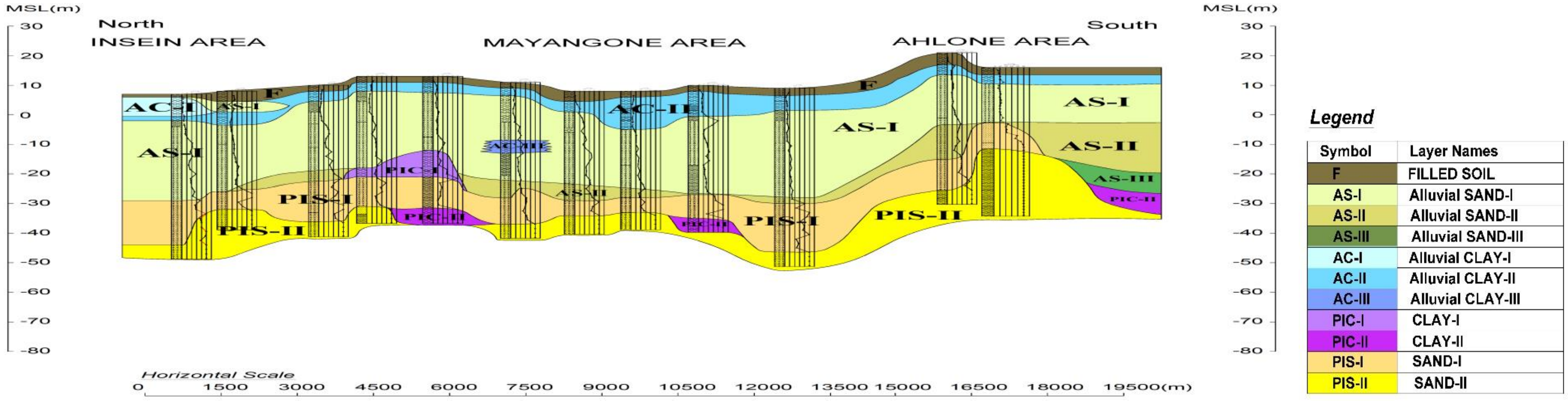


(d) Cross Section EW-4

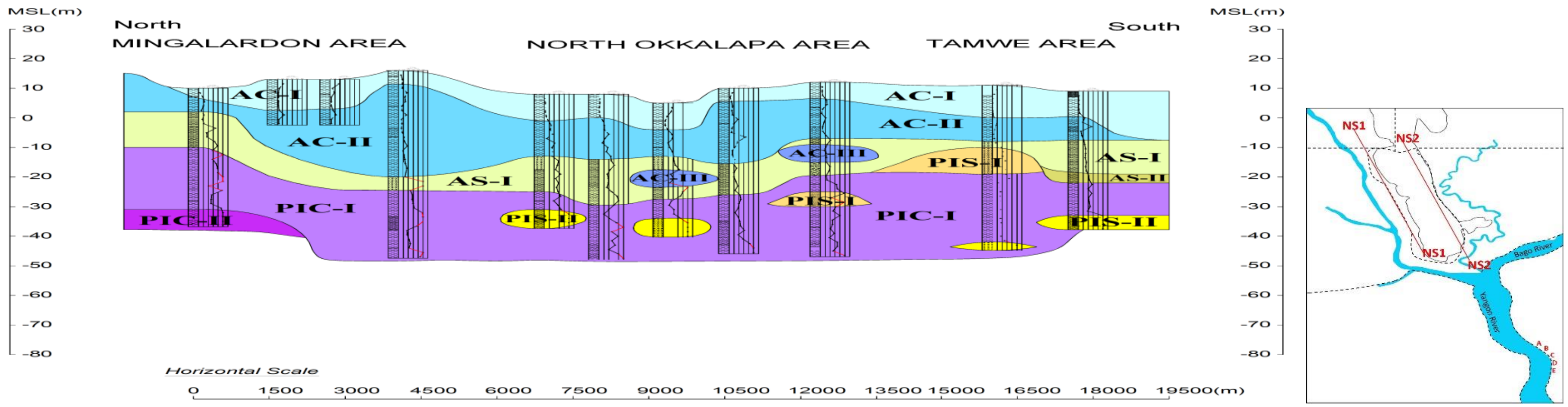


(e) Cross Section EW-5

Figure 4-10 East to West Soil Profiles for Engineering Purpose in Yangon Area (EW-4 to EW-5)

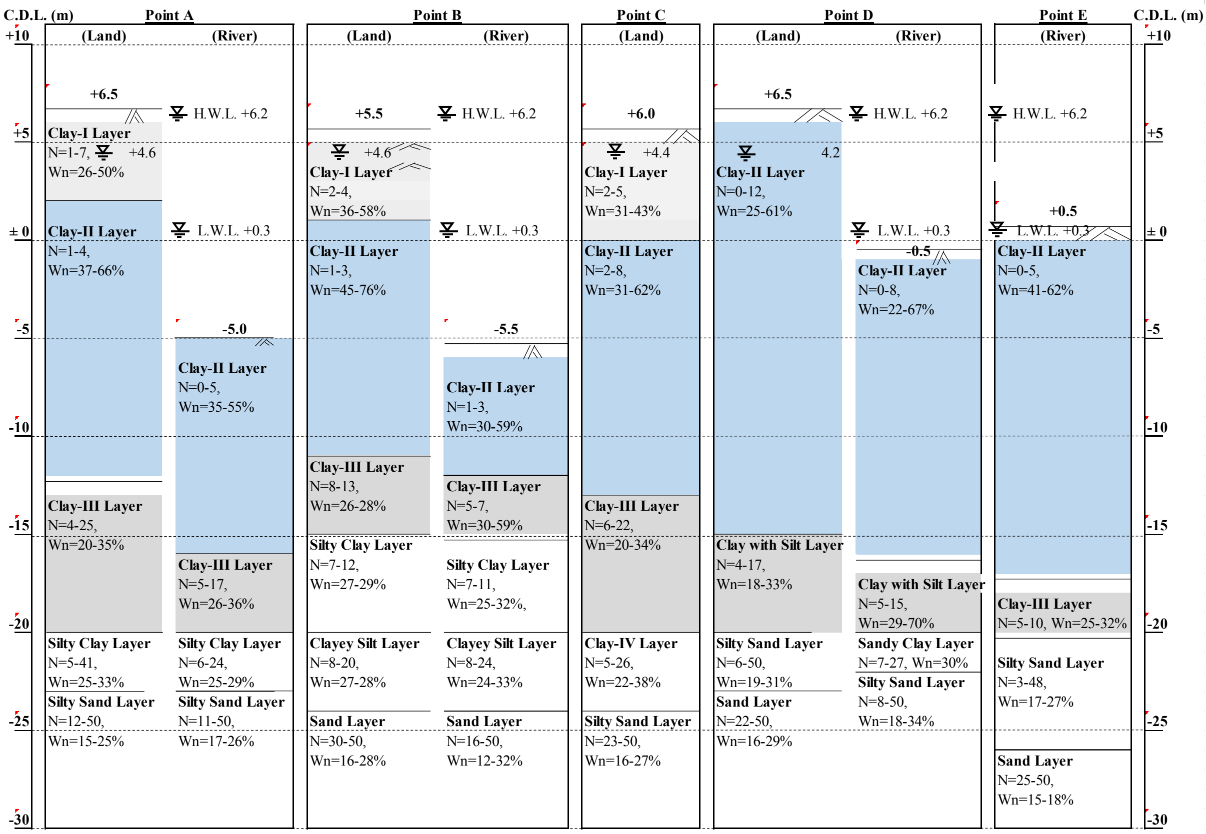


(a) Cross Section NS-1



(b) Cross Section NS-2

Figure 4-11 North to South Soil Profiles for Engineering Purpose in Yangon Area (NS-1 to NS-2)



1) AC-I: Alluvial clay (Clay-I & Clay-II Layer, very soft to soft),
2) AC-II: Alluvial clay (Clay-III Layer, firm to stiff),
3) AC-III: Alluvial clay (Silty Clay/ Clayey Silt/ Sandy Clay/ Clay-IV Layer, frim to stiff),

Figure 4-12 Soil Profiles for Engineering Purpose in Yangon area (From Point A to Point E at Thilawa in Thanlyin Sub-Area) (Yamada et al., 2018 [11])

4.2.4 Estimation of underground structure by microtremor observation in Yangon

Microtremor measurements were performed for the purpose of investigating the seismic amplification characteristics of the ground in Yangon. At the same time, the estimation of the deep underground structure of Yangon was carried out. In this clause, the underground structure in Yangon estimated by microtremor observation results is described.

Yangon City is formed at the confluence area of two large rivers, the Yangon River and the Bago River, and a deep underground structure up to the earthquake base is assumed. There are many boring survey data up to a depth of about 50 meters in Yangon, but deep underground structure exploration using reflection method, gravity survey, etc. have not been conducted, and the information on underground structure is hardly available. Therefore, in microtremor measurement, in addition to a relatively short period area of about 1 second or less that reflects ground characteristics of several tens of meters in depth, long period microtremors (pulsations that reflect deep underground structures of several hundred meters to several kilometers in depth) is also focused.

In this section, H / V spectrum by single point (three-component) tremor measurement carried out near Yangon Branch Office of Fukken Co., Ltd. [9] at North Dagon Township in Eastern Sub-area in Yangon (Refer to Figure 4-13), and dispersion curve of Rayleigh wave by array observation, and deep underground structure estimation based on both results are described. In addition, the comparison with the deep underground structure of Bangkok ground located at the mouth of the big river, Chao Phraya River, where the deep underground structure is assumed as well as Yangon, is also described here [14].

Figure 4-13 Location Map of Microtremor Observation in Yangon [22]

1) Boring data up to GL-40 meters and estimation of deep underground structure

In the vicinity of the tremor measurement point, three boring investigation were conducted at the time of construction of the office building in 2012. Figure 4-14 shows the N-value from the standard penetration test (SPT) and the S-wave velocity V_s estimated from N-values. Soil layers with an N-value of 10 or less are composed of clay, silty clay or clayey silt, and soil layers with an N-value of 10 or more are composed of alternated layers of sandy clay and clayey sand. As for the conversion from N-value to V_s, various conversion formulas are proposed in Japan, but there is almost no research data in Myanmar. For this reason, here, the following equations based on the "Specifications of Highway Bridges, Part V (Japan Road Association)" [15] was used.

$$Clay : V_S = 100N^{1/3} \quad (1)$$

$$Sand : V_S = 80N^{1/3} \quad (2)$$

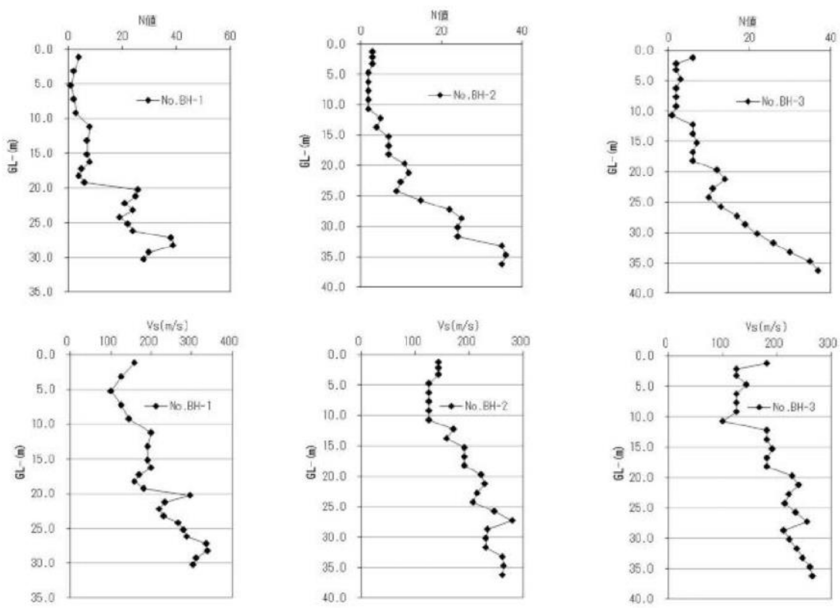


Figure 4-14 N-value by S.P.T and estimated S-wave velocity V_s [22]

First, the V_s profile up to GL-40 meters was determined from the boring data in Figure 4-14 as shown by the thick red line in Figure 4-15. Then, the deep underground structure fitted to the H/V spectrum and Rayleigh wave dispersion curve obtained by microtremor measurement is assumed as shown in Table 4-2 and Figure 4-16. As shown in Table 4-2 and Figure 4-16, V_s up to GL-40 meters is the same as in Figure 4-15. Equations (3), (4) [16), 17)] were used for obtaining the P-wave velocity V_p and the mass density ρ. The underground structural model between GL-40 meter and GL-1,230 meter was determined with reference to the transfer function of SHAKE, dispersion curves of Rayleigh waves, H/ V according to diffuse wave field theory [18), 19), 20)], and YGNgro by Hirokawa et al. [21)] (measured at Yangon University as shown in Figure 4-13).

$$V_p(km/s) = 1.29 + 1.1V_s(km/s) \tag{3}$$

$$\rho(t/m^3) = 1.4 + 0.67\sqrt{V_s(km/s)} \tag{4}$$

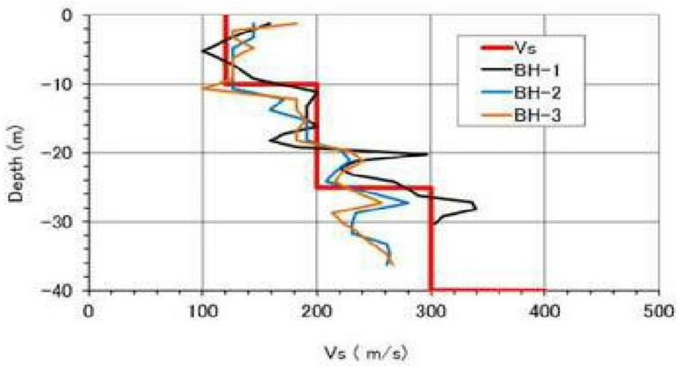


Figure 4-15 S-wave velocity V_s from ground surface to up to G.L. -40 meters [22)]

Table 4-2 Estimated underground structure at North Dagon Township in Yangon [22)]

Depth (m)	Thickness (m)	Vs(m/s)	Vp (m/s)	ρ (t/m³)	Poisson's Ratio
0m – 10m	10	120	1422	1.632	0.4964
10m –25m	15	200	1510	1.700	0.4911
25m –40m	15	300	1620	1.767	0.4822
40m –50m	10	400	1730	1.824	0.4718
50m –130m	80	520	1862	1.883	0.4577
130m –1230m	1100	1200	2610	2.134	0.3660
		3500	6030	2.653	0.2460

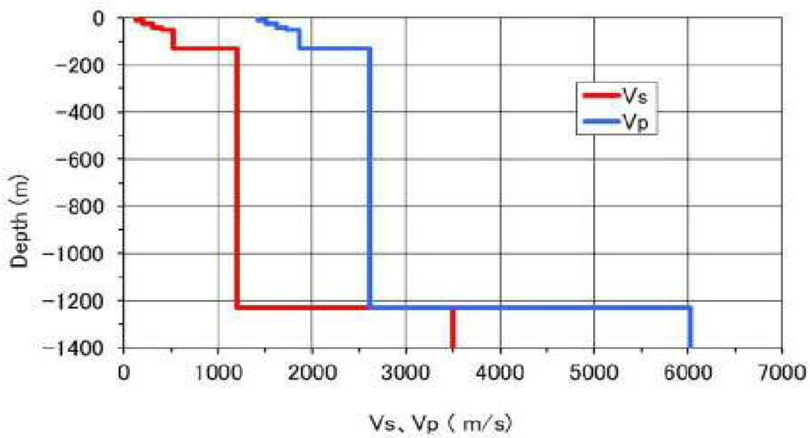


Figure 4-16 Estimated underground structure at North Dagon Township in Yangon [22)]

The peak of the H/ V spectrum is also seen in the long period area around 5 seconds in the tremor measurement by Hirokawa et al.[21]. They mentioned that "There is a high possibility that the peak seen at around 0.2 Hz is reflected in the frequency range below the primary peak, but due to the fact that there is not enough information to determine the underground structure because it is out of the observable frequency range by the array observation. Accordingly the underground structure model is set up with an engineering base of S-wave velocity V_s = 1,100 m/ s at a depth of 310 meters." On the other hand, in the model shown in Figure 4-16, a deep underground model with a V_s = 3,500 m/ s seismic base at a depth of 1,230 meters was assumed to represent the long period peak seen in the H/ V spectrum as shown in Figure 4-18.

2) Comparison with deep underground structure in Bangkok

Morio et al., 2012 [14] have also estimated the deep underground structure of Bangkok from the H/ V spectrum of tremors focusing on pulsation. Bangkok is located about 610 km southeast of Yangon and is formed at the mouth of the Great River and Chao Phraya River, and it is thought that a deep underground structure exists like Yangon.

Table 4-3 Underground structure in Bangkok [22]

Depth (m)	Vs(m/s)	Vp (m/s)	ρ (t/m3)
0m – 7m	60	300	1.70
7m – 15m	80	300	1.70
15m – 30m	290	800	1.85
30m – 60m	350	800	1.90
60m – 120m	410	900	1.90
120m – 240m	550	1100	2.00
240m – 720m	770	1500	2.10
	2000	3800	2.35

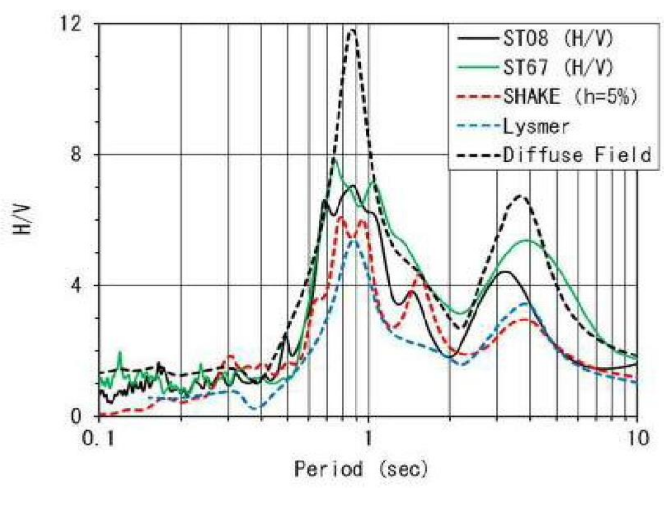


Figure 4-17 H/V Spectrum of the underground in Bangkok [22]

Figure 4-17 shows the transfer function (dumping constant h = 5%) of the S-wave by SHAKE using the H/ V spectrum of the microtremor at the two typical measurement points (ST08 and ST67) and the estimated underground structure (Refer to Table 4-3). In Bangkok ground, there are two major peaks near 4 seconds and near 1 second, as same as shown in Figure 4-18 of this study. The peak near 4 seconds reflects the dynamic characteristics up to the base of depth 720 m, and the peak near 1 second reflects the dynamic characteristics of extremely soft Bangkok clay which has V_s <100 m/ s with a layer thickness of 15 m (colored part in Table 4-3). The shear deformation mode by SHAKE of 4 seconds and 0.8 seconds are shown in Figure 4-18. 4 seconds is the primary mode of the whole ground with 720 meter deep, and 0.8 sec is the mode in which the primary ground mode of the surface ground with 15 m deep is coupled to the tertiary mode of the whole ground, and the primary mode of the soft ground is remarkably outstanding.

On the other hand, even in Figure 4-18 of Yangon ground, peaks around 4 seconds and some peaks appear at around 1 second, but unlike Bangkok ground, it has no extremely soft surface ground. Figure 4-20 shows the first to fourth shear deformation modes. As shown in Figure 4-20, 1.05 seconds of ③ and 0.80 seconds of ④ are modes in which the third order mode and the fourth order mode of the whole ground are coupled to the first order mode of the surface ground (up to GL-130 meters). Deformation of surface ground is outstanding in that case, but not as remarkable as 0.8 seconds in Figure 4-19 for Bangkok ground.

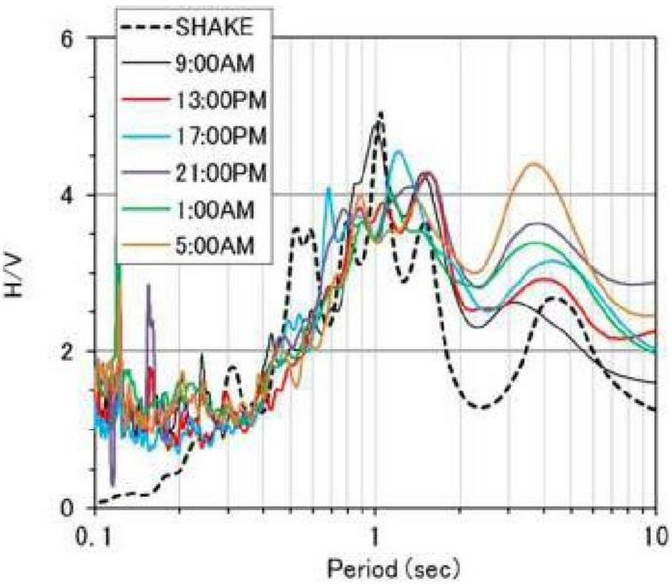


Figure 4-18 SHAKE transfer function (h=5%) and tremor H/V of Yangon ground [22)]

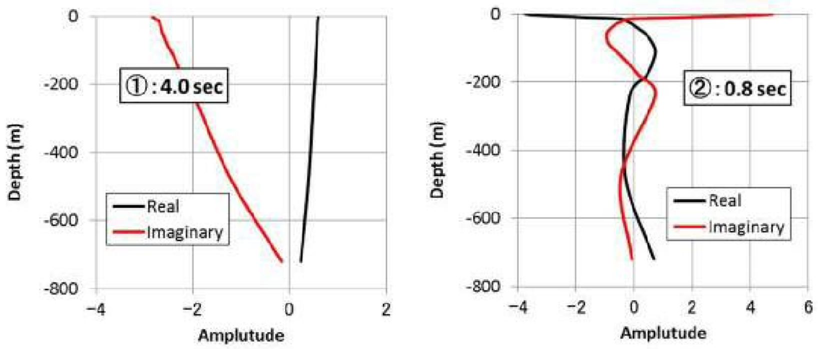


Figure 4-19 Shear deformation mode of Bangkok ground

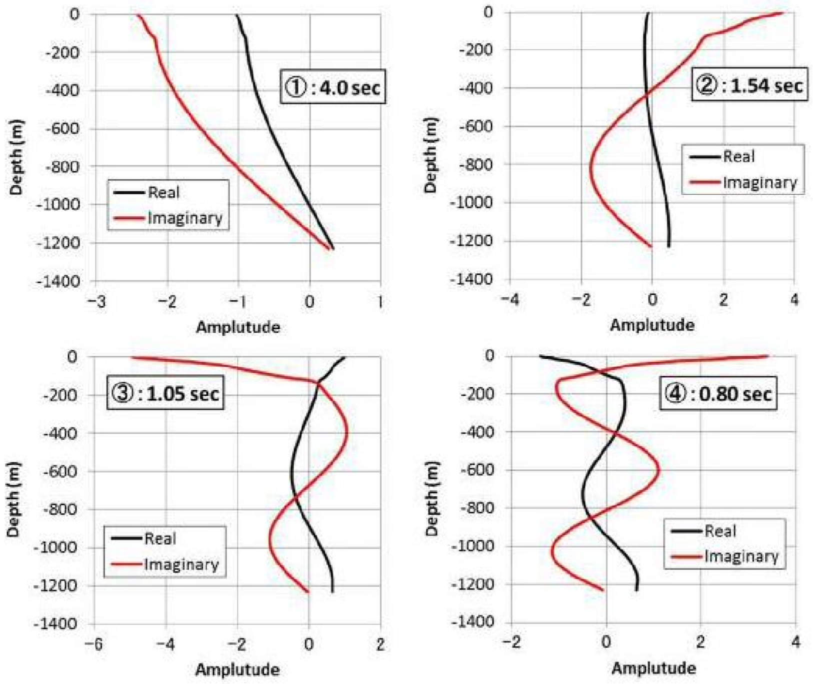


Figure 4-20 1st to 4th Shear deformation mode (h=5%) of Yangon ground [22)]

Currently, there is no data or information on the deep underground structure in Yangon. For this reason, in this research, microtremor measurement focusing on long-period tremor (pulsation) was performed and estimated an initial model as a first step of deep underground structure examination in Yangon. However, the reliability of the model seems to be insufficient. Also, in the observed H / V spectrum, there is only one peak near 1 second in the dry season, while in addition to the long-period peaks of 3 to 5 seconds in the rainy season, the causes of several peaks occurring in 0.8 to 1.5 seconds have not been elucidated. From now on, it is necessary to carry out array observation reflecting the structure of deeper part, to examine the validity of the underground structure model by this observation record, and to estimate the underground structure with higher accuracy.

4.3 Soil Samples and Sampling Method

The data used for this study is selected from data collated from laboratory tests that the Yangon Branch of Fukken Co., Ltd. [9] carried out in Yangon between 2010 and 2016, and in Hochiminh City between 2008 and 2015. Samples were taken with samplers (Fixed piston sampler with hydraulic pressure type) according to the standards, JGS 1221 “Method for obtaining soil samples using thin-walled tube sampler with fixed piston”, set by the Japanese Geotechnical Society (JGS, 2004 [24]). Therefore, there are no differences between the qualities of samples from Yangon, Hochiminh, and Japan. The number of clay samples from each sub-area used for this study are shown in Figure 4-22. All samples in Figure 4-22 were taken from the AC-I and AC-II layers (soft to firm clay layers), as illustrated in the soil profiles (Figure 4-9 to Figure 4-11), and were used in tests for physical properties (density test for soil particle, moisture content test, sieve and hydrometer tests, liquid limit and plastic limit tests) and mechanical properties (unconfined compression test, standard consolidation test). The thickness of the soft to firm clay layers (AC-I and AC-II layer) for each sub-area is shown in Table 4-4.

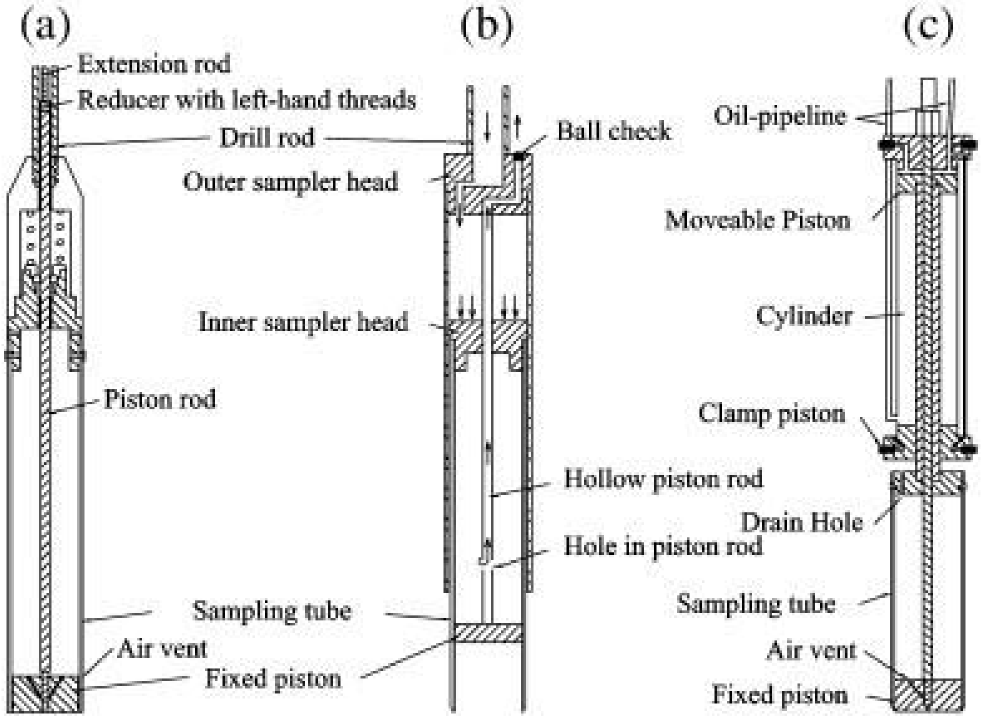


Figure 4-21 Fixed Piston Sampler (Hydraulic Pressure Type) [24]

Table 4-4 Thickness of Soft to Firm Clays at each Sub-area in Yangon

Sub-area	Thickness of Soft (AC-I) to firm (AC-II) clay layers		
	Soft clay layer (AC-I) (m)	Firm clay layer (AC-II) (m)	AC-I + AC-II (m)
a. North West*	(5 ~ 15)	(5 ~ 20)	(10 ~ 25)
b. North East**	4 ~ 11	5 ~ 31	14 ~ 36
c. Central***	2 ~ 10	2 ~ 10	2 ~ 10
d. Western*	9 ~ 20	2 ~ 13	2 ~ 26
e. Eastern**	5 ~ 17	0 ~ 5	5 ~ 20
f. Downtown*	4 ~ 16	1 ~ 8	4 ~ 24
g. Thanlyin*	17 ~ 22	3 ~ 8	21 ~ 27

*Belong to Irrawaddy Delta Sub-basin, **Belong to Pegu-Yoma Sittaung Basin, ***Belong to neither of them
(Numbers in ( ) shows they are estimated from other data not from soil profile sections.)

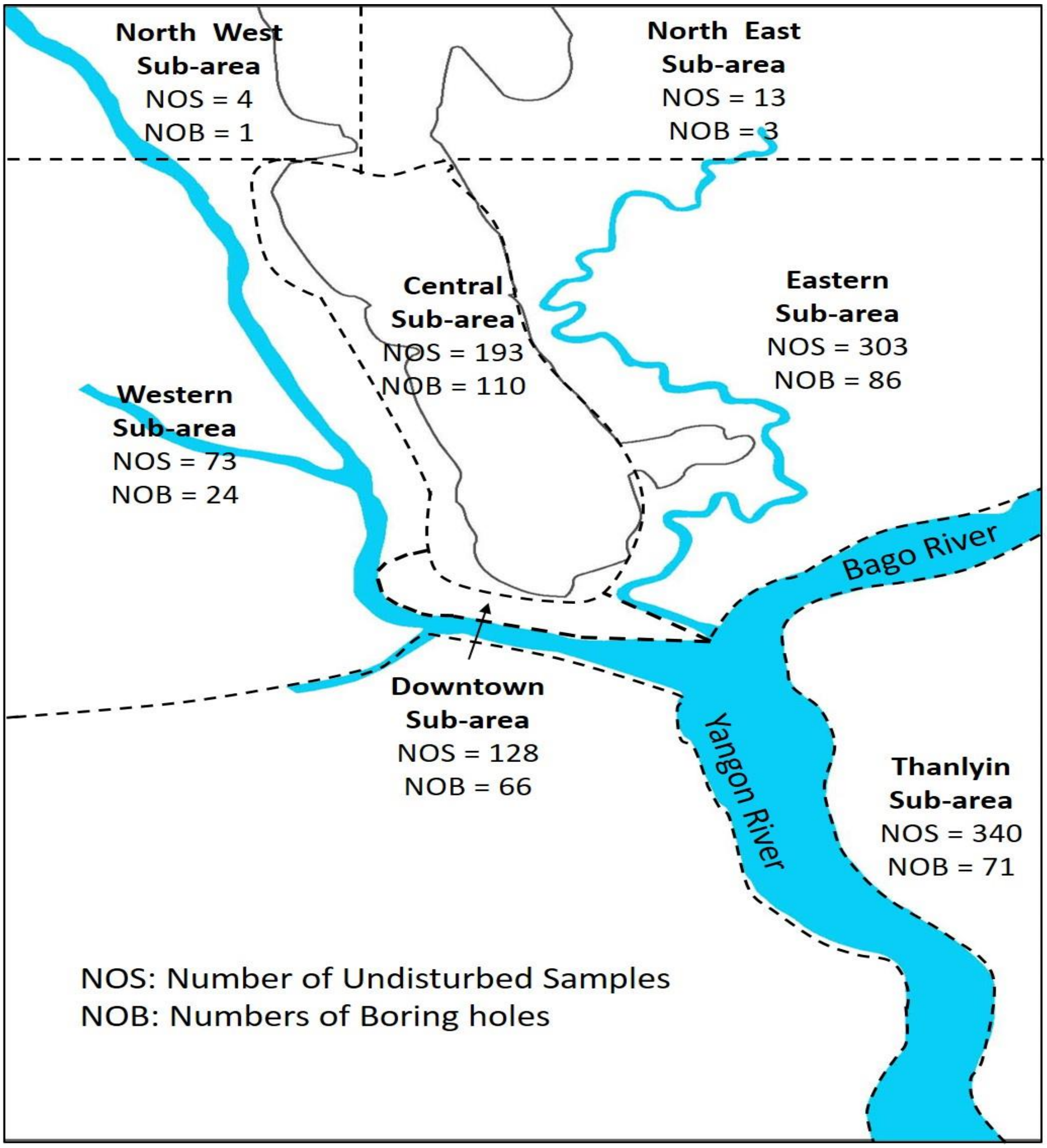


Figure 4-22 Number of samples and Borings at each sub-area used in this study

As for the physical properties of clays in Yangon, Density of soil particles test (JIS A1202:2009), Natural moisture content test (JIS A1203:2009), Grain size distribution test (JIS A1204:2009), Liquid limit and Plastic limit tests (JIS A1205:2009) and Bulk Density/ Unit Weight (JIS A1225:2009) test were carried out.

And also, as for the mechanical properties of clays in Yangon, Unconfined compression test (JIS A1216:2009), One-dimensional Consolidation test (JIS A1217:2009) and Triaxial compression (CIU) test (JGS 0523-2009) were performed. To carry out these tests, the appropriate calibrated testing apparatus were used as shown in Figure 4-23 ~ Figure 4-26.

Figure 4-23 Unconfined Compression Test Machine

Figure 4-24 Consolidation Test Apparatus

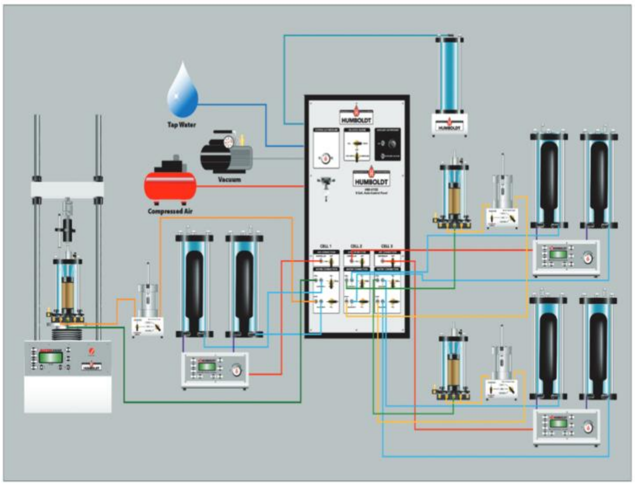


Figure 4-25 Schematic Diagram of Triaxial Compression Test Machine

Figure 4-26 Triaxial Compression Test Machine

4.4 A Comparison of the Physical Properties of Soft to Firm Clays between Sedimentary Basins, Sub-areas in Yangon and other Clays from other Countries

Soft to firm clays (where the SPT N-value is no more than 8) in the Yangon area are distributed at depths shallower than around GL-25m. Therefore, based on the data from undisturbed samples, and an N-value of less than or equal to 8, physical properties are compared between the sub-areas in Yangon and clays from other countries.

In this chapter, the soil data of different sub-areas were plotted with different colors. The red-colored plots show the data from sub-areas in Irrawaddy Delta Sub-basin, which is considered to include the North West, Western, Downtown, and Thanlyin sub-areas. The blue-colored plot shows the data for the sub-areas in the Pegu-Yoma Sittaung Basin, which is assumed to include the North east and Eastern Sub-areas. The data in the Central sub-area, which are shown with black colored plots in the graphs, do not belong to either sedimentary basins. As for the soil properties in the sub-areas in Yangon, significant differences between the sub-areas were not found except for in the Central Sub-area as mentioned below.

4.4.1 A comparison of the density of soil particles, clay content and clay activity among the Sub-areas in Yangon and between two basins

Figure 4-27 shows density of soil particle distribution with depth. This ranges between 2.6 g/cm^3 and 2.8 g/cm^3, with an average value of about 2.7 g/cm^3. This means that most of the clays in the Yangon area are inorganic clays. Significant differences between the two sedimentary basins are not apparent from the figure (Blue plots are the data from the Irrawaddy Delta Sub-basin and red plots are the data from the Pegu-Yoma Sittaung Basin). Significant differences between soil particle densities among the seven Sub-areas are also not seen.

Figure 4-28 shows the range and average clay content (particles of size less than 5 μm) for each sub-area in Yangon. They are distributed over a wide range of between 5 % and 90 %, with averages between 30 % and 50 %. As shown in the figure, little difference is seen between the clay contents in the sub-areas of Yangon, or between the two different basins and the Central Sub-area of Yangon.

The ratio of the Plasticity Index to content in clay particles of less than 2 μm (%) is termed the activity *(A)* of the soil with the following definition (Skempton, 1953 [34)], Asakawa, 1972 [28)]),

$A = I_P$ (Plasticity Index of soil)/ Content of clay particles less than 2 μm (%). (1)

It is known that the value of A is reflected by the water retention capacity of clay minerals and its surface activation and range is related to the clay minerals in soil as follows (Skempton, 1953 [34)], Asakawa, 1972 [28)]),

$$\left.\begin{array}{ll} \text{Active clays (ex. Smectite clay) :} & A \geq 1.25 \\ \text{Normal clays (ex. Illite clay):} & 0.75 < A < 1.25 \\ \text{Inactive clays (ex. Kaolinite clay):} & 0.75 \geq A \end{array}\right\} \quad (2)$$

In this study, using a particle clay content (%) of less than 5μm instead of 2μm, the activity of clay is defined as A', because all hydrometer tests were carried out based on the JIS procedure in which clay particle size is defined as having a grain size of less than 5μm.

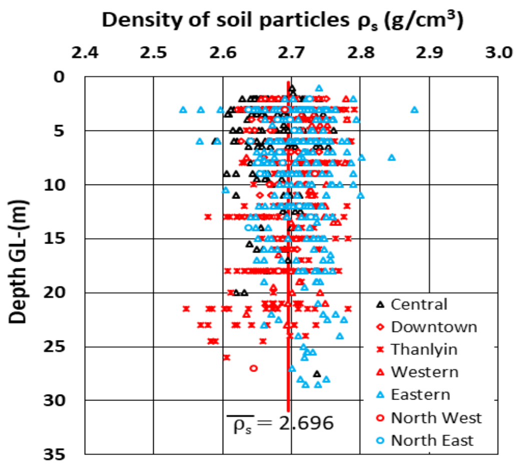


Figure 4-27 Density of Soil Particles with the depth

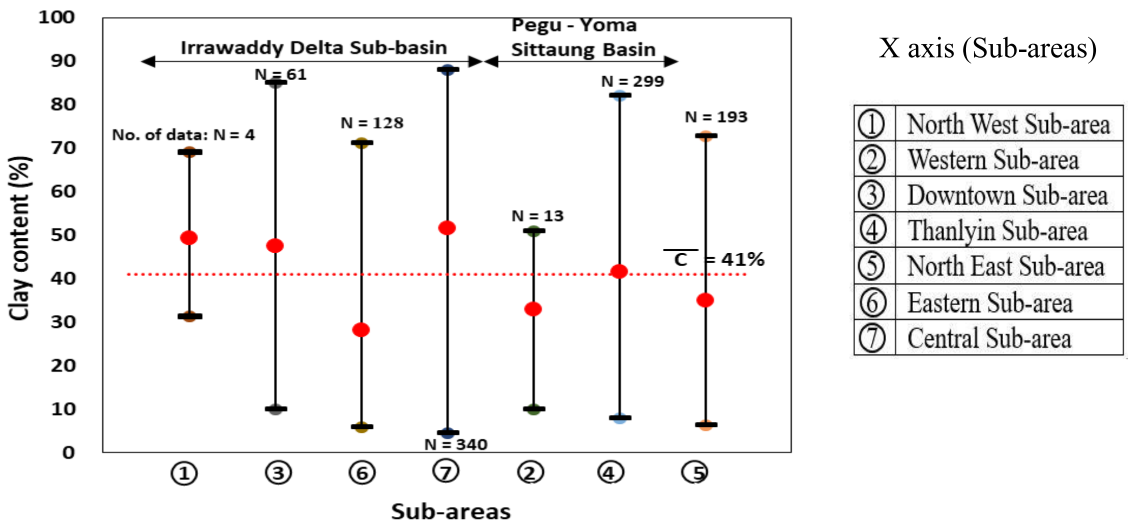


Figure 4-28 Clay Content C at the each sub-area

Figure 4-29 shows the histogram of clay activity (*A′*) for the two basins and the Central sub-area of Yangon. The distribution of *A′* for the Irrawaddy Delta Sub-Basin and the Pegu-Yoma Sittaung Basin is similar, as 55 % of the samples have $A' \leq 0.75$, 30 % is $0.75 < A' < 1.25$ and 15 % are $1.25 \leq A'$. This means that 55 % of the samples of both basins except for the Central sub-area are mainly composed of inactive clay (ex. Kaolinite clay) and 30% are of normal clay (ex. Illite clay). However, in the Central sub-area, 40 % of samples have $A' \leq 0.75$, 30 % are $0.75 < A' < 1.25$ and 30 % are $1.25 \leq A'$. The percentage of active clay (ex. Smectite clay) is a little higher than found in the soil from the other two basins. As shown in Figure 4-29, the average value of *A′* for clays in Japanese port and harbor area (Ogawa and Matsumoto, 1978 [13)]) is 1.11 (Standard deviation: 0.53), and the *A′* for clays in whole of Myanmar (Murakami et al., 2015 [10)]) is 1.14 (Standard deviation: 0.97). Compared to these values, the *A′* for clays in Yangon (average of whole Yangon area) is 0.91 (Standard deviation: 0.59), meaning that the activity of clays in Yangon is less than clays in Japan and the whole of Myanmar.

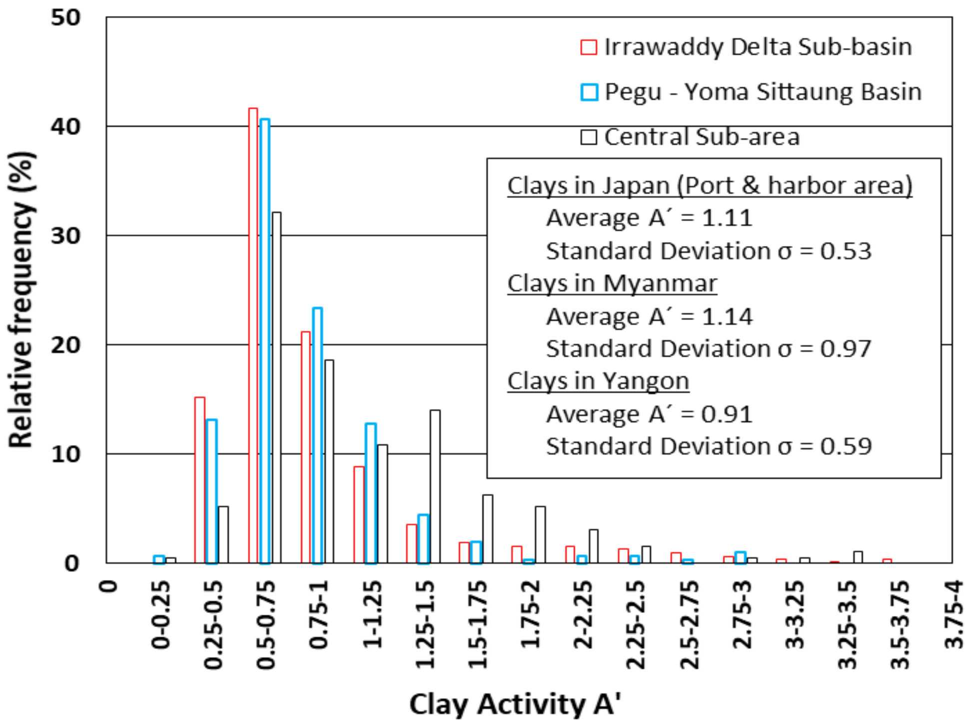


Figure 4-29 Histogram for clay activity *A′* for clays in Yangon

4.4.2 A comparison of the relationship between physical properties and Atterberg Limits among Sub-areas in Yangon, between two basins, and clays from other countries

Figure 4-30 is the plasticity chart that shows the relationship between the plasticity index (I_P) and the liquid limit (w_L) for the seven sub-areas in Yangon. Most of the data are plotted on the upper side of the A-Line with about 10% of variation in width along the A-Line. The liquid limits are mainly plotted within a range between 20% and 90%, and the plasticity index range from 10 to 60. Significant differences between the two sedimentary basins, the Irrawaddy Delta Sub-basin (plotted in blue) and the Pegu-Yoma Sittaung Basin (plotted in red) were not found. Significant differences among plots of the seven Sub-areas are also not apparent. The trend line for this relationship between the clays in Yangon is similar to that for clays from Japanese port and harbor areas [13)], however the trend line for clays in Hochiminh is almost located at the bottom boundary of the plots for the clays in Yangon.

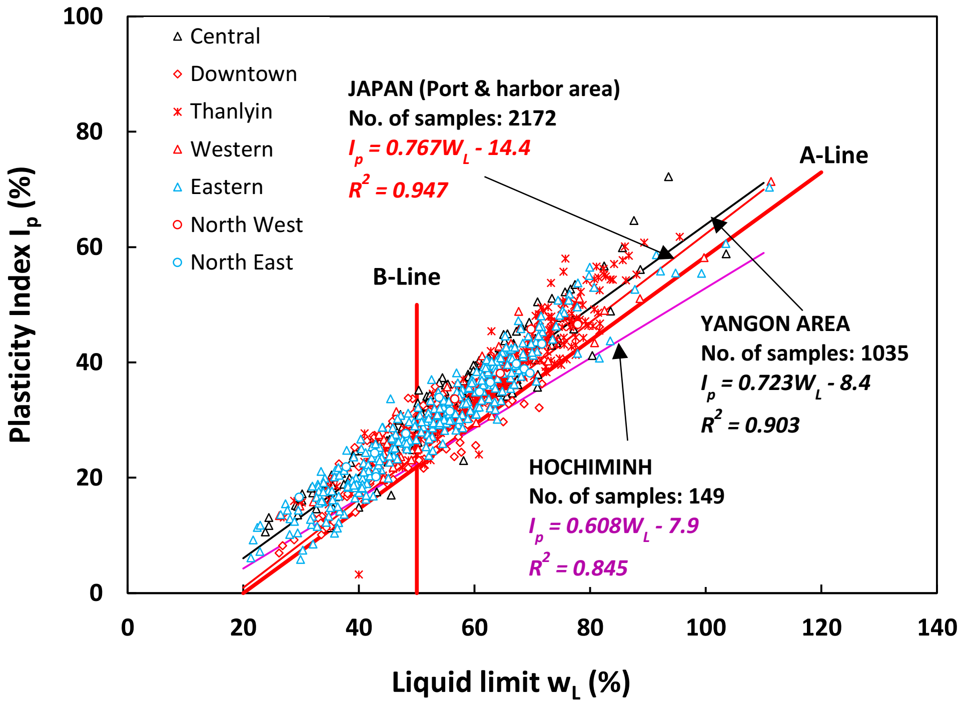


Figure 4-30 Plasticity Chart for clays in Yangon

Figure 4-31 shows the relationship between unit weight (γ_t) and natural moisture content (w_n). In the clays in Japanese port and harbor areas, the unit weight is distributed widely from 13 kN/m^3 to 19 kN/m^3. However, Yangon clay has a distribution between 15 kN/m^3 and

21 kN/m^3. The natural moisture content for clays in Yangon is mostly less than 60% and the average unit weight is about 18 kN/m^3, which is much larger than the clays in Japanese port and harbor areas [13)]. This higher value is affected by the highly over-consolidated state of the clays, such as inorganic clays above the water table and lateritic clays distributed at shallow depths, ground surface to G.L.-5 m ~ -6 m.

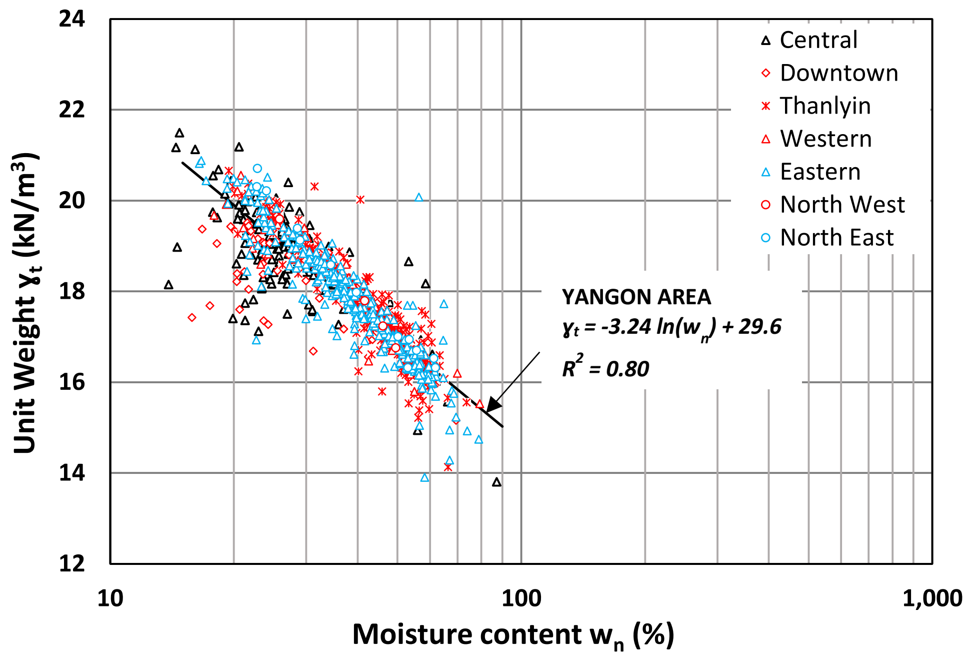


Figure 4-31 Relationship between the unit weight γ_t and the moisture content w_n

The relationship between natural moisture content (w_n) and liquid limit (w_L) is shown in Figure 4-32. From this figure, it is apparent that there is a data group where the moisture content (w_n) fell in a narrow range of 20 % to 30%, even though the liquid limit ranges widely between 20 % and 90 %. This data group mostly belongs to the Central Sub-area in Yangon.

The lines shown in the figure are regression lines obtained from the data from other countries in past studies, given as follows,

w_n = 0.69 (Yangon clays, present study) (3)

w_n = 0.62 (Whole Myanmar clays, Murakami et al.,2015 10)) (4)

w_n = 0.92 (Clays in Japanese port and harbor areas, Ogawa & Matsumoto,1978 13)) (5)

w_n = 0.96 (Hochiminh clays, present study) (6)

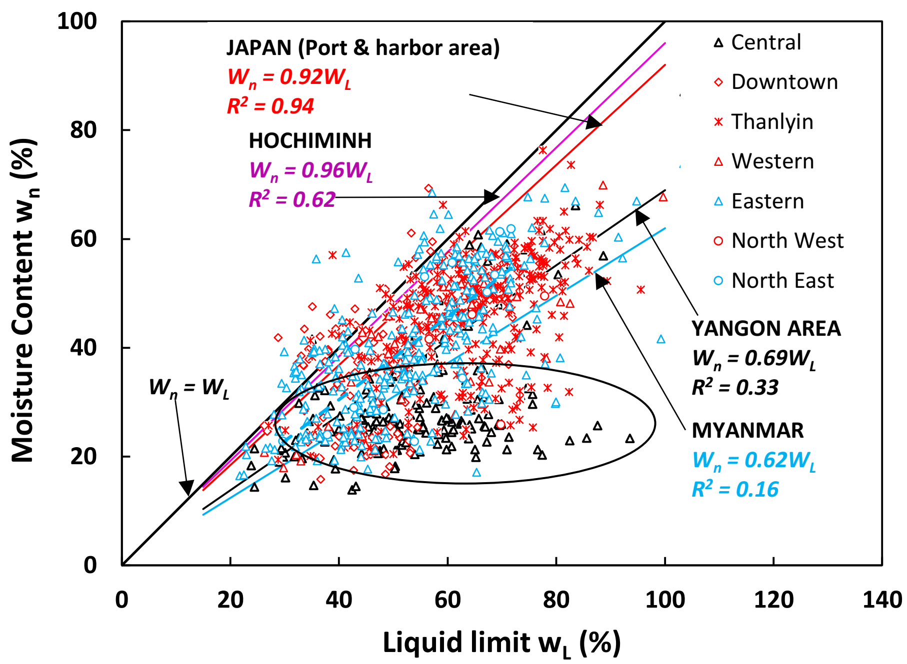


Figure 4-32 Relationship between the moisture content w_n and the liquid limit w_L

The regression line for Yangon clay was close to the regression line of the whole Myanmar clay [10)], which is located at a lower position than clays from Hochiminh and Japan (port and harbor areas [13)]). In the case of clays from Japan (port and harbor areas), a significant number occur with a natural moisture content above the liquid limit, however as for almost all clays in Yangon area, the natural moisture contents are less than the liquid limits; in particular, the data from the Central sub-area shows the lowest moisture content with about 20 % of the rest of the Yangon area. It is thought that one of the reasons for the low water content in the Central sub-area is over-consolidation or different sedimentary conditions as compared to other sub-areas in Yangon.

Except for the Central Sub-area, the distribution of data is similar among the sub-areas of Yangon and between the two sedimentary basins.

4.5 A Comparison of the Mechanical Properties of Soft to Firm Clays between the Sub-areas in Yangon and Clays from other Countries

The mechanical properties of clays are very important characteristics for the design of countermeasures for stability and the ground settlement which are common engineering problems during the construction of infrastructure. In this chapter, shear strength, compression, and consolidation characteristics are studied and the properties of clays in the two basins, and the sub-areas of Yangon and the clay from other countries are compared.

4.5.1 Unconfined Compressive Strength (q_u) and Shear Strength Ratio (c_u/p')

The distribution of unconfined compressive strength (q_u) with depth (G.L.-) is shown in Figure 4-33. Very large variations are apparent within the plots of all the sub-areas, and also within the plots of the Irrawaddy Delta Sub-basin (plotted in blue), the Pegu-Yoma Sittaung Basin (plotted in red) and the Central sub-area in Yangon. A trend suggesting that q_u increases with depth can be seen even with very wide variations.

In Figure 4-33, the following trend line is found, based on the data from Yangon, except for the data which are from clays found at depths shallower than GL- 6 meters, as follows.

$$q_u = 23.9 + 5.2\,z \quad (z = \text{G.L. - :m})\ (R^2 = 0.25) \qquad \text{(Yangon clay)} \qquad (7)$$

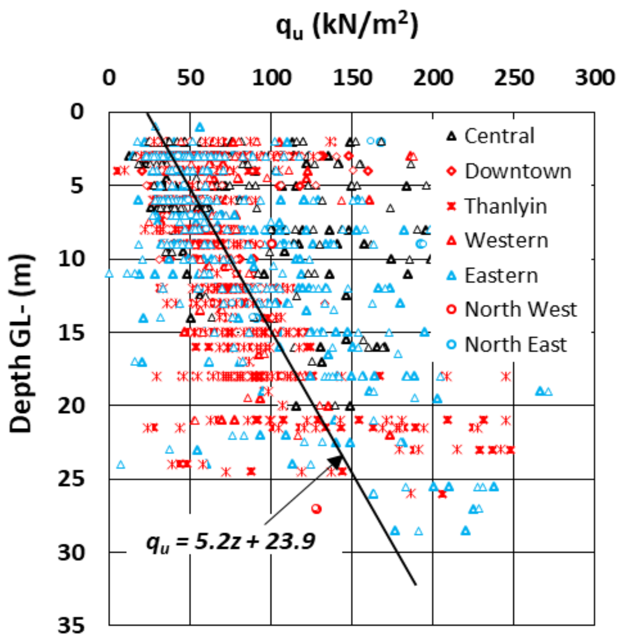


Figure 4-33 Unconfined compressive strength q_u with the depth for clays in Yangon

In the range of depth between G.L.± 0 m and G.L.- 6 m, it is considered that the large values of q_u from 100 (kN/m^2) to 250 (kN/m^2) are due to the effect of over-consolidation caused by repeated conditions from dry and wet periods, in clays such as the inorganic clay found above the ground water level and lateritic clay.

Figure 4-34 shows the histogram for failure strains of unconfined compression tests. It is generally suggested that a sample with a failure strain of less than 6% is undisturbed. As shown in the figure, more than 60 to 70 % of samples showed failure strains of less than 6%, which means that most of samples used for this study can be presumed to be undisturbed. According to the histogram, considerable differences cannot be found in the distribution of relative frequency between the data for the Irrawaddy Delta Sub-basin (colored with red), the data for the Pegu-Yoma Sittaung Basin (colored with blue), and the Central Sub-area (colored with black).

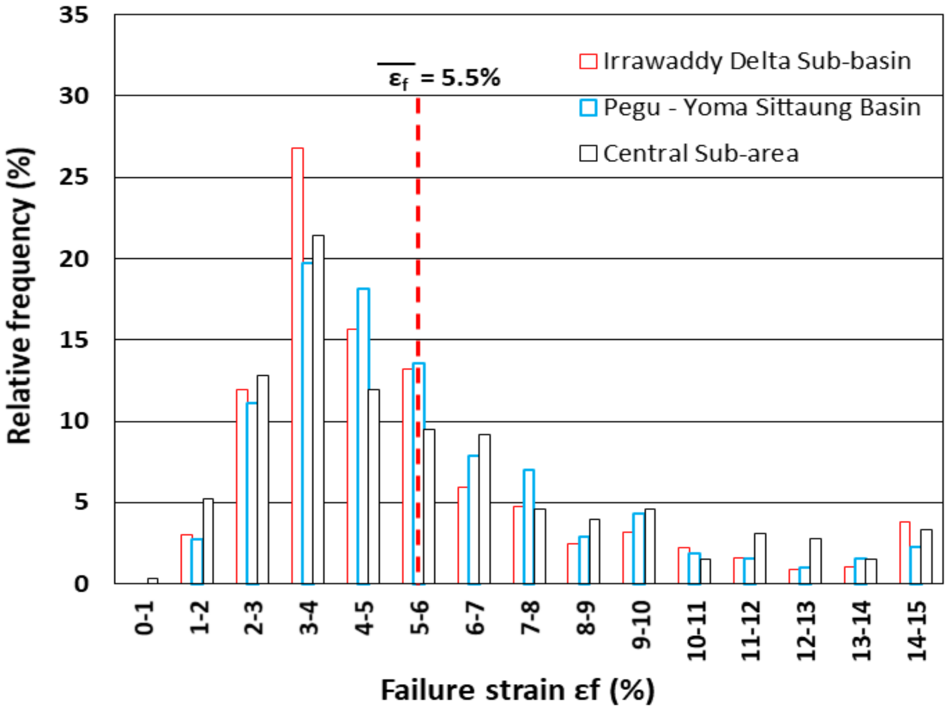


Figure 4-34 Histogram for failure strain ε_f of unconfined compression test

Figure 4-35 shows the over-consolidation ratio (OCR) with depth (G.L.-) for clays in the seven sub-areas in Yangon. As shown in the figure, except for the data from between G.L. ± 0 m and G.L. - 5 ~ - 6 m, for which the OCR is anomalously large, the average OCR with depth is approximately 1.6 and is constant down to G.L. -25 m. The fact that OCRs for the upper portions shallower than G.L. - 5 ~ - 6 m show very large values is strongly related with the fact that high unconfined compressive strengths are seen at shallower depths. Clays from deeper than G.L.-10 m, tend to have a lesser value for OCR in the Irrawaddy Delta sub-basin (plots colored with red) (softer) than clays in the Pegu-Yoma Sittaung Basin (plots colored with blue).

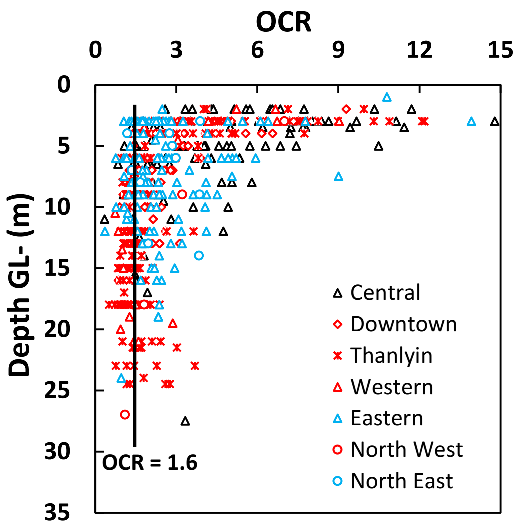


Figure 4-35 Over-consolidation ratio (OCR) with the depth for clays in Yangon

Figure 4-36 and Figure 4-37 show correlations between the effective over-burden pressure σ_{v0}' and the consolidation yield stress p_c for the sub-areas in Yangon. As shown in the figure, the gradients of trend lines indicate an average OCR (Over-consolidation Ratio = p_c / σ_{v0}') of between 2.2 and 2.8 for sub-areas except for the Central Sub-area (average OCR = 4.4) and the northern sub-areas (North East and North West Sub-areas), from where the data is sparse. A significant difference in OCRs between the sub-areas was not found, except for in the Central Sub-area. However, some OCR trend lines in some of the sub-areas such as Thanlyin and the Eastern Sub-areas are affected with an OCR for firm to stiff clays which are distributed near ground surface at shallower depths than GL- 5 to -6 meters. Therefore, the OCRs for clays from the whole Yangon area ranges between 1.0 and 3.0, if the data of firm to stiff clays at shallow depths near the ground surface are excluded. As shown in figure (f) of Figure 4-37, due to effect of these high p_c at shallow depths, the average OCR of the whole Yangon area averages at about 2.96, which is a greater value than that of Japan (port and harbor areas) and the whole of Myanmar, with OCR = 1.25 and 1.62, respectively.

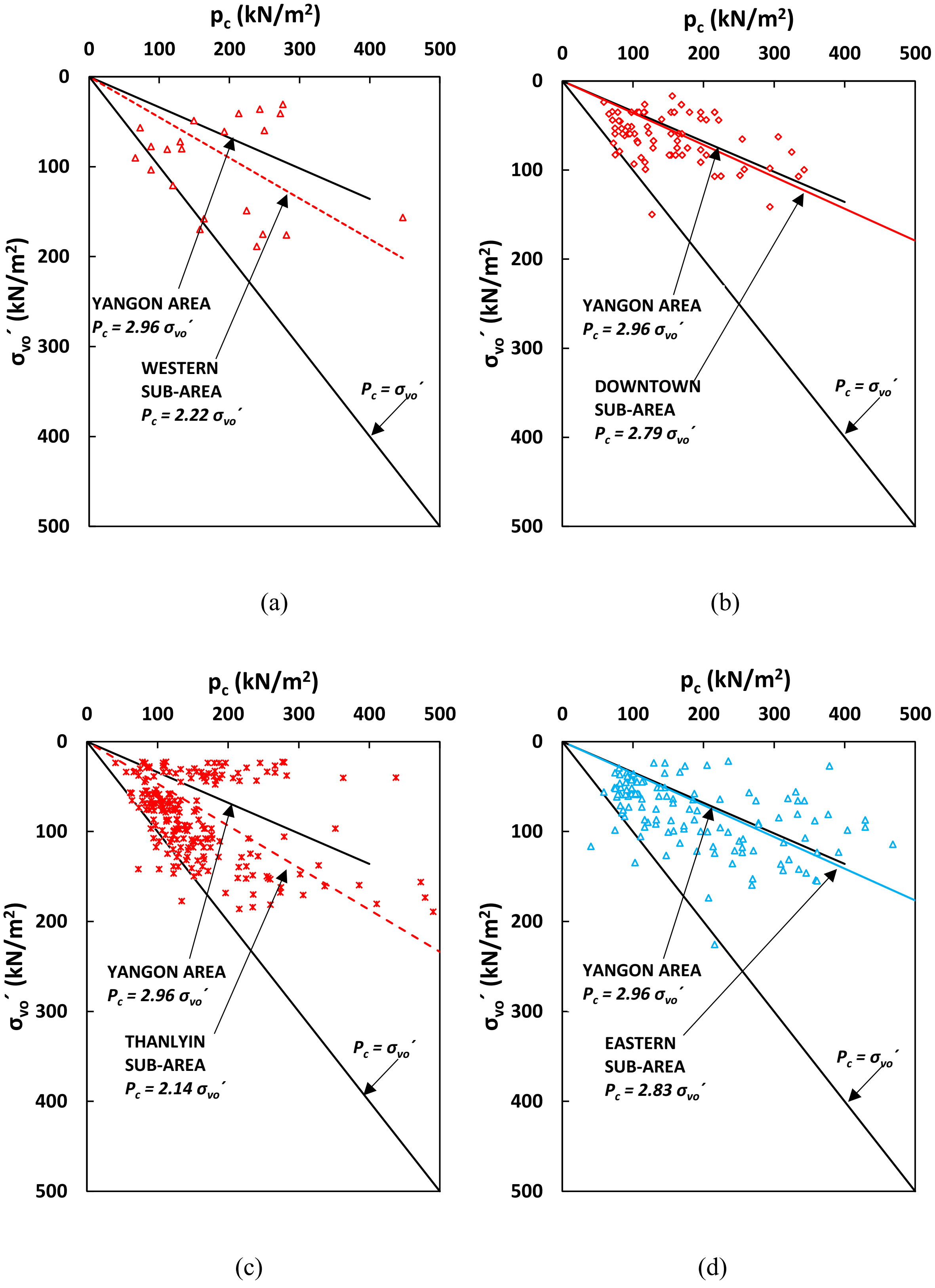
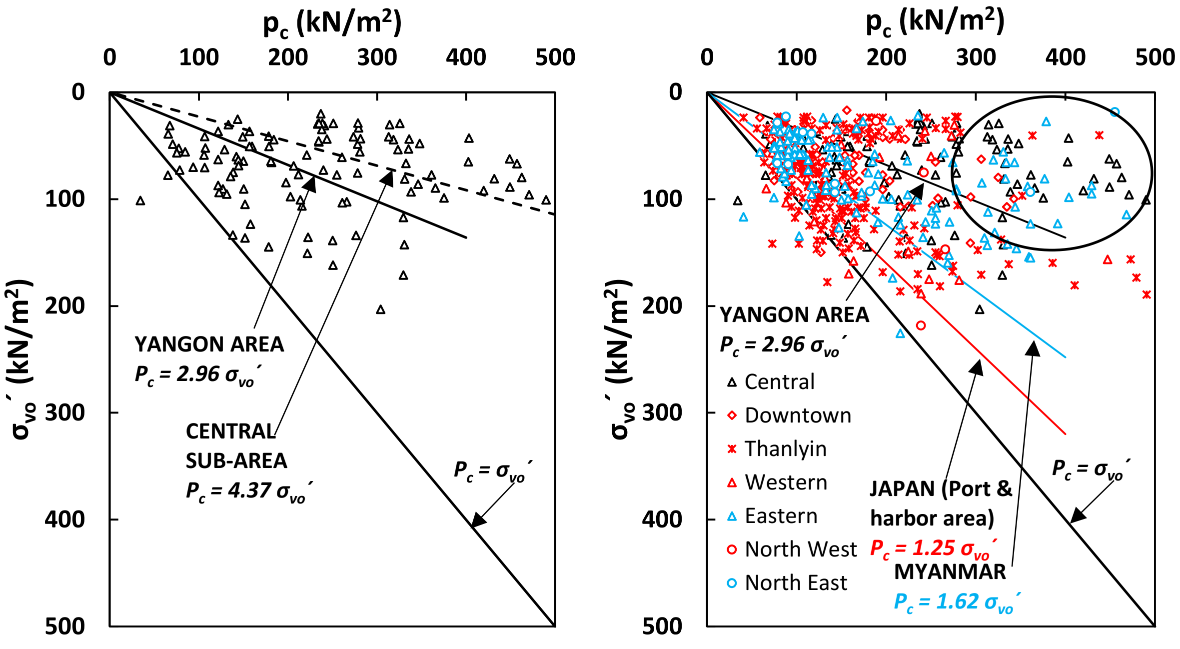


Figure 4-36 Correlations between the effective overburden pressure σ_{v0}' and the consolidation yield stress p_c for sub-areas in Yangon (a) ~ (d)



(e) (f)

Figure 4-37 Correlations between the effective overburden pressure σ_{v0}' and the consolidation yield stress p_c for sub-areas in Yangon (e) ~ (f)

Figure 4-38 shows the relationship between q_u and the consolidation yield stress (p_c) for clays in Yangon, the whole of Myanmar, Hochiminh and the Japanese port and harbor area [13]. As shown in the figure, a group composed of lateritic clays belonging to the Pegu-Yoma Sittaung Basin and the Central Sub-area, and another of inorganic clays at a depth of G.L. - 5, -6 m above the groundwater table, have large p_c values of more than 300 kN/m^2. In this basin and sub-area, even though the area is of soft to firm clay, it includes is yellowish brown or reddish-brown lateritic clay, which has repeatedly suffered dry and wet conditions for a long time (Sueoka, 1990 [2]).

In the associate figures, the considerable differences in the q_u - p_c relationship is seen among the clays from different countries and areas. The regression lines for each country and each area are given as follows,

$q_u = 0.369\, p_c$ (Yangon clays, present study) (8)

$q_u = 0.339\, p_c$ (Whole Myanmar clays, Murakami et al.,2015 [10]) (9)

$q_u = 0.598\, p_c$ (Clays in Japanese port and harbor areas, Ogawa & Matsumoto, 1978 [13]) (10)

$q_u = 0.617\, p_c$ (Hochiminh clays, present study) (11)

As shown in the above equations, compared to clays from the whole of Myanmar (Murakami et al., 2015 [10]), the clays in Yangon have almost the same relationship between q_u and p_c. However, clays from Hochiminh and the Japanese clays (port and harbor areas) have a ratio q_u/p_c of about 1.6 times larger than that in Yangon. This means that clays in Yangon have a smaller normalized shear strength (c_u/p_c) than clays in Hochiminh and Japan (port and harbor areas [13]). From the above equations and $q_u = 2c_u$, the normalized shear strength (= c_u/p_c, here not c_u /p', p': effective consolidation pressure) can be calculated for clays in the whole of Myanmar, Japan (port and harbor area), Hochiminh, and Yangon as 0.17, 0.30, 0.31 and 0.18, respectively. The normalized shear strength of clays in Yangon is about 60% of that of the Hochiminh clays and Japanese clays (port and harbor areas [13]). The strength reduction by swelling due to over-consolidation effect and the strength reduction due to disturbances during sampling and testing procedures might be ones of the reasons for having such a small normalized shear strength.

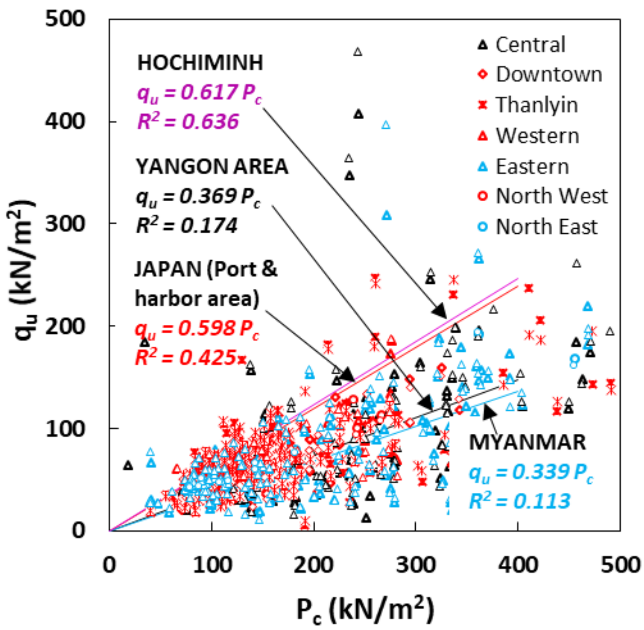


Figure 4-38 Relationship between q_u and p_c for clays in Yangon

With regards to the relationship between shear strength ratio (c_u/p') and the index properties of clays, the following equation is proposed for normally consolidated clay by Skempton (1954 [35], 1957 [36]),

$$c_u/p' = 0.11 + 0.0037\, I_P \qquad (12)$$

Figure 4-39 shows the relationship between (c_u/p') and plasticity index I_P. As shown in Figure 4-39, the plasticity index (I_P) of Yangon Clay ranges from between 10 and 60, and the shear strength ratios (c_u/p') are plotted with large variations between 0.05 and 0.4. The clear linear relationship between c_u/p' and the plasticity index I_P as in equation (12) by Skempton, 1954 [35)] is not found in clays from Yangon.

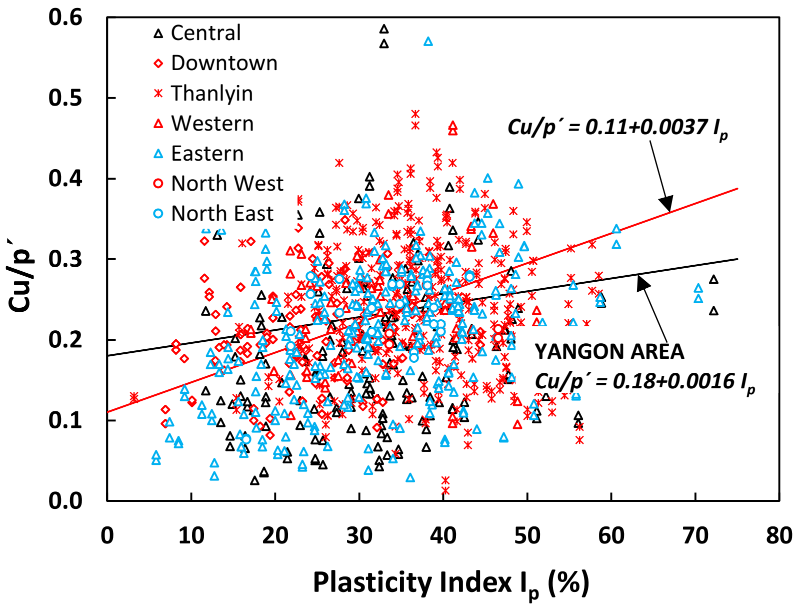


Figure 4-39 Relationship between c_u/p' and I_P for clays in Yangon

4.5.2 Compression Index, C_c

Figure 4-40 to Figure 4-43 show the correlation between compression Index C_c and the natural moisture content w_n for the sub-areas in Yangon. As shown in the figure, good correlations can be found between C_c and w_n for the sub-areas, and the determination factor R^2 for the sub-areas ranges between 0.5 and 0.7, except for in the data from the North East and the North West Sub-areas, where the data is limited. Significant differences in the tendencies for correlation among the sub-areas cannot be found. The regression line between C_c and w_n for the whole Yangon area is given as follows,

$$C_c = 0.0124\, w_n - 0.072 \qquad (R^2 = 0.27) \qquad (13)$$

Table 4-5 Regression Lines between C_c and w_L proposed by Existing Researches

Area	Correlation Equation	Range of Data	Coefficient of Determination R^2	Source
Japan (Port and Harbor area)	$C_c = 0.015\ (w_L-19)$	$w_L = 33.5 - 194.0\%$	0.67	Ogawa et al [13],1978
South shore area of South Korea	$C_c = 0.012\ (w_L+16.4)$	$w_L = 28.4 - 120.2\%$	0.64	Yoon et al [25],2004
East shore area of South Korea	$C_c = 0.011\ (w_L-6.4)$	$w_L = 23.0 - 107.0\%$	0.64	Yoon et al [25],2004
West shore area of South Korea	$C_c = 0.010\ (w_L-10.9)$	$w_L = 24.5 - 77.9\%$	0.67	Yoon et al [25],2004
Egypt	$C_c = 0.0063\ (w_L-10.0)$	$w_L = 10.0 - 110.0\%$	-	Abdrabbo et al [26],1990
Brazil	$C_c = 0.0046\ (w_L-9.0)$	-	-	Cozzolino et al [29],1961
Greece and the USA	$C_c = 0.006\ (w_L-9.0)$	-	0.59	Azzous et al [27],1990
Myanmar (Whole area)	$C_c = 0.007\ (w_L+8.7)$	$w_L = 26.5 - 93.5\%$	0.25	Murakami et al [13],2015
Hochiminh	$C_c = 0.013\ (w_L+11)$	$w_L = 59.7 - 95.8\%$	0.27	Present Study
Yangon	$C_c = 0.007\ (w_L+2.3)$	$w_L = 21.0 - 95.5\%$	0.27	Present Study

A relatively good correlation was found between the compression index (C_c), the natural moisture content w_n, and the initial void ratio e_0. The median value of the distribution moisture content of Yangon clay is 40%, the C_c of Yangon clay is 5/9 of the clays in Japan (port and harbor area[13]), and 2/3 of the clays in Hochiminh. On the other hand, in the relationship between C_c and the initial void ratio e_0, the clays in Yangon show almost the same value as clays in Japan (port and harbor area) for the same initial void ratio, and clays in Hochiminh show about 0.2 larger C_c than clays in Yangon.

(a)

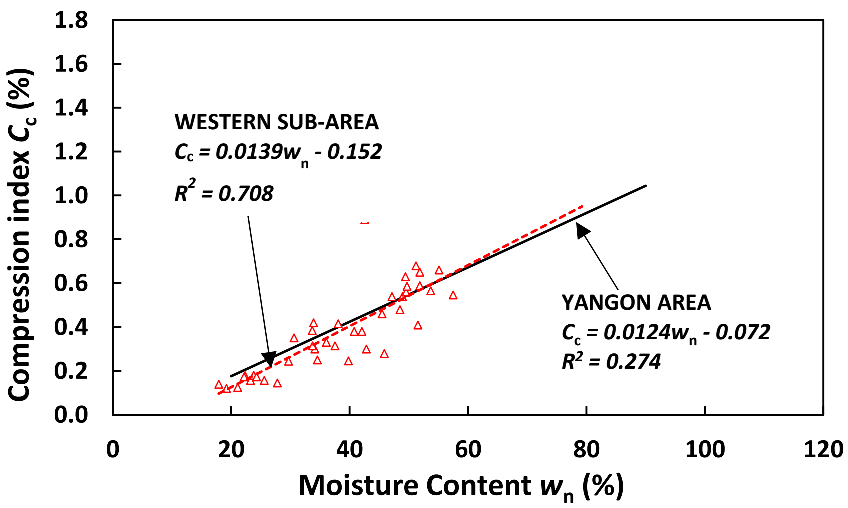


Figure 4-40 Correlations between Compression Index C_c and Moisture Content w_n for sub-areas in Yangon (a)

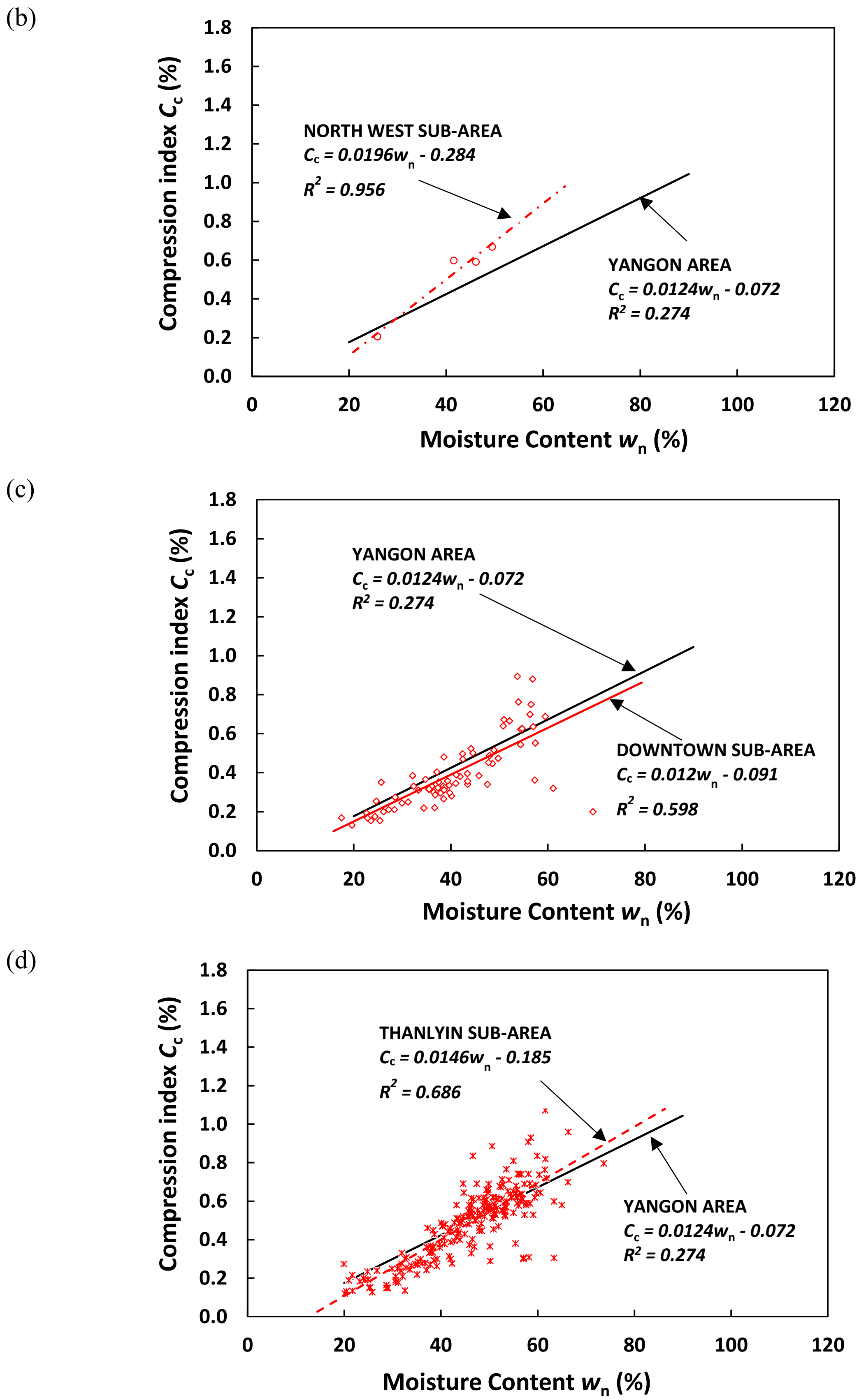
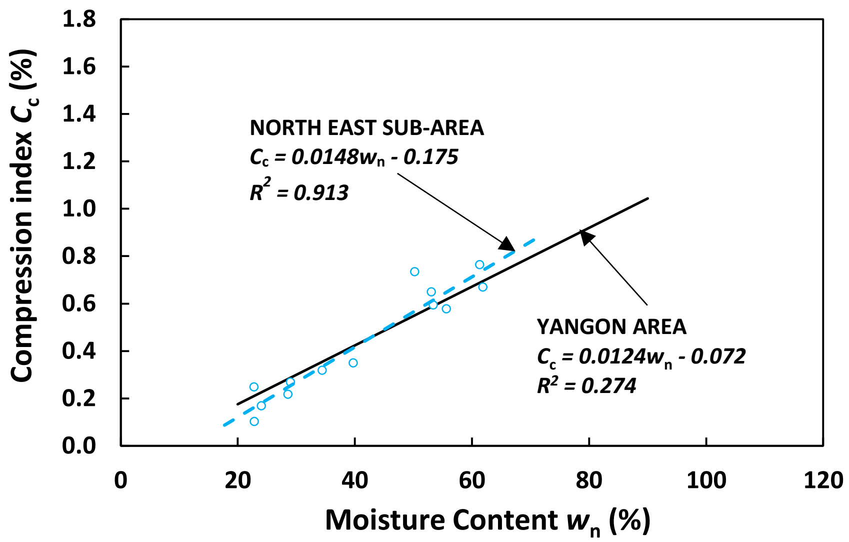
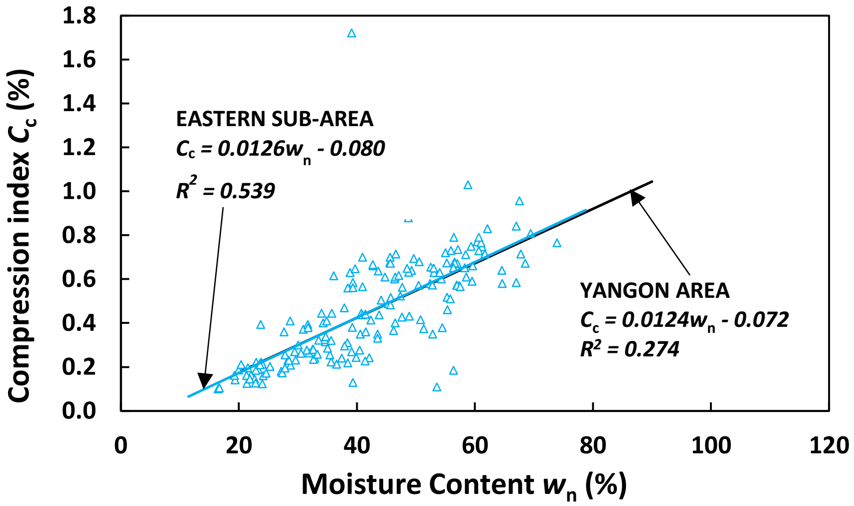


Figure 4-41 Correlations between Compression Index C_c and Moisture Content w_n for sub-areas in Yangon (b) ~ (d)

(e)



(f)



(g)

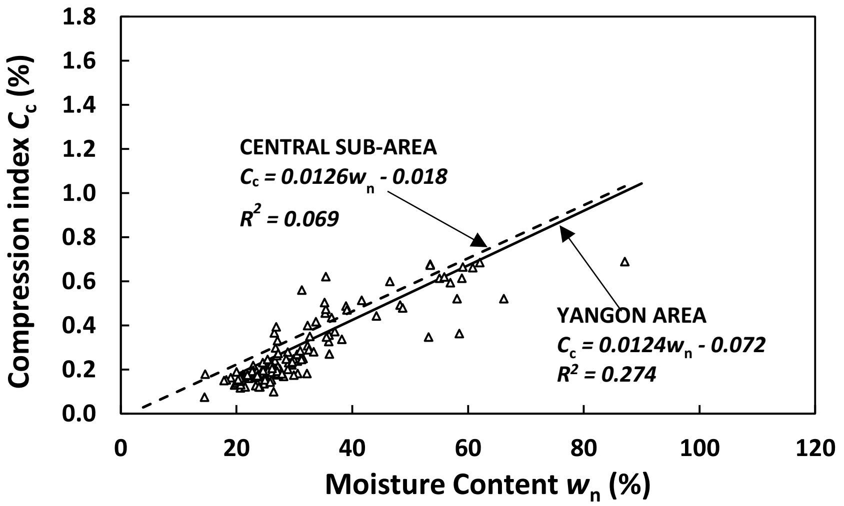


Figure 4-42 Correlations between Compression Index C_c and Moisture Content w_n for sub-areas in Yangon (e) ~ (g)

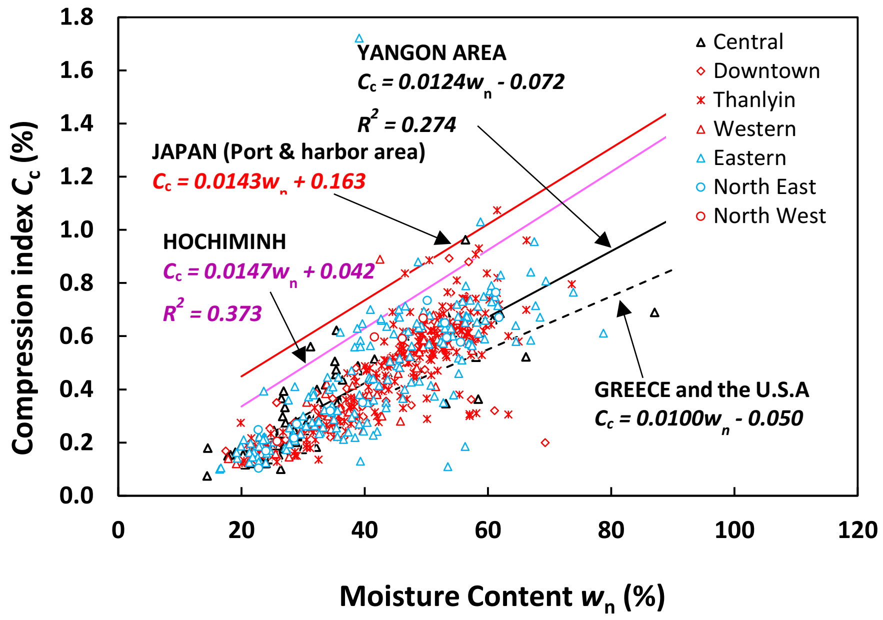


Figure 4-43 Correlations between Compression Index C_c and Moisture Content w_n for sub-areas in Yangon (h)

Figure 4-44 to Figure 4-46 show correlations between the compression Index C_c and the liquid limit w_L for the sub-areas in Yangon. As shown in the figure, good correlation between C_c and w_L were not found and the determination factor R^2 ranges between 0.2 and 0.4, except for the data for the North East and North West sub-areas. A significant difference in correlation tendency between each sub-area was not found, except for the Central sub-area. C_c data in the Central Sub-area has very low C_c value group which is about C_c = 0.2 against its whole liquid limit range.

The regression line between C_c and w_L for the whole Yangon area is shown as follows,

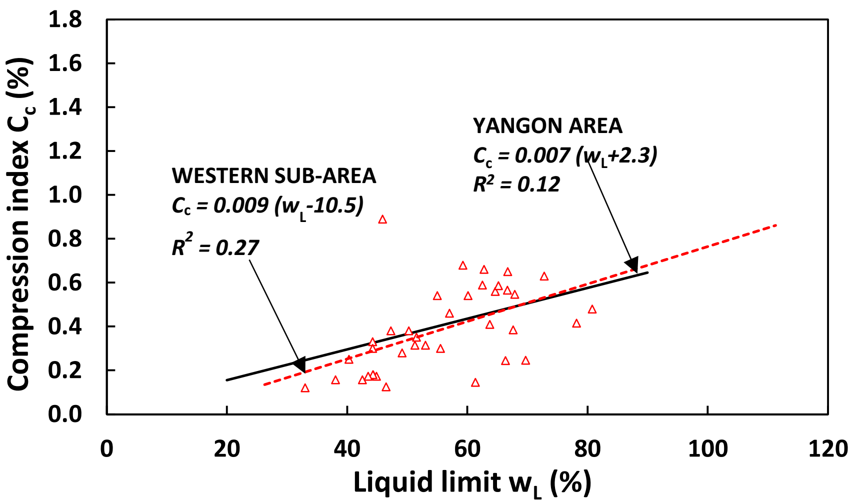
$$C_c = 0.007\,(w_L + 2.3), \qquad (R^2 = 0.12) \qquad (14)$$

Many studies have been carried out on the C_c - w_L relationship for the different clays in different countries. Those correlations are shown in Figure 4-44 to Figure 4-46 for the purpose of comparison. As shown in the figure, the clays in Yangon are not so much different from clays from the whole of Myanmar [10)], however the region shows different tendencies from clays in Hochiminh and Japan (port and harbor areas[13)]). It is thought that one of the reasons for these differences is due to a data group having a very low C_c of between 0.1 and 0.3 of the clays in Yangon.

The C_c - w_L correlation equations in the previous studies are summarized in Table 4-5 including the correlation equations for clays in Japanese port and harbor areas, Korea (Yoon et al., 2004 [25)]), Egypt (Abdrabbo and Mahmoud, 1990 [26)]), Brazil (Cozzolino, 1961 [29)]), Greece and USA (Azzouz et al., 1976 [27)]), Hochiminh and the whole of Myanmar (Murakami et al., 2015 [10)]). Among those regression lines, the largest compressibility to liquid limit is shown in the Hochiminh clay, after which the values become smaller in the following order, South Korea (South coastal area), Japan (port and harbor area), South Korea (East shore area), South Korea (West shore area), Myanmar, Egypt, Greece and the USA, and Brazil. The clays in Yangon and over the whole of Myanmar are located between South Korea (West shore area) and Egypt, and it can be said that the compressibility of clays in Yangon is average to relatively small from a global point of view. However, the trend line for the C_c - w_L correlation of Yangon clay is close to that for the equation for remolded clay,

$$C_c = 0.007\,(w_L - 7),\ \text{as proposed by Skempton (1944 [33)])} \qquad (15)$$

(a)



(b)

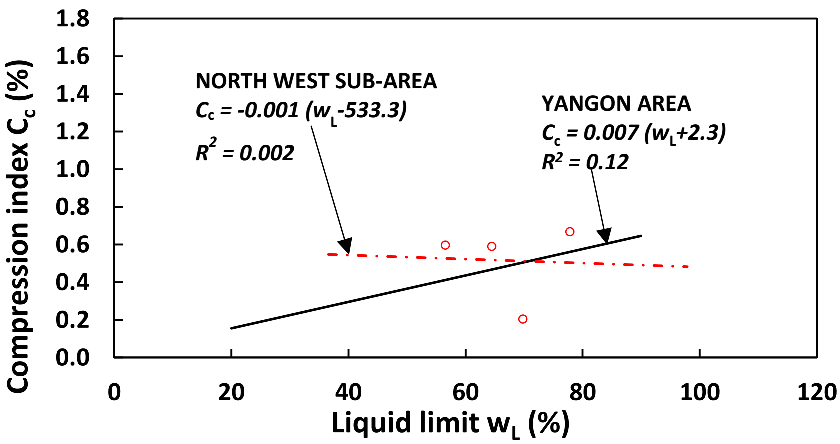
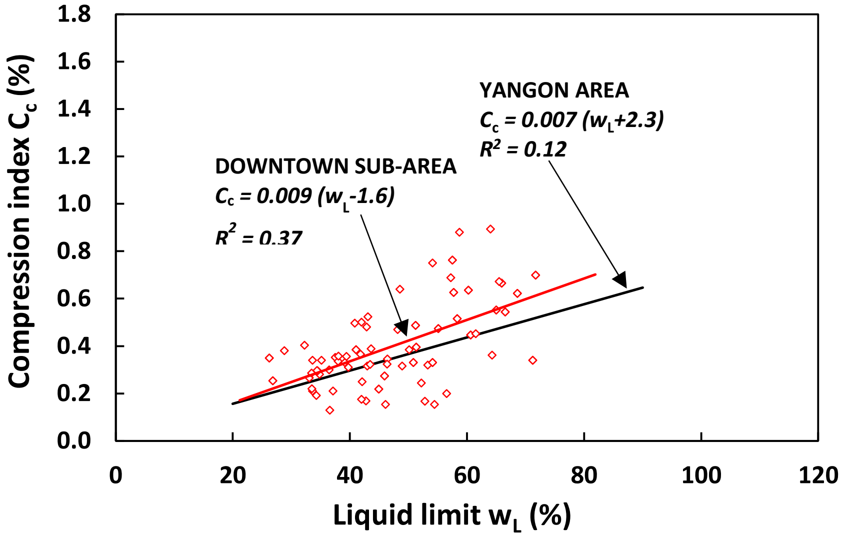
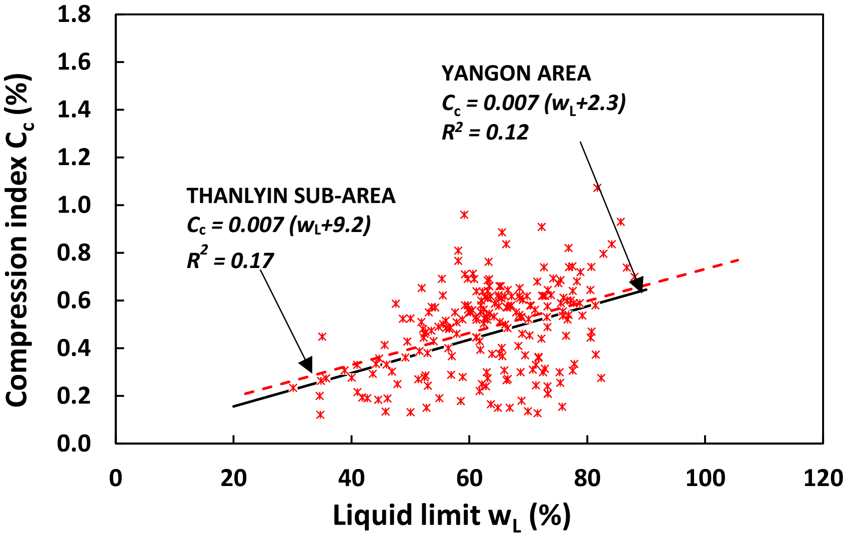


Figure 4-44 Correlations between Compression Index C_c and the liquid limit w_L for sub-areas in Yangon (a) ~ (b)

(c)



(d)



(e)

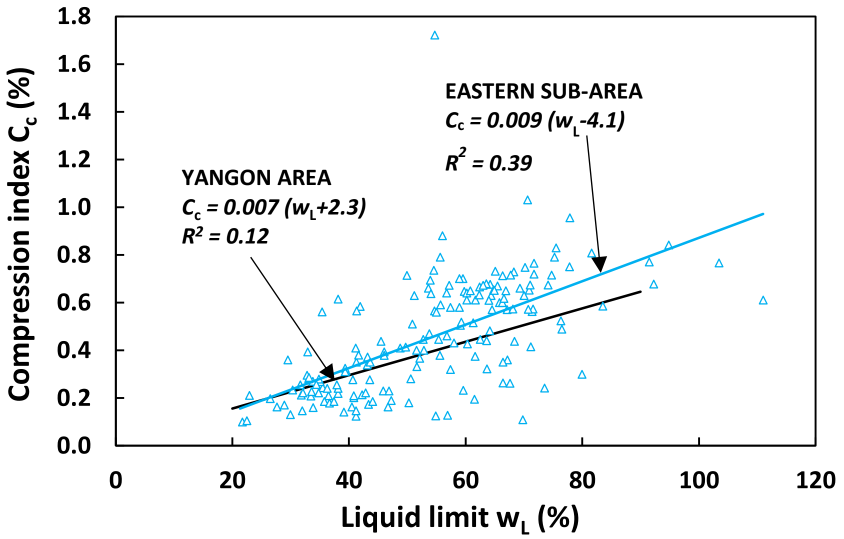


Figure 4-45 Correlations between Compression Index C_c and the liquid limit w_L for sub-areas in Yangon (c) ~ (e)

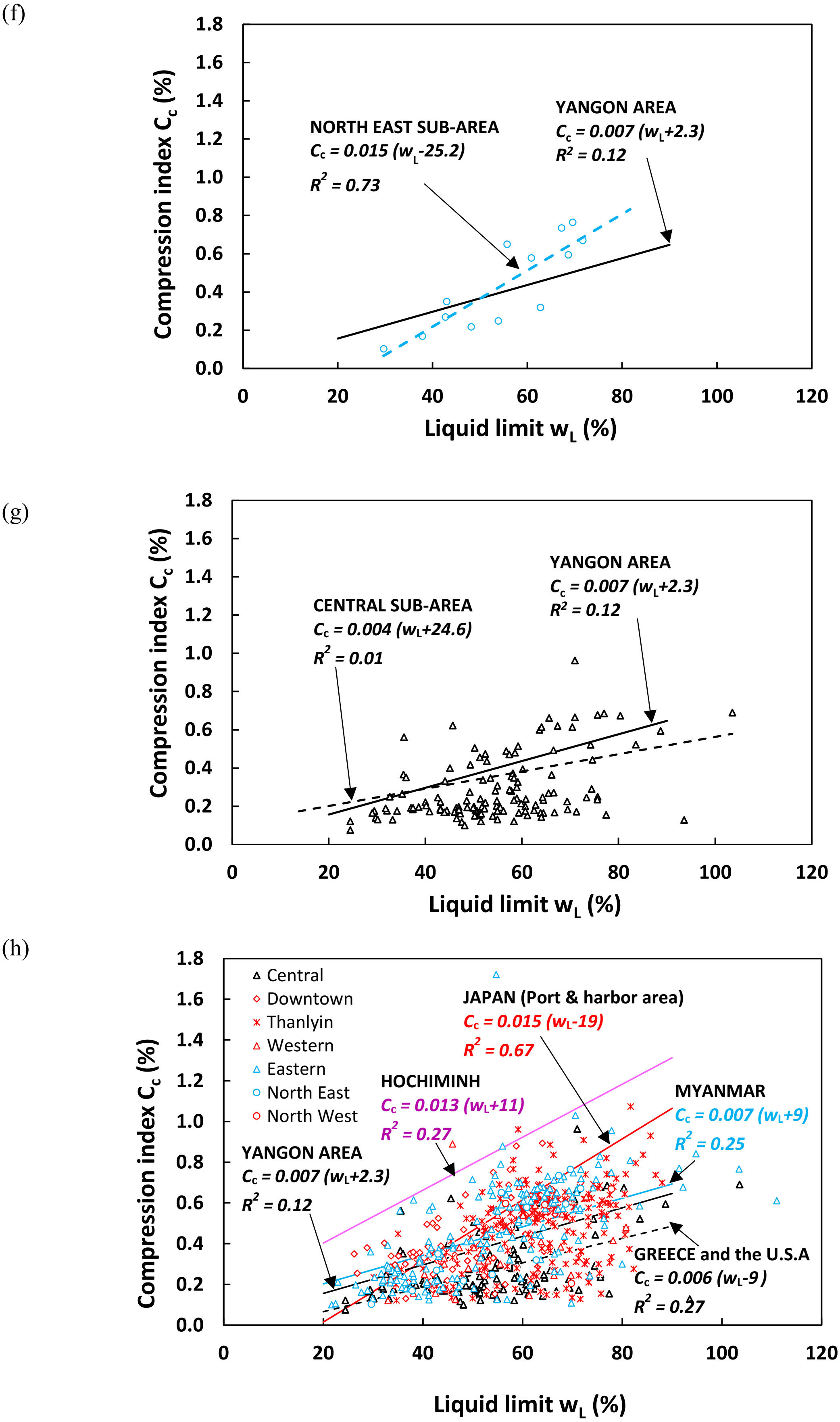


Figure 4-46 Correlations between Compression Index C_c and the liquid limit w_L for sub-areas in Yangon (f) ~ (h)

It is also known that initial void ratio, e_0 has some correlation with compression index (C_c) (Hough, 1957 [12]), Koppula, 1981 [31]), Al-Khafaji and Andersland, 1992 [32])). Figure 4-47 shows the relationship between C_c and e_0. The correlation lines between C_c and e_0, which have been proposed by researchers, are shown together with correlation for clays in Yangon as follows,

$C_c = 0.53\ (e_0 - 0.32)$	(Yangon clays, present study)	(16)
$C_c = 0.52\ (e_0 - 0.23)$	(Whole Myanmar clays, Murakami et al.,2015 [10]))	(17)
$C_c = 0.43\ (e_0 - 0.25)$	(Clays in Japanese port and harbor areas, Ogawa & Matsumoto,1978 [13]))	(18)
$C_c = 0.55\ (e_0 + 0.04)$	(Hochiminh clays, present study)	(19)

The coefficient of determination of Yangon clay (R^2 = 0.32) is not so high but the correlation line is close to that for the clays in Japanese port and harbor areas.

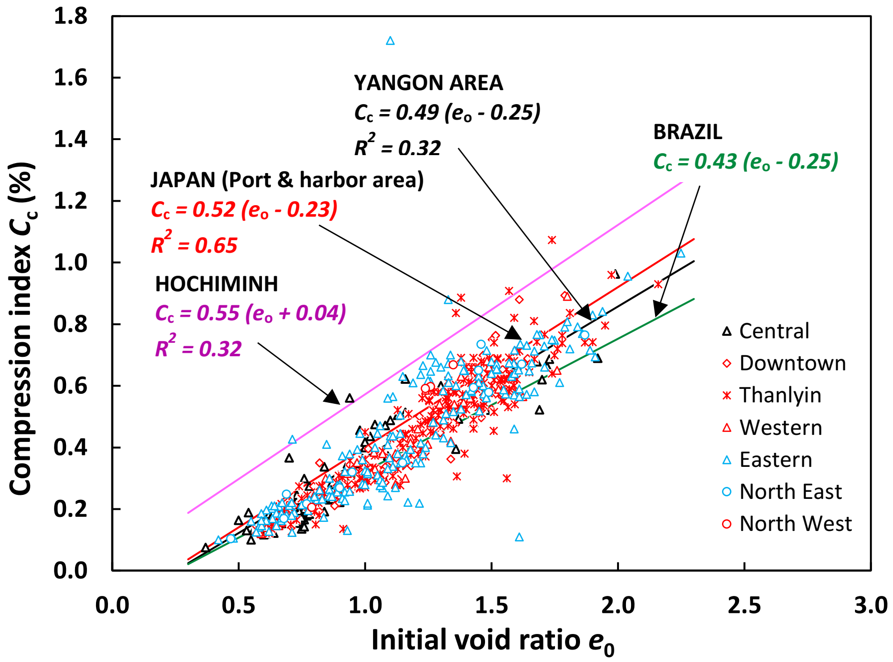


Figure 4-47 Relationship between C_c and e_0 of clays in each sub-areas in Yangon

Figure 4-48 shows the relationship for clays in Yangon between C_c and the swelling index, C_s, which is the gradient of e - log p curve at the unloading state in the consolidation test. As shown in the figure, there is a good correlation between C_c and C_s. The regression line is as follows,

$$C_s = 0.22\ C_c \qquad (R^2{=}0.70, \text{ clays in Yangon}) \qquad (20)$$

It is known that clays in Japan (port and harbor area [13)]) are usually at $C_s/C_c = 0.1$, however, the C_s of clays in Yangon is about 0.22 of the value for C_c. Yangon clays have almost half the C_c value of clays in Japan (port and harbor areas) as shown in Figure 4-48. Accordingly, the C_s value for Yangon clays are almost the same as clays in Japan (port and harbor areas).

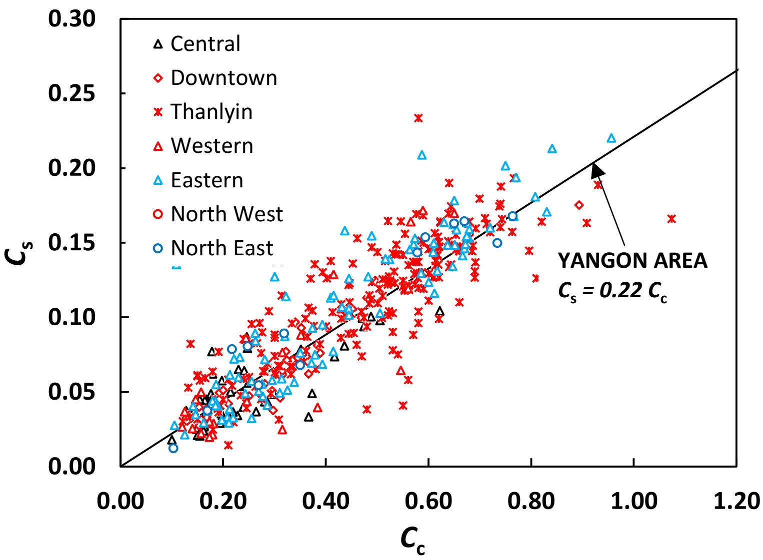


Figure 4-48 Relationship between C_s and C_c of clays in each sub-areas in Yangon

4.5.3 Coefficient of Consolidation, c_v

The coefficient of consolidation, c_v, that is related to the permeability of the soil and the speed of consolidation settlement, is a very important parameter for the planning of the construction projects. c_v for normally consolidated clays in Yangon is widely ranging between 30 and 1,500 cm^2/day. The average values of c_v for normally consolidated clays in Yangon is about 50 cm^2/day, when the natural moisture content w_n is more than 40%. Table 4-6 shows the average value of c_v under the normally consolidated state for Yangon and other countries. The average value of c_v for clays in Yangon is the almost same as clays from Japan and Hai Phong, Vietnam, and larger than those in Singapore and Bangkok as shown in Table 4-6.

Table 4-6 Average c_v for Yangon and Other Countries under Normally Consolidated State (Tanaka et al., 2002 [30])

Area	Silt (%)	Clay (%)	Liquid Limit w_L (%)	Coefficient of Consolidation c_v (cm^2/day)
Minami Honmoku, Yokohama, Japan	44.1	50.5	118	53
Osaka South Port, Japan	40.7	58.2	109	14
Yamashita Park, Yokohama	40.4	54.7	119	48
Hachirogata, Japan	36.2	63.4	176	60
Ariake, Saga, Japan	32.0	68.0	107	40
Bangkok, Thailand	20.9	77.0	89	6
Singapore	28.8	70.0	85	15
Chivai, Vietnam	21.5	78.5	82	12
Hai Phong, Vietnam	53.0	38.7	62	50
Yangon*, Myanmar	51.5	42.0	57	50

Figure 4-49 shows the relationship between the average c_v under a normally consolidated state obtained from standard consolidation tests and the liquid limit w_L. As shown in the figure, the plots are widely distributed and significant differences among the sub-areas in Yangon and between the two sedimentary basins are not found, except for a group shown with the open black circle in the figure, which is the data obtained from the Central Sub-area which has a larger c_v value but the same w_L. It also indicates that clays in the Central Sub-area is highly over-consolidated.

The followings are regression lines for clays in Yangon, the whole of Myanmar, Japan (port and harbor areas [13]) and Hochiminh respectively,

$c_v = 1{,}802 \cdot 10^{-2.00\,(w_L/100)}$ (Yangon clays, present study) (21)

$c_v = 2{,}240 \cdot 10^{-2.34(w_L/100)}$ (Whole Myanmar clays, Murakami et al.,2015 [10]) (22)

$c_v = 3{,}000 \cdot 10^{-1.59\,(w_L/100)}$ (Clays in Japanese port and harbor areas, Ogawa & Matsumoto,1978 [13]) (23)

$c_v = 1{,}886 \cdot 10^{-1.61(w_L/100)}$ (Hochiminh clays, present study) (24)

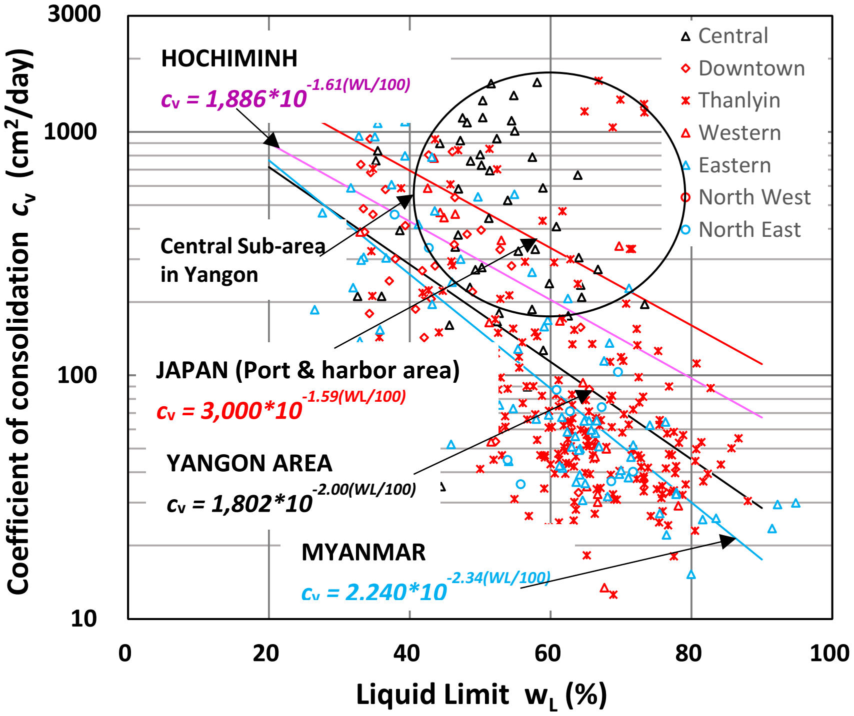


Figure 4-49 c_v – w_L relationship of clays in each sub-areas in Yangon

Compared to the clays in Japanese port and harbor areas and Hochiminh, the coefficient of consolidation c_v of clays in Yangon is 3/7, 3/4 at w_L = 40 %, 1/3, 1/2 at w_L = 60 % and 1/4, 1/2 at w_L = 80 %, respectively. Accordingly, the time required for Yangon clay to complete the required degree of consolidation is from 2 to 4 times that of clays in Japanese port and harbor areas and from 1.3 to 2 times that of Hochiminh clays under the same liquid limit.

4.6 Conclusion of this Chapter

Physical and mechanical properties for soft to firm clays in Yangon area were identified using undisturbed soil samples obtained with fixed piston sampler. Soil laboratory test results were studied, analyzed, and comparisons were made for the properties of clays in two different sedimentary basins, seven sub-areas in Yangon, and also clays in other countries. From the results, the followings can be concluded.

1) Based on the soil tests of samples taken by boring works, the soil profiles (cross sections) which covers Yangon area were presented. According to those soil profiles, it was identified that the thickness of the soft to firm clay layers in Yangon ranged from 5 to 25 meters. It was also found that the firm clay of the upper layer of the Tertiary deposit located at the Central Sub-area in Yangon includes lateritic clay, which showed physical and mechanical properties different from other sub-areas in Yangon.
2) The overview of the physical and mechanical properties of soft to firm clays in Yangon was presented. Comparisons of the physical and the mechanical properties was made between two sedimentary basins (Irrawaddy Delta Sub-basin and Pegu-Yoma Sittaung Basin) and among the seven sub-areas in Yangon. As a result, significant differences were not found between two sedimentary basins and among seven sub-areas in Yangon except for the Central Sub-area. The ranges and averages of the physical properties of clays in Yangon are summarized as follows,

- Density of soil particle, ρ_s : 2.7 g/cm^3 ($2.6 \leqq \rho_s \leqq 2.8$)
- Clay content, C : 41 % ($5 \leqq C \leqq 90$)
- Activity, A' : 1.11 ($0.25 \leqq A' \leqq 2.75$)
- Unit weight, γ_t : 18 kN/m^3 ($15 \leqq \gamma_t \leqq 21$)
- Natural moisture content, w_n : 40 % ($20 \leqq w_n \leqq 60$)
- Liquid limit, w_L : 57 % ($20 \leqq w_L \leqq 90$)
- Plastic limit, w_P : 24 % ($10 \leqq w_P \leqq 35$)
- Plasticity index, I_P : 32 % ($10 \leqq I_P \leqq 60$)

3) Yangon clay is the mostly over-consolidated clay. The over-consolidation ratio (OCR) of Yangon clays, which are at depths of shallower than 5 to 6 meters, and above the ground water level, is between 5 and more than 10. For the clays at depths of deeper than 5 to 6 meters, the OCR is ranges from 1.0 to 3.0, with an average of 1.6.
4) For most clays with depths of shallower than 5 to 6 meters and above ground water level, the unconfined compressive strength is more than 100 kN/m^2. Although large variation is shown, the unconfined compressive strength (q_u) of clays in Yangon increases with the depth deeper than 5 to 6 meters from ground surface.

5) Comparing the q_u to the p_c, the consolidation yield stress, q_u of the Yangon clay was 60 % of Japanese marine clays [13] under the same value of p_c. The strength reduction by swelling due to over-consolidation effect and the strength reduction due to disturbances during sampling and testing procedures might be ones of the reasons for having such a small normalized shear strength.
6) A good correlation between the compression index, C_c, and the natural moisture content w_n, was found in the Yangon clay. Comparing the C_c value at the median value of natural moisture content of 40 % of Yangon clay, the C_c of Yangon clay is 5/9 that of Japanese marine clay, and 2/3 that of Thi Vai clay in Vietnam.
7) In the relationship between the coefficient of consolidation c_v and the liquid limit w_L, c_v decreases with the increase of w_l, although its variation is large. Under the same liquid limit (w_L), the cv of Yangon clay is smaller than those of Japanese clays (port and harbor area [13]) and the Hochiminh clay in Vietnam, and the time required for Yangon clay to complete the required degree of consolidation is from 2 to 4 times that of Japanese marine clays and from 1.3 to 2 times that of the Hochiminh clays under the same liquid limit.

In Myanmar, numerous engineers from various countries have been joining projects for the development of infrastructure. In this situation, it is very important to understand the localities of the geotechnical properties of soft to firm clays in Yangon, which often become problematic soils in construction projects. The soil properties in Yangon portrayed in this paper would be useful for carrying out construction projects smoothly and successfully.

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5. Geotechnical engineering characteristics of soft clays in Thilawa area along the Yangon River

5.1 Outline of Study

In Thilawa where is located in Thanlyin sub-area at the eastern part of the Yangon River in southeast of Yangon, about 20ha of port development (20 billion yen in loan amount) is being advanced by Japanese ODA. Onshore area condition and shoreline condition are shown in Photo -5.1 to Photo -5.4. But soft clayey soil is so thickly (20m thick) distributed that Prefabricated Vertical Drain (PVD) method is adopted as a countermeasure against soft ground. Also, the development of the Thilawa Industrial Park (Special Economic Zone) with area of 2,400 ha was also proceeded behind the Thilawa Port area, the construction of the first stage (211 ha) has already been completed and about 80 companies have been contracted to set up their factories and half of them are Japanese companies (Refer to Photo – 5.5).

Photo - 5.1 Conditions of onshore area along Yangon River at Thilawa

Photo - 5.2 Conditions of shoreline of Yangon River at Thilawa

Photo - 5.3 Standing conditions on riverbed at Thilawa (No penetration into clay layer but not everywhere at Thilawa)

Photo - 5.4 Riverbed conditions at Thilawa Port Area (People can easily walk on the riverbed at some area at Thilawa due to thin firm clay layer (Thickness ≤ 0.3 ~ 0.5m) at riverbed in Thilawa)

From viewpoints of whole Yangon area, clays are not so homogeneous depending on the locations and the existence of tertiary deposit and its ridge running from north to south. In this chapter, the characteristics of clay in Thilawa along the Yangon River are focused on, which are relatively homogeneous compared to the clays at other areas in Yangon. The data processing and analysis of physical and mechanical properties of clayey soil was carried out. And the comparisons of those properties with ones in other areas and countries including Japan were also performed.

Figure 5-1 Sampling Points (Point A – E)
(Retouched to Google Map)

The soil data of the clays in the Thilawa area used in this study is based on laboratory test results of collected samples by soil investigation survey conducted by the Yangon office of Fukken Co., Ltd. [1)] in 2010-2016. These drilling and undisturbed sample collections by fixed piston type hydraulic sampler were all done using the Japanese boring machine and equipment and the methods prescribed in the Standards of Japanese Geotechnical Society [14)]. Therefore, there is no difference in quality of samples due to the difference in boring and sampling method. Accordingly, it can be compared with the laboratory test data in Japan. In this chapter, the data comparison with data in Hochiminh (hereinafter abbreviated as "Hochiminh clay") was also conducted, the data of which were collected by Fukken Minami Consultant Co., Ltd. in Vietnam from soil investigation works under similar conditions.

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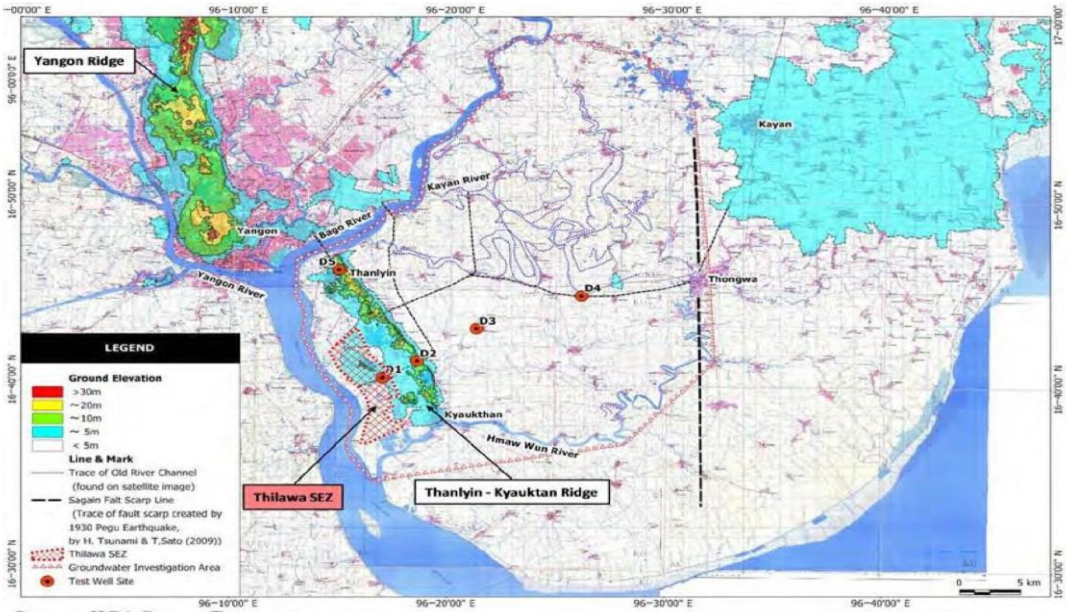
Photo - 5.5 On-going Projects in Thilawa Area

5.2 Topography and Geology of Thilawa Area

5.2.1 Topography of Thilawa Area

Between the two rivers, Irrawaddy River and Sittaung River, Pegu-Yoma Ridge runs from north to south to divide the lowland into east and west side. The southern end of Pegu-Yoma Ridge is the Yangon Ridge where the Yangon City is located. The Thanlyin-Kyauktan Ridge stretches southeastwards over the Bago River in Thanlyin and Kyauktan Townships as shown in Figure 5-2.

The Thanlyin-Kyauktan Ridge is a low relief hill with 5 to 25 m in elevation stretching in NNW - SSE direction, where shallow valleys develop, and paddy field spread in the western part. Eastern side of the ridge is very low and flat where the paddy field plain spreads very widely then its elevation is from 0.7 to 3.5 m (JICA,2014) [3].



Source: JICA Survey Team

Source: Data Collection Survey on Water Resources Potential for Thilawa Special Economic Zone and Adjoining Areas Final Report, September 2014, JICA [3]

Figure 5-2 Topography and Elevation in and near the Groundwater Investigation Area

5.2.2 Geology of Thilawa Area

There are not so many past geological survey reports in Thilawa area. However, JICA studied the groundwater conditions in 2013 to 2014 for "Data Collection Survey on Water Resources Potential for Thilawa Special Economic Zone and Adjoining Areas". JICA studied geological conditions in Thanlyin Area in that occasion. Then, geological conditions of Thanlyin area become clearer and are described as follows. [3]

(1) Regional Geology

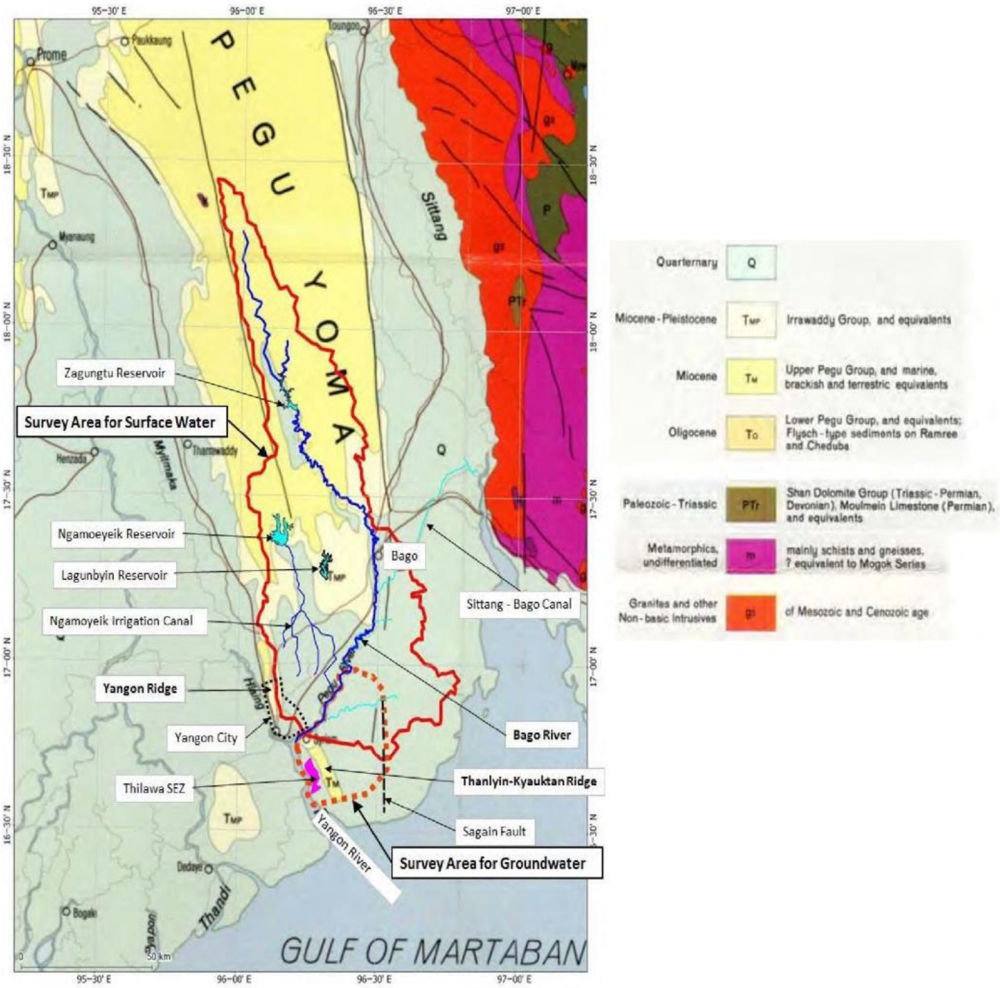
Pegu-Yoma Ridge consists mainly of the Pegu Group. The Irrawaddy Formation distributes along its periphery. The Pegu Group comprises weakly to moderately-consolidated sedimentary rocks of Tertiary Oligocene to Miocene age. They are considered to be marine deposits. The upper part mainly consists of an alternation of sandstone and mudstone. The middle part consists of sandstone, and the lower part consists of shale.

The Irrawaddy Formation is made up of continental deposits comprising weakly-consolidated medium to coarse-grained sandstone and conglomerate intercalated with weakly to moderately-consolidated muddy rocks.

The lowland, out of the ridge, is covered with unconsolidated silt, clay and sand of Quaternary age. These sediments were conveyed by river flow, so that their formation is named "Alluvial" deposit.

In Pegu-Yoma, some anticlines are identified. In the Yangon Ridge and the Thanlyin-Kyauktan Ridge, an anticline is inferred. A famous active lateral fault called the Sagaing Fault runs along the linear boundary between Pegu-Yoma and the Sittaung plain. The Bago Earthquake occurred in 1930 on the fault with an epicenter located to the north of Thongwa at south of Bago (JICA, 2014) [3)]. The linear fault scarp created by the earthquake is traceable even at present [12)].

Figure 5-3 shows the regional geology and Table 5-1 shows a stratigraphic table around the Thilawa area.



Source: Bender (1983); retouched by JICA, September 2014 [3)]

Figure 5-3 Regional Geological Map of Pegu-Yoma Ridge Area including Thanlyin-Kyauktan Ridge

Table 5-1 Stratigraphic Sequence in Eastern Irrawaddy Delta, Pegu-Yoma and Sittaung Basin (JICA, September 2014 [3)])

Age	Formation		Lithology	Approx. Maximal Thickness (m)
Quaternary	Alluvium		Sand, silt, clay	
Pliocene-Miocene?	Irrawaddy Group		Medium- to coarse-grained sandstone, gravel, conglomerate; thick-bedded, cross-bedded, massive	1,200
Angular unconformity				
Miocene ～Oligocene?	Pegu Group	Upper	Alternation of fine- to medium-grained sandstone and mudstone	1,000
		Middle	Fine- to medium-grained sandstone with little shale	900
		Lower	Sandy shale and shale intercalated with some fine-grained sandstone	850

Source: Bender(1983); partly changed on age and added with Alluvium.

(2) Geology of Thanlyin-Kyauktan Ridge

According to JICA Study (Data Collection Survey on Water Resources Potential for Thilawa Special Economic Zone and Adjoining Areas Final Report, September 2014) [3)], geology along Thanlyin-Kyauktan Ridge is described as follows.

Figure 5-4 shows the geological map and Table 5-2 shows stratigraphic table of Thanlyin-Kyauktan Ridge Area. Figure 5-4 was compiled based on geological maps by Win Naing et al. (1991) and Aye Thanda Bo (2001), topographical analysis of a satellite image, and outcrop reconnaissance.

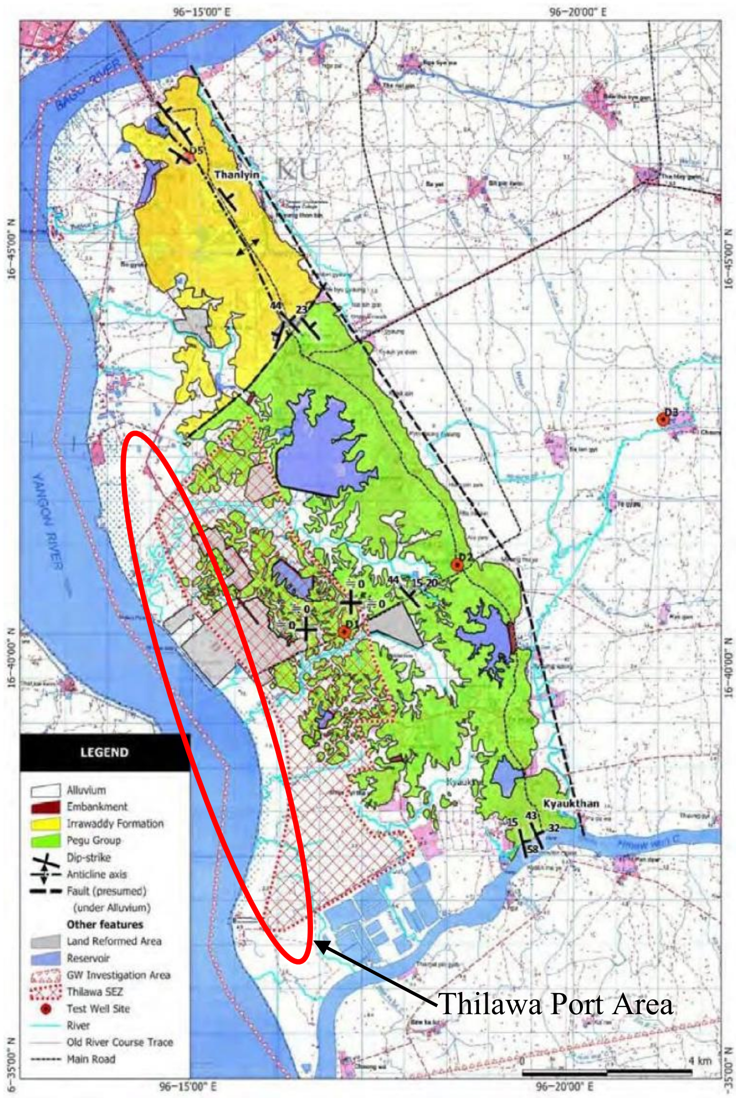
According to existing geological maps including regional ones, geologic units distributed in the area are the Irrawaddy Formation in the north and the Pegu Group in the south. They contact each other on an inferred fault running near Kyaik Kauk Pagoda in Thanlyin. Another fault is inferred along the eastern edge of the hill, because it shows a clear lineament.

The Irrawaddy Formation consists of semi-consolidated medium to coarse-grained sandstone with mudstone layers. The Pegu Group comprises an alternation of shale (or mudstone) and fine to medium-grained sandstone with ferruginous bands. According to the core boring and test well drilling results by JICA Study [3)], sandstone of the Irrawaddy Formation distributes down to approximately 100m from the ground and mudstone prevails below this depth. In the southern area where the Pegu Group distributes, the ground is covered mainly with mudstone. However, sandstone prevails in the core sample obtained at an eastern edge of the central part of the hill. The ferruginous bands were found in the core above 100m from the ground.

In the surface of the hill, the lateritic soil develops. Generally, a few meters of the surface are reddish brown or brownish in color and contain spherical or sphere-connected ferruginous concretions in the middle and lower portions. According to the JICA Report [3),] at the test well site in Thanlyin where the Irrawaddy formation distributes, lateritic weathering is remarkable as that the layer is brownish down to 25m below ground and a thick tabular iron vein with five centimeters thick was also found. In the Pegu-Group-covering area, lateritic weathering reaches about five meters. Such a surface layer of the hill was used as fill material for the embankment in SEZ and the nearby development area. This material is generally not permeable, because its mother rock is mainly mudstone and it contains much silt and clay as well as the concretions.

Table 5-2 Stratigraphic Table of Thanlyin-Kyauktan Ridge (Win Naing et al., 1991) [3)]

Geologic Age	Geologic Units	Symbol	Lithology	Lithology (detail)
Recent	Alluvium	A	Sands and Clayes	Yellowish grey, bluish grey, brownish grey coloured sands and clyas.
Pliocene	Irrawaddy Formation	I	Sand rocks interbedded with Clays and Mudstones	Medium to coarse grained sand rocks interbeded with clays and mudstones.
Oligocene ? to Miocene	Pegu Group	P	Alternationof Shale and Sandstone with ferrugenous bands	Alternation of shales and well consolidated argillaceous, bluish grey to brownish grey coloured, fine to mediumgrained micaceous sandstone with ferrugenous band.



Source: Compiled based on geological maps by Win Naing et al. (1991) and Aye Thanda Bo (2001), topographical analysis of satellite image and outcrop reconnaissance. Inferred faults indicated only for majors.

Figure 5-4 Geological Map of Thanlyin-Kyauktan Ridge (JICA, 2014) [3)]

5.3 Soil data and method of the study

5.3.1 Sampling points and sampling method

Figure 5-1 shows sampling location (points) in Thilawa in Thanlyin sub-area used in this study. Sampling points are 5 points in the Thilawa port area, located along the Yangon River at west side of the Thilawa Industrial Park (SEZ), with 750 m wide in the east-west direction and 7.4 km long in north to south direction (from Point A to Point E).

Figure 5-5 shows representative stratification of ground from Point A to Point E. The clay layer that is the object layer of this study was Clay I layer and Clay II layer which are equivalent to AC-I in Yangon Sub-areas which are soft clay layers with average N value of 4 or less. The representative photo of soft clay (Clay-II Layer) at Thilawa is shown in Photo – 5.6. The layer thickness is 16 m to 22 m on land and 11 m to 16 m in the river.

Photo - 5.6 Soft clay (Clay-II, equivalent to AC-I Layer in Sub-areas in Yangon) at Thilawa

The numbers of collected samples at each point is 172 at Point A, 38 at Point B, 55 at Point C, 227 at Point D and 29 at Point E. The soil data from these samples are compiled, analyzed and compared with the data already presented as a paper on properties on clays in Myanmar [5)] (hereinafter referred to as "Myanmar clay") and the data from Port and Harbor Research Institute, Japan [15)] (hereinafter referred to as "Japan clay"). As Thilawa area is located about 40 km away from the mouth of the Yangon River, this area has about 6 meters of tidal difference and it is a brackish water area containing salt. Therefore, it is conceivable that the comparison between Thilawa clay and the marine clay in Japan is possible. For some soil data such as compression index, the following data comparison has been carried out such as with the data analysis results of samples collected on the west, south and east coast of Korea by Yoon et al. [7)] (hereinafter referred to as "Korea clay"), the data on compressibility of clay samples taken in Egypt (3/4 of all samples in Greece, 1/4 in the United States) by Abdrabbo et al. [8)] (hereinafter referred to as "Egypt clay" and “United States clay”) and the data of samples taken in Brazil by Cozzolino et al [10)] (hereinafter referred to as "Brazil clay").

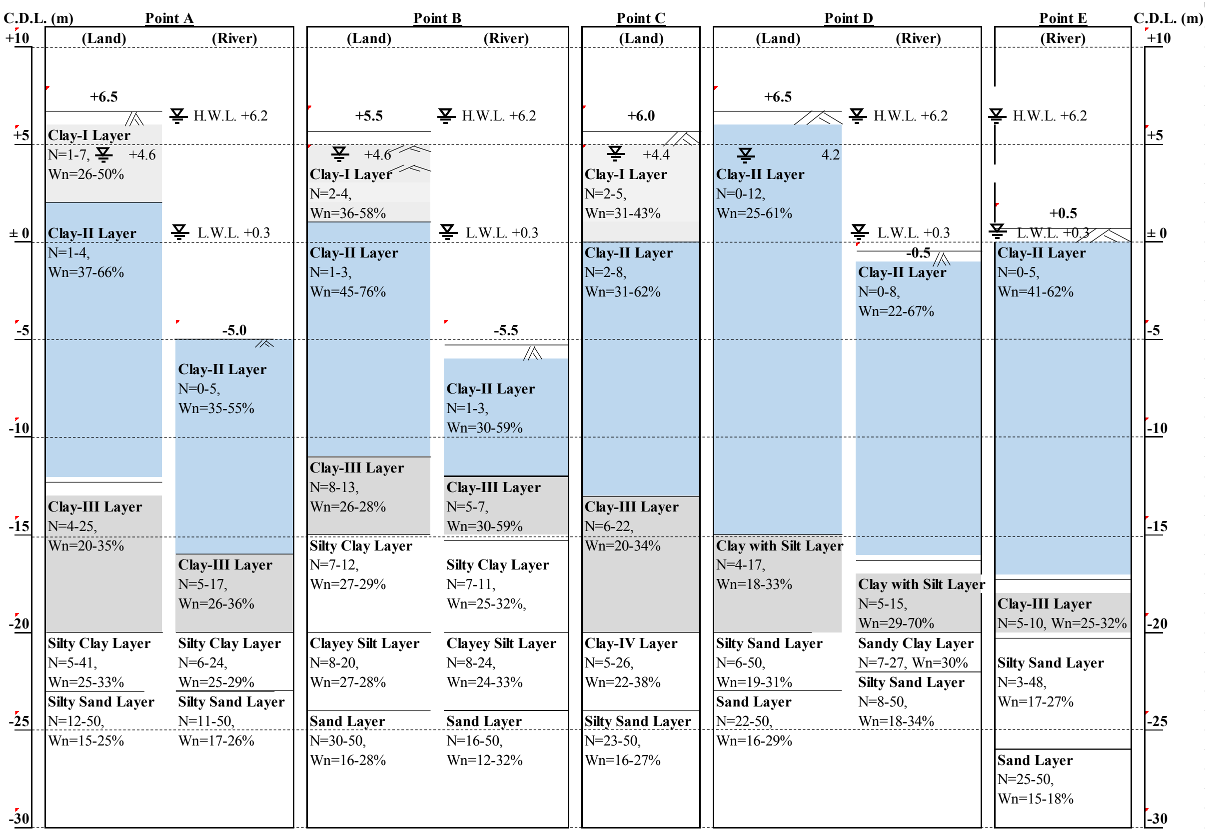


Figure 5-5 Representative Soil Stratification at Each Point [2)]

5.4 Physical and Mechanical Properties of Soft Clays at Thilawa Area

5.4.1 Physical Properties of Soft Clays at Thilawa Area and Comparison with other Country's Clays

Figure 5-6 shows the density of soil particle of Thilawa clay. From the figure, the average value and standard deviation value of the density of soil particle of Thilawa clay are 2.705 g/cm^3, 0.036 g/cm^3, and 2.686 g/cm^3, 0.052 g/cm^3 for the Japan and 2.681 g/cm^3, 0.034 g/cm^3 for Myanmar clay (average value for whole Myanmar [5)]). In comparison with the above values, Thilawa clay shows a slightly larger value than Japan and Myanmar clay.

Figure 5-7 shows the clay content (ratio of clay fraction with particle size of 5 μm or less) for each point from Point A to Point E in Thilawa area. Distribution range of clay content throughout the Thilawa area is as wide as 5 to 90%. However, the average value is in the range of 35 to 60%. The value at Point A out of 5 points is somewhat larger than other points but the average distribution range at all points is between 30 and 90%, and the average value is about 60%.

The ratio of the Plasticity Index to content in clay particles of less than 2 μm (%) is termed the activity *(A)* of the soil with the following definition (Skempton, 1953 [6]), Asakawa, 1972 [13])),

$A = I_P$ (Plasticity Index of soil)/ Content of clay particles less than 2 μm (%). (1)

It is known that the value of A is reflected by the water retention capacity of clay minerals and its surface activation and range is related to the clay minerals in soil as follows (Skempton, 1953 [6]), Asakawa, 1972 [13])),

$$\left.\begin{array}{ll} \text{Active clays (ex. Smectite clay):} & A \geq 1.25 \\ \text{Normal clays (ex. Illite clay):} & 0.75 < A < 1.25 \\ \text{Inactive clays (ex. Kaolinite clay):} & 0.75 \geq A \end{array}\right\} \quad (2)$$

In this study, using a particle clay content (%) of less than 5μm instead of 2μm, the activity of clay is defined as A', because all hydrometer tests were carried out based on the JIS procedure in which clay particle size is defined as having a grain size of less than 5μm.

Figure 5-8 is a histogram of activity A' and 60% of all samples is $A' \leq 0.75$, 35% is $0.75 < A' \leq 1.25$, then 5% is $1.25 < A'$. Therefore, it is conceivable that about 60% of these samples are classified into inactive clays (e.g. Kaolinite clay) and 35% is normal clays (e.g. Illite clay). Compared with the average of A' in clays in Japan (port and harbor area) [15]) which has average A' of 1.11 with a standard deviation 0.53, and average A' of 1.14 with a standard deviation 0.97 for Myanmar clay [5]), and average A' of 0.74 with a standard deviation of 0.25 for Thilawa clay. Accordingly, it can be said that the A' of Thilawa clay is slightly smaller than one of Japan clay (port and harbor area) and Myanmar clay (whole area).

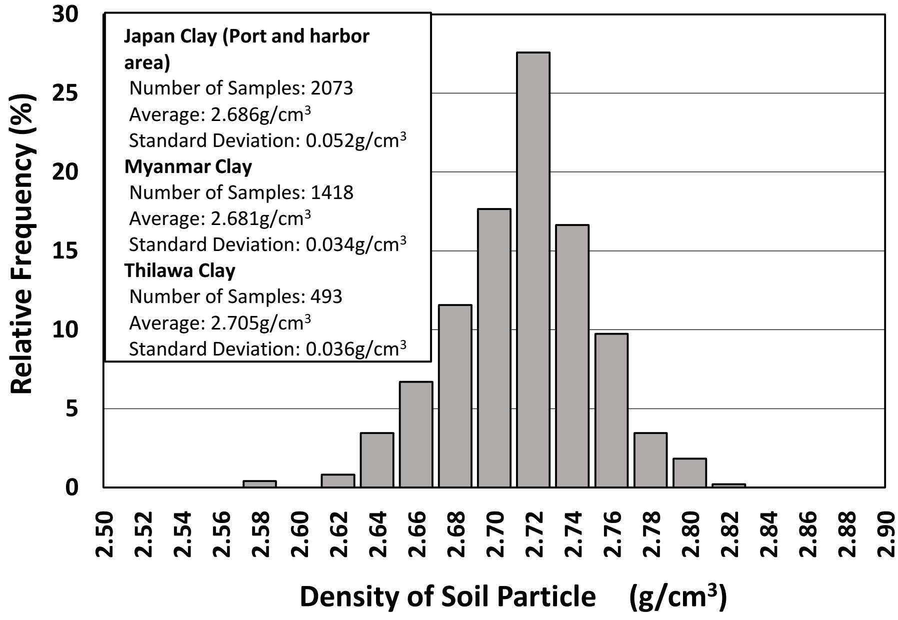


Figure 5-6 Histogram of Density of Soil Particle (Thilawa Area)

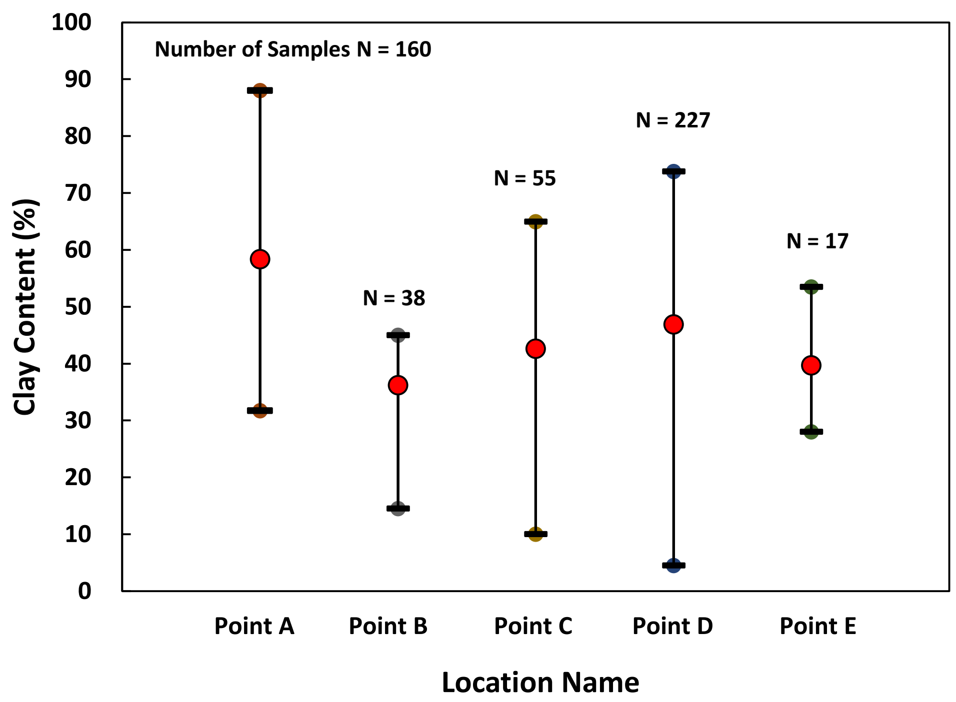


Figure 5-7 Clay Content at Each Sampling Point in Thilawa Area

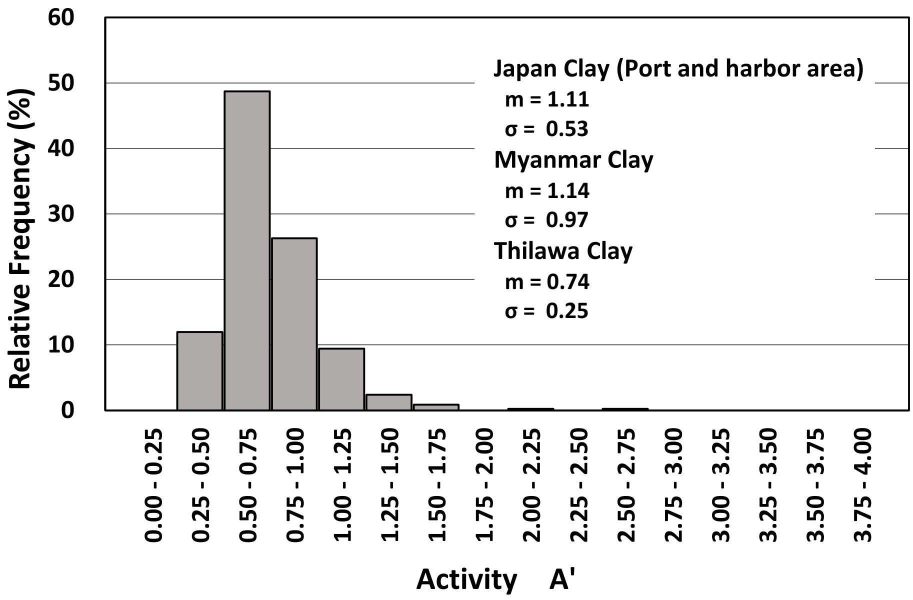
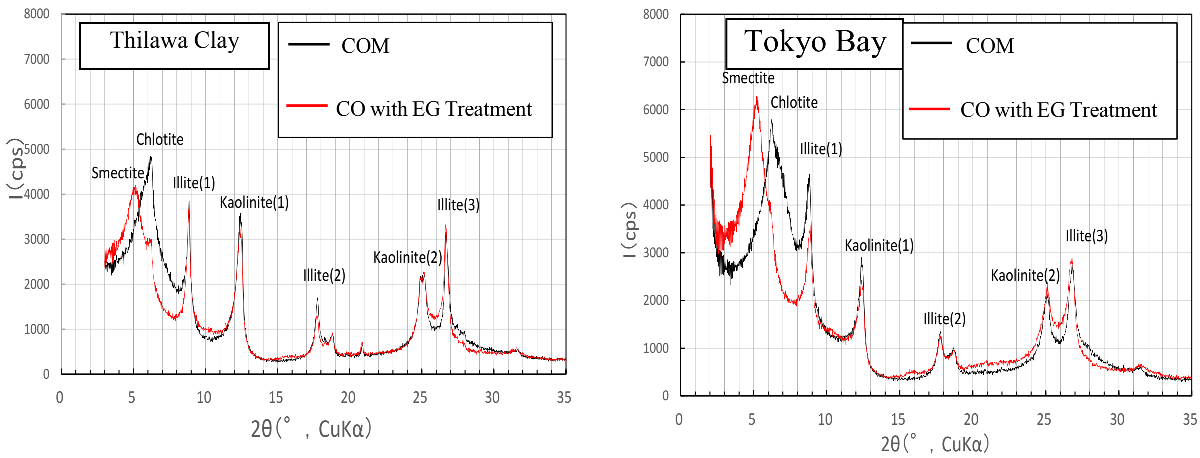


Figure 5-8 Histogram for Activity (*A'*) of Thilawa Clay

Figure 5-9 shows the results of X-ray diffraction test (XRD) of Thilawa clay and Tokyo Bay clay (liquid limit 133.9%, plasticity limit 60.4%) which has many previous studies in Japan. According to the figures, it is similar to Tokyo Bay clay that Thilawa clay mainly consists of kaolinite soil and illite soil. Both clays include smectite that has high water retentivity and ion exchange property together with kaolinite, illite and chlorite. Accordingly, there is no big difference in the clay mineral composition.



COM : Constant Orientation Method

CO with EG Treatment = Constant Orientation with EG Treatment

Figure 5-9 Result of XRD for Thilawa Clay and Tokyo Clay

Figure 5-10 is a plasticity chart showing the relationship between the plasticity index I_P and the liquid property limit w_L. Thilawa clay is located above the A line in the plasticity chart, most of which shows high plasticity with a liquid limit of 50% or more and is classified as CH (clay: high plasticity limit). In Japan, marine clay has high liquid limit up to about 150% which are seen in many places, whereas in Thilawa clay liquid limits exceeding 100% were not found.

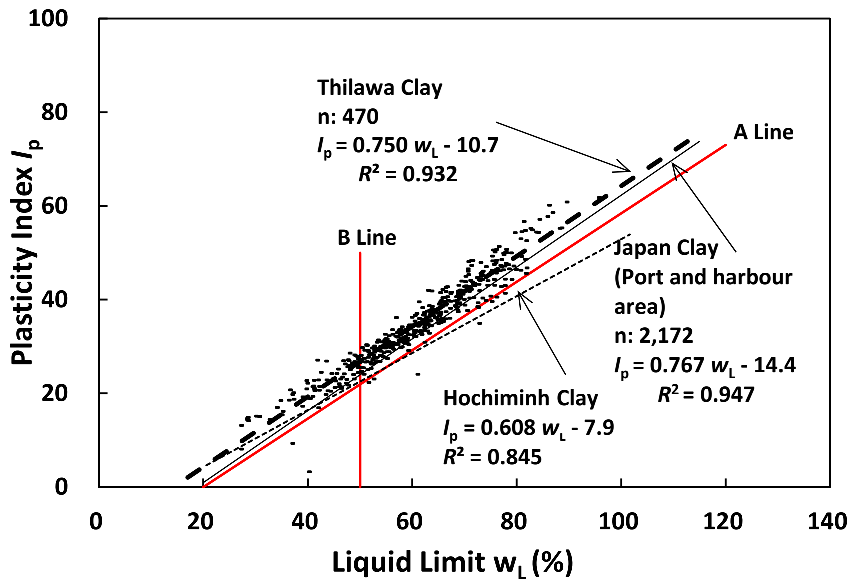


Figure 5-10 Thilawa Clay on Plasticity Chart

Figure 5-11 shows the relationship between the natural moisture content w_n and the liquid limit w_L. The straight line in the figure is the regression line obtained from those raw data, and the regression line of each clay is shown as follows.

$w_n = 0.77\, w_L$ (Thilawa clay) (1)

$w_n = 0.62\, w_L$ (Myanmar clay) [5)] (2)

$w_n = 0.92\, w_L$ (Japan clay, port and harbor area) [15)] (3)

$w_n = 0.96\, w_L$ (Hochiminh clay) (4)

The regression line of Thilawa clay is located approximately in the between the Japanese clay and the Myanmar clay. In the case of Japanese clay, the natural moisture content is often higher than the liquid limit, but in the case of Thilawa clay, most of the samples like the Myanmar clay, the natural moisture content was below the liquid limit. This is considered to be one reason that the over-consolidation ratio of Myanmar clay is larger than Japanese clay as described below.

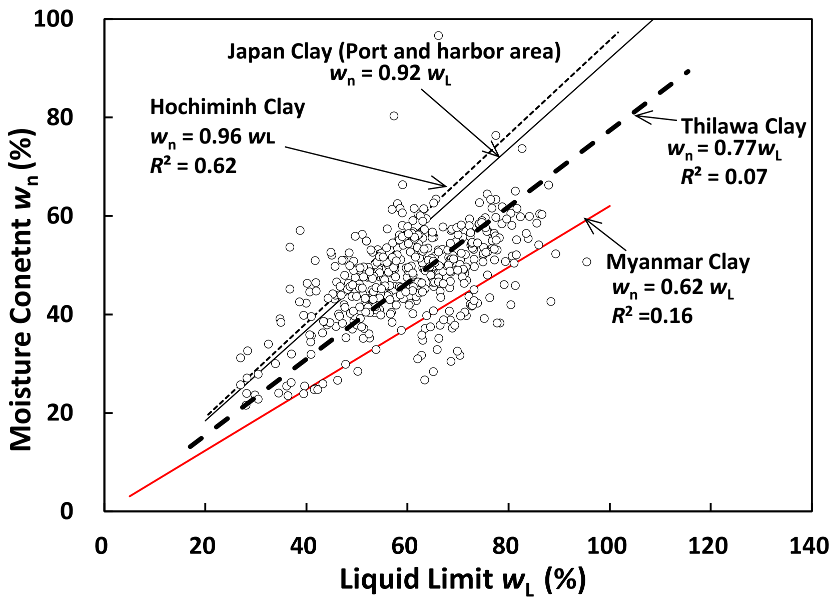


Figure 5-11 Relationship between Natural Moisture Content (w_n) and Liquid Limit (w_L)

Figure 5-12 shows the relationship between effective soil overburden pressure and consolidation yield stress of Thilawa clay. As shown in the figure, the consolidation yield stresses of most samples are larger than the overburden pressure. In Japan clay (Port and Harbor area), the consolidation yield stress is 1.25 times the soil overburden pressure (σ'_{vo}), whereas in Myanmar clay it is 1.62 times. While plotted data of Thilawa clay is widely ranged, its average was 2.26 times of σ_{v0}' and it is larger than Myanmar clay.

Figure 5-13 and Figure 5-14 show the relationship between the depth (CDL) and the over-consolidation ratio (OCR) on land and in the river of Thilawa clay respectively. As shown in the figures, except for the layer with a large over-consolidation ratio at depth of 5 to 6 m (up to around CDL ± 0 m) from the ground surface on land area in Thilawa, the representative value of OCR is about 1.4 in the depth direction. On the other hand, in the river, the soil overburden thickness decreases even at the same level (CDL) compared to on land. So, OCR varies in the range of 1 to 6. It seems that clay in the river has undergone fluctuation (erosion) of the river bed so that OCRs becomes large values than clays on land.

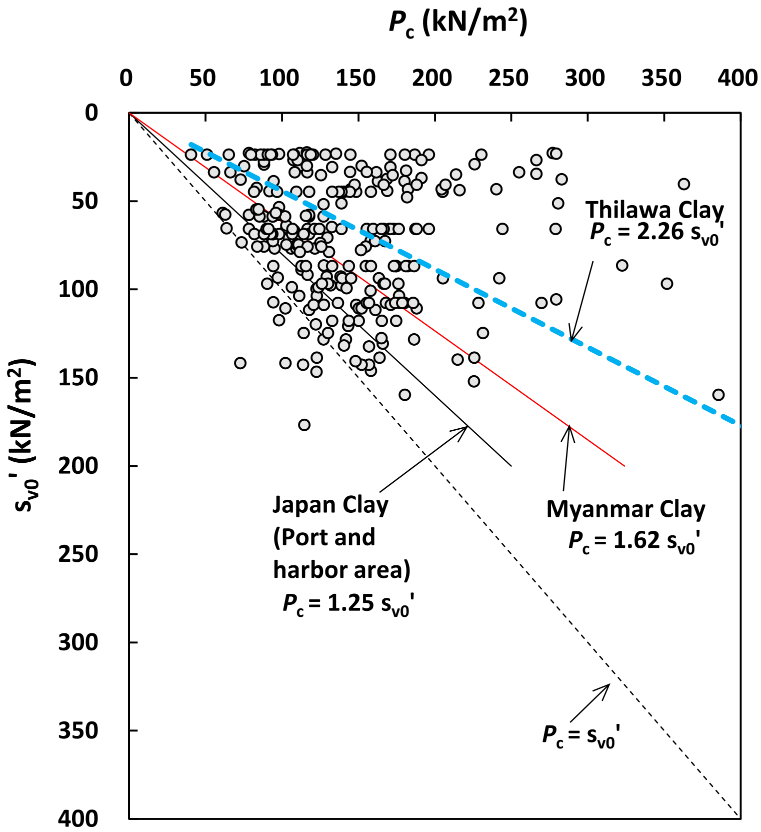


Figure 5-12 Effective Overburden Pressure (σ'_{vo}) and Consolidation Yielding Stress (p_c)

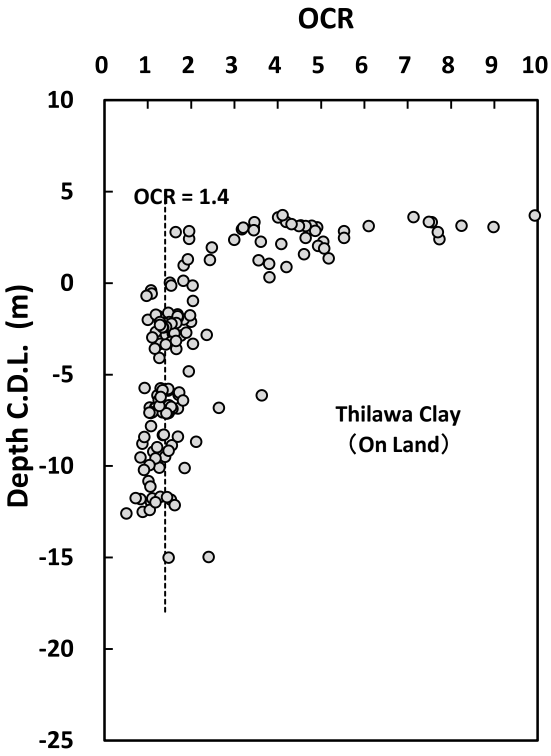


Figure 5-13 Over-consolidation Ratio (OCR) with Depth (C.D.L) (On Land)

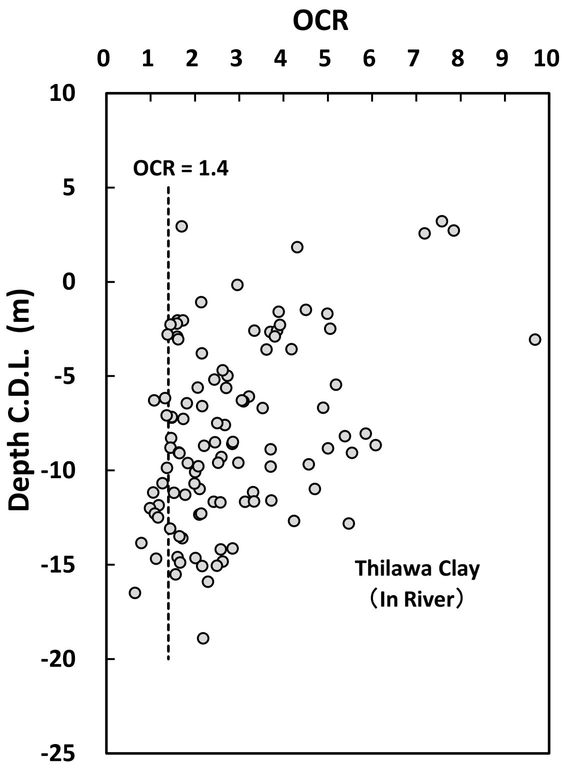


Figure 5-14 Over-consolidation Ratio (OCR) with Depth (C.D.L) (In River)

5.4.2 Mechanical Properties of Soft Clays at Thilawa Area and Comparison with other Country's Clays

The mechanical properties of clay are important characteristics for studying measures against stability and settlement of ground, which often become problems in design and construction stage. In this chapter, the strength properties and consolidation properties of Thilawa clay is studied by comparing them with other country's clays.

(1) Unconfined Compressive Strength (q_u) and Shear Strength Ratio (c_u/p')

Figure 5-15 and Figure 5-16 show the unconfined compressive strength q_u with depth (CDL) of Thilawa clay for on land and in the river respectively. As shown in the figures, the unconfined compressive strength q_u (kN/m^2) can be approximated well by the following formula, although slight variations are observed in both the on land and in the river.

$$q_u = 45 - 4.0\,z \quad (z : \text{CDL (m)}) \qquad (5)$$

The strength can be represented by the same approximate expression for both on land area and in the river as described above. After the past flat accumulation of soils (equal maximum overburden load) at both on land and in the river, river bed was eroded and formed without exceeding the same past maximum overburden load (pre-consolidation pressure). In addition, there are samples with high q_u of 70 to 140 kPa around at the depth of CDL + 4 m on land area, which is considered to be the effect of drying.

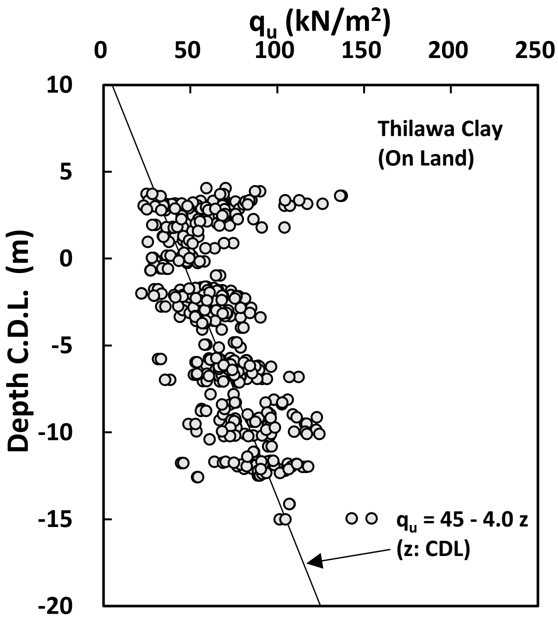


Figure 5-15 Unconfined Compressive Strength (q_u) with Depth (C.D.L) (On Land)

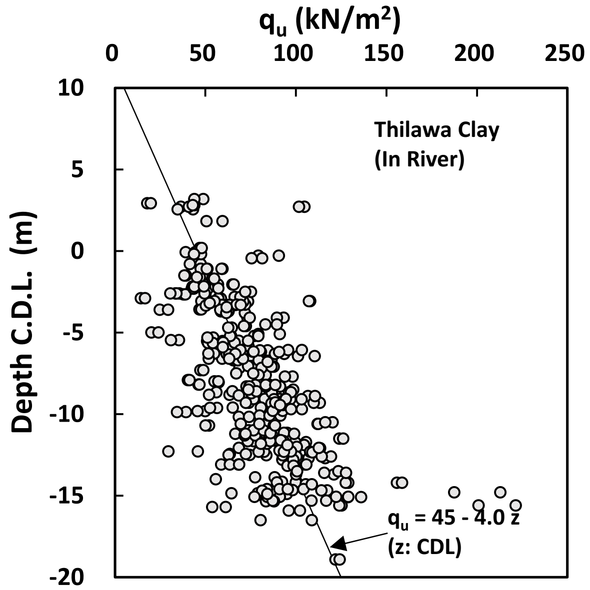


Figure 5-16 Unconfined Compressive Strength (q_u) with Depth (C.D.L) (In River)

Figure 5-17 is a histogram of failure strain during the unconfined compression test. In general, it is said that a failure strain of 6% or less is considered to be less undisturbed sample [14]. About 80% of the total number of samples of Thilawa clay have a failure strain of 6% or less. Ogawa et al. [15] show the relationship between ε_f and sampling depth for Japan clays (port and harbor area). Reading data shallower than 20 meters of sampling depth from the figure of Ogawa et al. [15], similar histogram can be found as shown in Figure 4-17. According to the figure, it is judged that the quality of the samples of Thilawa clay taken by the same sampling method as in Japan is almost the same as that of Japanese samples as judged from failure strain.

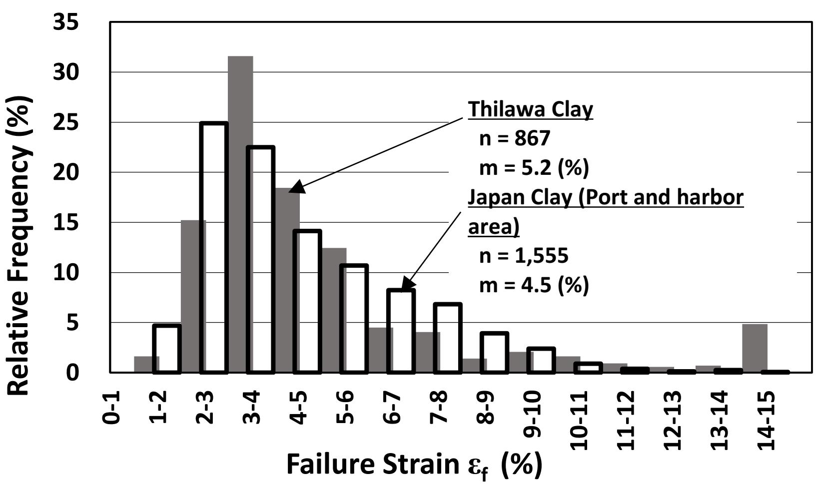


Figure 5-17 Histogram for Failure Strain of Thilawa Clay and Japan Clay

Figure 5-18 shows the relationship between unconfined compressive strength q_u for Japan clay, Hochiminh clay, Myanmar clay and Thilawa clay and consolidated yield stress p_c obtained from the standard consolidation test. As shown in the figure, the relationship between q_u and p_c is different by clay, and the respective regression lines are obtained as follows.

$q_u = 0.445\, p_c$ (Thilawa clay) (6)

$q_u = 0.339\, p_c$ (Myanmar clay [5]) (7)

$q_u = 0.598\, p_c$ (Japan clay, port and harbor area [15]) (8)

$q_u = 0.617\, p_c$ (Hochiminh clay) (9)

From the above, compared to Japan clay, q_u of Hochiminh clay is similar to Japan clay for the same p_c, but for Myanmar clay it is about 57% of Japan clay and 74% for Thilawa clay. The c_u obtained by using $c_u = q_u / 2$ from the formulas (6), (7), (8) and (9), the normalized shear strength c_u / p_c calculated for Japan Clay, Hochiminh clay, Myanmar clay and Thilawa clay were 0.30, 0.31, 0.17 and 0.22, respectively. The ratio of the undrained strength of Thilawa clay to the consolidation yield pressure was about 2/3 of that of Japan clay (port and harbor area). It is conceivable that the Thilawa clay becomes over-consolidated due to unloading as a cause of the small shear strength ratio to p_c [16]. Assuming that the cause of over-consolidation of Thilawa clay adjacent to a river is caused by unloading due to river bed erosion, it is conceivable that the shear strength is lowered due to unloading and swelling by water absorption and expansion. The rebound on expansion of Thilawa clay is also studied later in the consolidation properties.

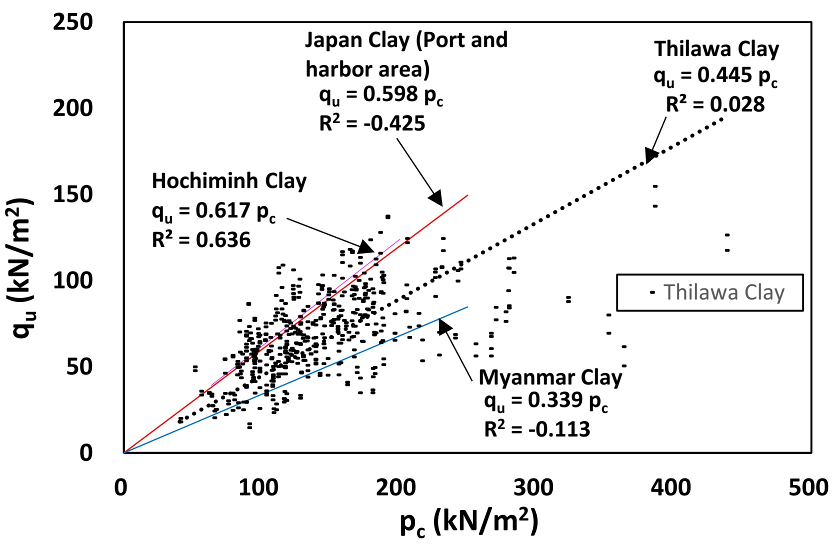


Figure 5-18 Relationship between Unconfined Compressive Strength (q_u) and Consolidation Yielding Stress (p_c)

The shear strength ratio (c_u/p) can be considered that the ratio between the undrained shear strength c_u and the consolidation pressure p_c. The relation between c_u/p_c and the physical properties were calculated for Thilawa clay from the relationship between the normalized shear strength c_u/p_c and plasticity index I_P of formula (10) by Skempton [11)] and liquid limit w_L of formula (11) by Hansbo [18)]. These formulas are known as follows.

$$c_u/p = 0.11 + 0.0037\, I_P \quad \text{(Skempton)} \qquad (10)$$

$$c_u/p = 0.0045\, w_L \qquad \text{(Hansbo)} \qquad (11)$$

For Thilawa clay, undrained shear strength c_u was obtained as $c_u = q_u/2$ and c_u/p_c was calculated by the consolidation yield stress p_c obtained from the standard consolidation test as consolidation pressure, and the relationship with the plasticity index is shown in Figure 5-19. As shown in the figure, the plasticity index I_P of Thilawa clay is mainly distributed in the range of $20 \leq I_P \leq 50$, the normalized shear strength c_u/p_c is randomly distributed in the range of 0.1 to 0.5, and no linear correlation indicated by Skempton was found. Figure 5-20 shows the relationship between liquid limit w_L and c_u/p_c. For the range of $45 \leq w_L \leq 80$, the normalized shear strength c_u/p_c is randomly distributed in the range of 0.1 to 0.5, and linear correlation indicated by Hansbo is not recognized.

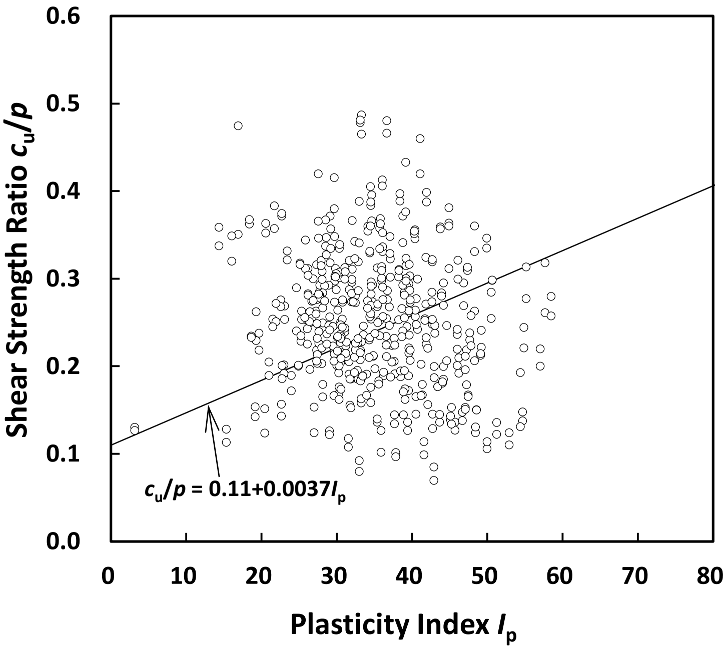


Figure 5-19 Relationship between normalized shear strength (c_u/p_c) and Plasticity Index (I_P)

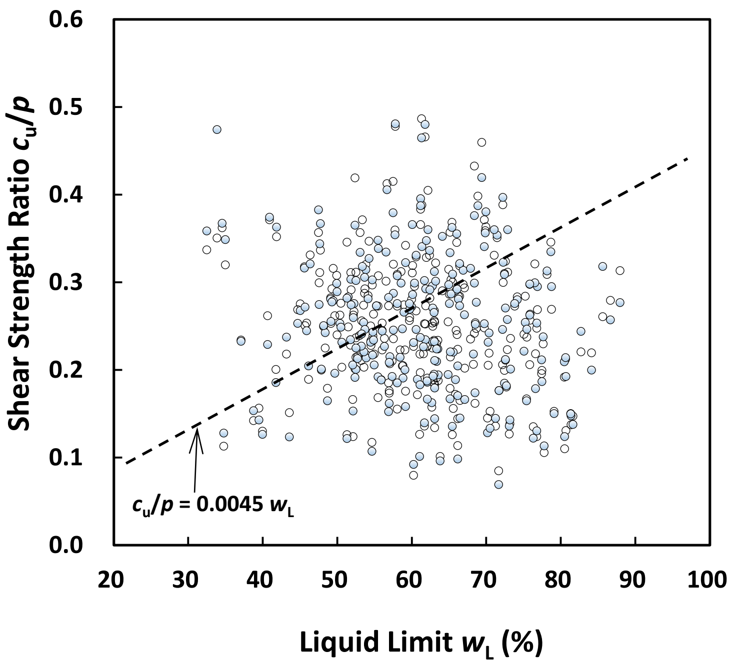


Figure 5-20 Relationship between normalized shear strength (c_u/p_c) and Liquid Limit (w_L)

(2) Characteristics of Shear Strength by Triaxial Compression Test (CIU)

Figure 5-21 shows lines connecting points obtained by plotting the strength c_u (1/2 of the maximum deviator stress) of each sample obtained by triaxial compression test (CIU) against the consolidation pressure σ_c'. Here, c_u/σ_c' is the shear strength ratio by the CIU test. In the test with the sample of GL-3 m near the ground surface, the shear strength ratio was more than $c_u/\sigma_c' = 0.4$ but for the other samples deeper than that, c_u/σ_c' is about 0.30 to 0.35. It is about the same value as the Japan clay (port and harbor area) [16), 19)], but it is larger than the shear strength ratio 0.22 obtained from the unconfined compressive strength q_u and the consolidation yield stress p_c. This is because the triaxial tests were carried out under isotropic condition which is not K_0 condition like in-situ conditions. Strength reduction by unloading due to sampling is significant in the unconfined compression test, and as a result, it is lower than one of the triaxial compression (CIU) test. As a result, it is possible that the shear strength ratio obtained from the unconfined compression test was smaller than ones from triaxial compression test (CIU).

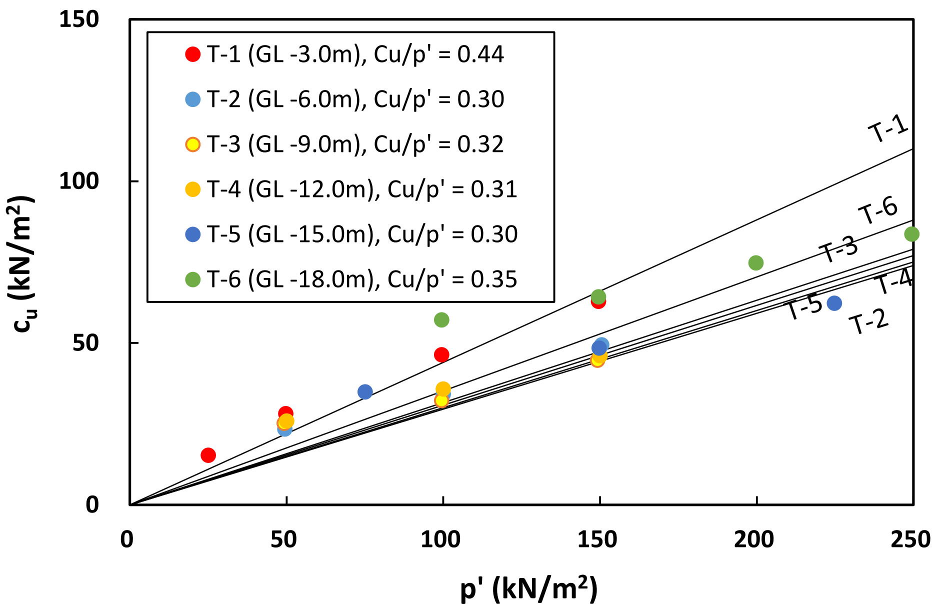


Figure 5-21 Shear Strength Ratio (c_u/p') obtained from Tri-axial Compression Test (CIU) (Thilawa Point D)

Figure 5-22 is a stress path diagram in triaxial compression test. The failure envelope curve at the time of failure is a point obtained by plotting the vertex of the Mohr circle with respect to mean effective stress ($s' = (\sigma_1'+\sigma_3')/2$) at the maximum of axial deviator stress ($t = (\sigma_1-\sigma_3)/2$). And the average gradient at that time was θ = 22.7°. The frictional angle ϕ' of the effective stress can be obtained from the relationship, sin ϕ′ = tanθ. The results of comparing the relationship between ϕ′ and the plasticity index I_P of Thilawa clay with Alluvial and Diluvial clay in Osaka Bay is shown in Figure 5-23. As shown in the figure, the shear resistance angle ϕ' of Thilawa clay against the same plasticity index I_P was equivalent to Diluvial clay of Osaka Bay (ϕ′ = 20 to 28 °) and the average value was ϕ′ = 25 °. This is slightly smaller value compared to marine clay in Japan (Japan clay, port and harbor area), which is almost ϕ′ = 30 to 35 °.

Coefficient of pore water pressure at failure in triaxial compression test A_f (= Δ_{uf} / $\Delta(\sigma_1-\sigma_3)_f$, $\Delta(\sigma_1-\sigma_3)_f$: Maximum deviator stress, Δ_{uf} : Pore water pressure at failure) of Thilawa clay is shown in comparison with Osaka Bay clay in Figure 5-24. Since coefficient of pore water pressure A_f varies depending on the over-consolidation ratio, horizontal axis σ_c'/p_c in the figure is consolidation pressure normalized by consolidation yield stress. As shown in the figure, coefficient of pore-water pressure A_f at failure of Thilawa clay matches well with that of Osaka Bay clay. It can be said that there is no big difference in the dilatancy characteristics.

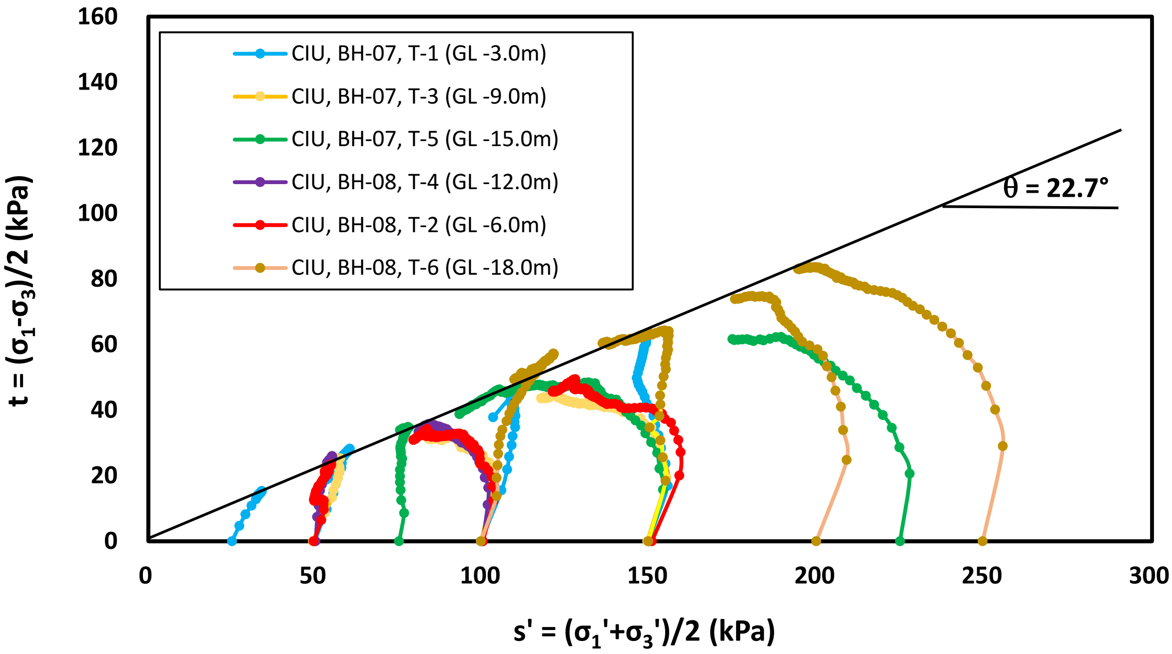


Figure 5-22 Stress Paths of Tri-axial Compression Test (CIU) (Thilawa Point D)

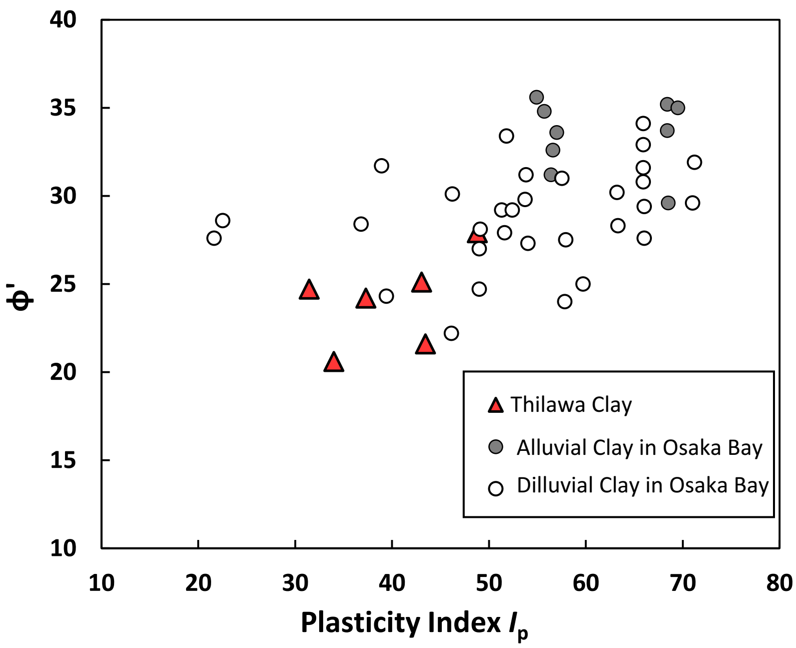


Figure 5-23 Relationship between Internal Friction Angle (φ′) and Plasticity Index (I_P) (Clays in Osaka Bay [19])

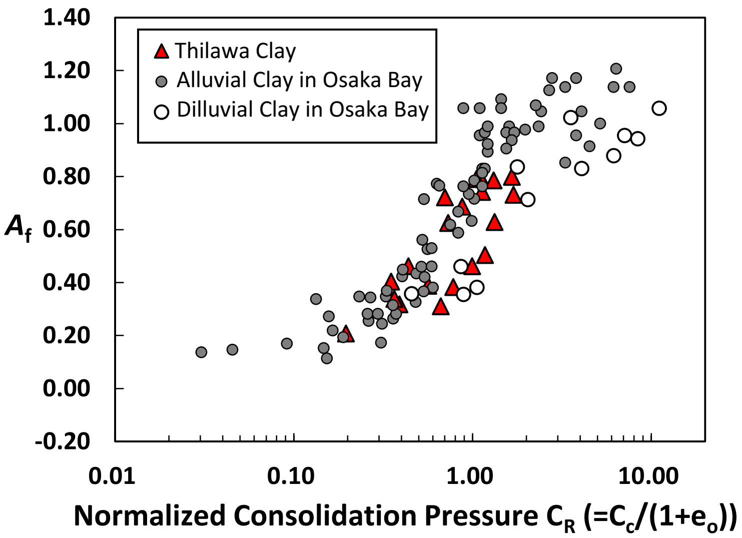


Figure 5-24 Relationship between Pore water Pressure Factor (A_f) and Normalized Consolidation Pressure (σ'_c/p_c) (Compared with Clays in Osaka Bay [19])

Figure 5-25 shows the comparison of the shear strength ratio of Thilawa clay and Tokyo Bay clay [20] obtained from the isotropic consolidated triaxial compression test (CIU). As shown in the figure, the shear strength ratio of Tokyo Bay clay is almost distributed around c_u/σ' (= c_u/p') = 0.45 for both the Alluvial and Diluvial clays with the liquid limit more than 50%. On the other hand, shear strength ratio of Thilawa clay was approximately ranging 0.30 to 0.44 and the average was 0.32 which was near the lower limit of the distribution of Tokyo Bay clay as shown in the figure.

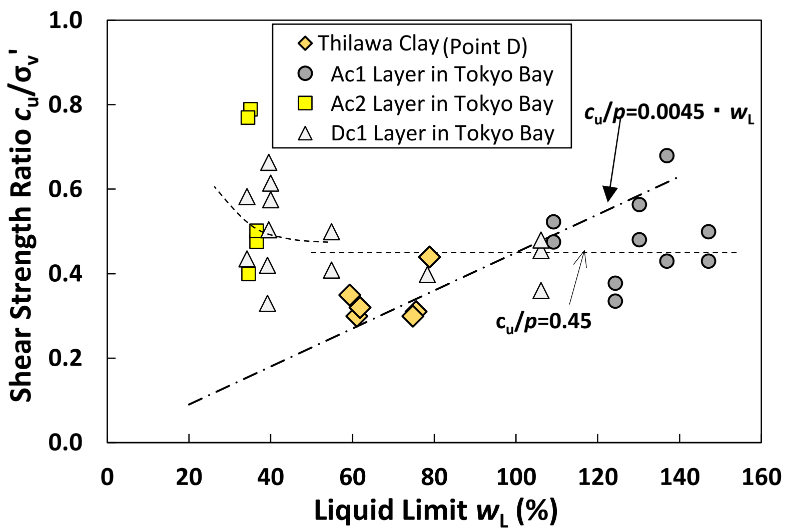


Figure 5-25 Relationship between Shear Strength Ratio (c_u/σ'_v) and Liquid Limit (w_L) (Compared with Clays in Tokyo Bay [20])

(3) Characteristics of Unconfined Compressive Strength (q_u) and Vane Shear Strength (τ_v)

The field vane shear test was conducted in the borehole at Point C out of five points (Point A to Point E) within this study area. The result is shown in Figure 5-26 together with the unconfined compression test result. Although the vane shear test results show the large shear strength in the upper portion of clay layer, the shear strength τ_v obtained from the field vane shear test and the shear strength $c_u = q_u /2$ obtained from the unconfined compression test result shows a good match each other with slight variations. The dotted line in the figure is an approximate average line obtained from the unconfined compression test result of Thilawa area.

The sensitivity ratio S_t (= shear strength in undisturbed state / shear strength after disturbance) obtained from the field vane shear test at Point C and the unconfined compression test result is shown in Figure 5-27. The sensitivity ratio obtained from the field vane shear test is distributed in the range of 2 to 7, and the sensitivity ratio obtained from the unconfined compression test is distributed in the range of 1 to 7. Therefore, sensitivity ratios obtained from both tests matched each other very well. In general, it is "sensitive" if the sensitivity ratio is 4 or more, and it is considered to be "super-sensitive" if the sensitivity ratio is 8 or more [21]. Although some results of 4 or more are also recognized, as shown in Figure 5-27, since the natural moisture content of the clay is below the liquid limit, Thilawa clay is evaluated as a "somewhat sensitive" soil rather than being a "sensitive" soil.

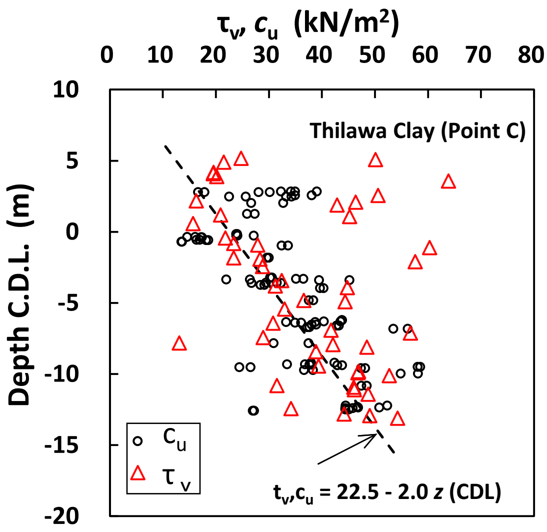


Figure 5-26 Undrained Shear Strength by Field Vane Shear Test (τ_v) and Unconfined Compression Test (c_u) with Depth (C.D.L)

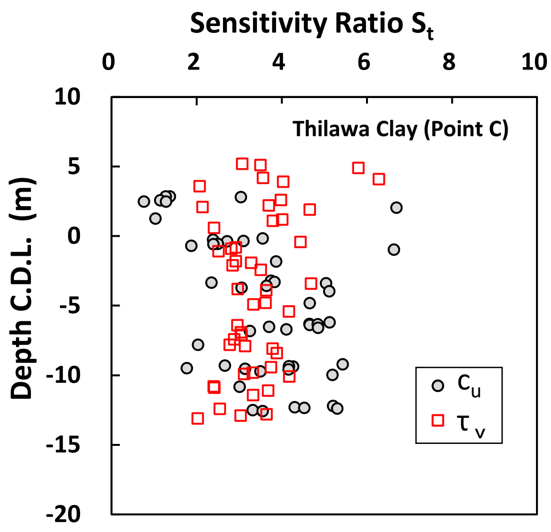


Figure 5-27 Sensitivity Ratio (St) from Field Vane Shear Test (τ_v) and Unconfined Compression Test (c_u) with Depth (C.D.L)

(4) Compression Index (C_c)

Figure 5-28 shows the e-log p curve of Thilawa clay. From the figure, when the average e-log p curve is obtained (red line in the figure), the compression index C_c is about 0.6. In addition, many researchers have studied the correlation between compression index C_c and physical properties such as liquid limit, plasticity index and void ratio. In general, it is known that the compression index C_c has a high correlation with the liquid limit w_L, and the correlation between both of the Thilawa clay is focused here.

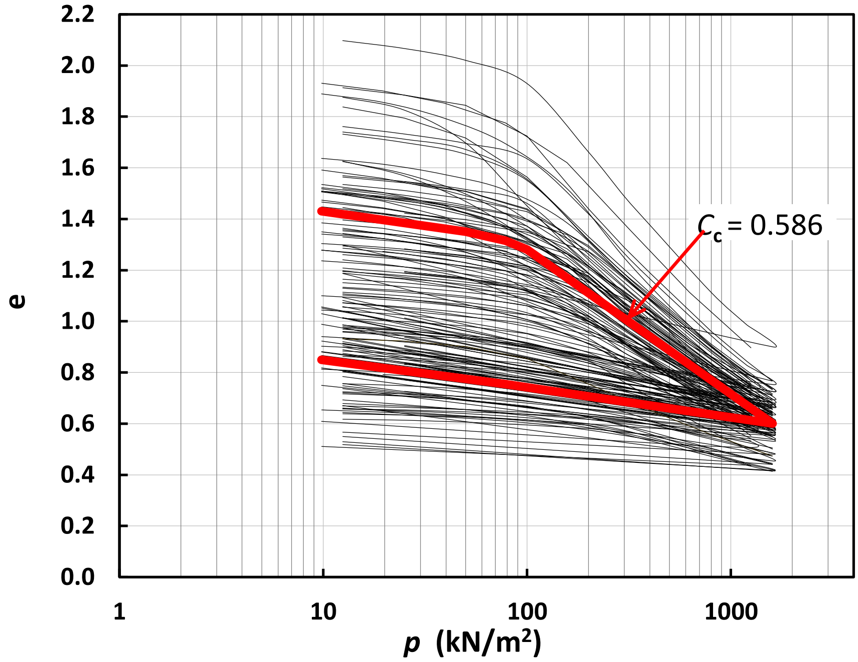


Figure 5-28 $e \sim$ Log p Curves from Consolidation Tests of Thilawa Clay

Figure 5-29 shows the relationship between the compression index C_c and the liquid limit of w_L. From the same figure, the regression line of Thilawa clay is found as follows.

$$C_c = 0.005\ (w_L + 47) \quad \text{(Thilawa clay)} \qquad (12)$$

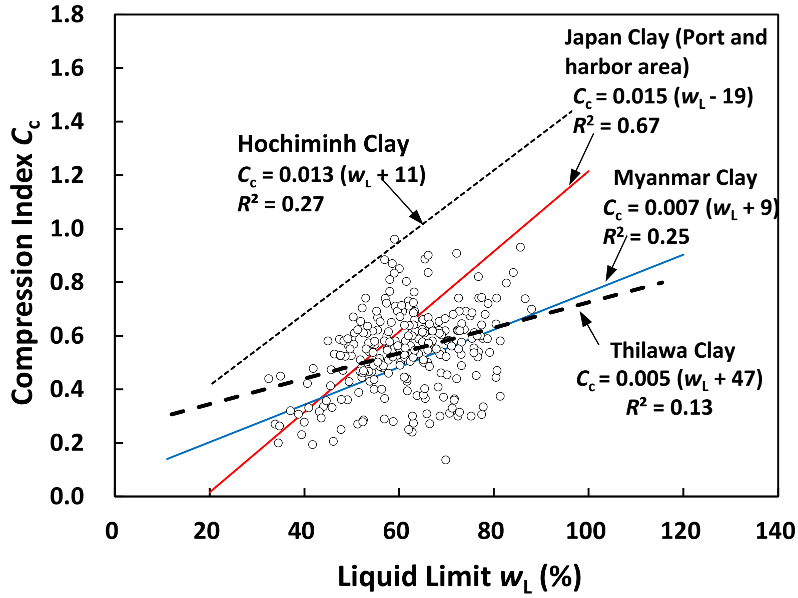


Figure 5-29 Relationship between Compression Index (C_c) and Liquid Limit (w_L)

On the other hand, the regression line concerning Myanmar clay by Murakami and Tsuchida et al. [5], Japan clay (port and harbor area) by Ogawa and Matsumoto [15] is shown in Eqs. (13), (14) and Hochiminh Clay is expressed by the formula (15).

$C_c = 0.015\ (w_L - 19)$ (Japan clay (Port and harbor area) [15]) (13)

$C_c = 0.007\ (w_L + 9)$ (Myanmar clay [5]) (14)

$C_c = 0.013\ (w_L + 11)$ (Hochiminh clay) (15)

As shown in the figure, although there is no big difference between Thilawa clay and Myanmar clay, there is a data group with a small value of C_c ranging from 0.2 to 0.3 but there is a large difference from the Japan clay. It is considered that the gradient of the regression line greatly differs due to this data group. Table 5-3 shows the regression formula between C_c and w_L shown in the past study on clay of Japan, Korea, Egypt, Brazil, Greece and the United States, Vietnam (Hochiminh), Myanmar. For the liquid limits of each soil sample used in these past studies, the greatest compression index is shown with clay in Hochiminh, South Korea South Coast, Japan, the East Coast of Korea, the West Coast of Korea and Myanmar, Egypt, Greece and United States and Brazil in that order. Thilawa clay together with Myanmar clay is located between the West Coast of Korea and Egypt.

Table 5-3 Regression Equation for C_c-w_L relation by Past Research

Area	Correlation Equation	Range of Data	Coefficient of Determination R^2	Source
Japan (Port and harbor area)	$C_c = 0.015\ (w_L-19)$	$w_L = 33.5 - 194.0\%$	0.67	Ogawa et al [15],1978
South shore area of South Korea	$C_c = 0.012\ (w_L+16.4)$	$w_L = 28.4 - 120.2\%$	0.64	Yoon et al [7],2004
East shore area of South Korea	$C_c = 0.011\ (w_L-6.4)$	$w_L = 23.0 - 107.0\%$	0.64	Yoon et al [7],2004
West shore area of South Korea	$C_c = 0.010\ (w_L-10.9)$	$w_L = 24.5 - 77.9\%$	0.67	Yoon et al [7],2004
Egypt	$C_c = 0.0063\ (w_L-10.0)$	$w_L = 10.0 - 110.0\%$	-	Abdrabbo et al [8],1990
Brazil	$C_c = 0.0046\ (w_L-9.0)$	-	-	Cozzolino et al [10],1961
Greece and the USA	$C_c = 0.006\ (w_L-9.0)$	-	0.59	Azzous et al [9],1990
Myanmar (Whole area)	$C_c = 0.007\ (w_L+8.7)$	$w_L = 26.5 - 93.5\%$	0.25	Murakami et al [5],2015
Hochiminh	$C_c = 0.013\ (w_L+11)$	$w_L = 59.7 - 95.8\%$	0.27	This study
Yangon (Whole area)	$C_c = 0.007\ (w_L+2.3)$	$w_L = 21.0 - 95.5\%$	0.27	This study
Thilawa	$C_c = 0.005\ (w_L+47)$	$w_L = 27.0 - 95.5\%$	0.13	This study

Also, it is known that the compression index shows a good correlation with the natural moisture content w_n. Figure 5-30 shows the relationship between the compression index C_c and the natural moisture content w_n. From the figure, the regression line of the compression index

C_c and the natural moisture content w_n of Thilawa clay is as follows. The coefficient of determination R^2 is 0.53, and the natural moisture content is distributed mainly in a narrow range between 30% and 60%. But there is a better correlation with the compression index C_c than the liquid limit w_L and the plasticity index I_P.

$$C_c = 0.0126\, w_n - 0.066 \qquad \text{(Thilawa clay)} \qquad (16)$$

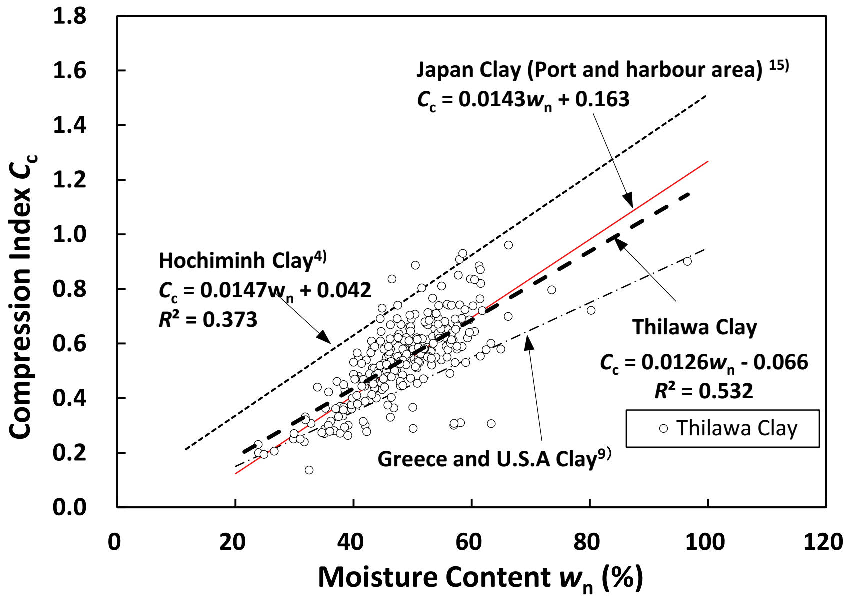


Figure 5-30 Relationship between Compression Index (C_c) and Natural Moisture Content (w_n)

Furthermore, there is an initial void ratio e_0 having a correlation with the compression index C_c. Figure 5-31 shows the relationship between the compression index C_c and the initial void ratio e_0. The followings are regression lines formula of C_c and e_0 of the past studies together with Thilawa clay.

$$C_c = 0.53\,(e_0 - 0.32) \qquad \text{(Thilawa clay)} \qquad (17)$$

$$C_c = 0.54\,(e_0 - 0.35) \qquad \text{(Clays in Japan by Nishida [22])} \qquad (18)$$

$$C_c = 0.43\,(e_0 - 0.25) \qquad \text{(Brazil clay [10])} \qquad (19)$$

$$C_c = 0.55\,(e_0 + 0.04) \qquad \text{(Hochiminh clay)} \qquad (20)$$

According to the figure, the coefficient of determination of the compression index and the void ratio of Thilawa clay has a strong correlation with $R^2 = 0.76$. It is clear that it is very similar to the regression line of clays in Japan indicated by Nishida [22].

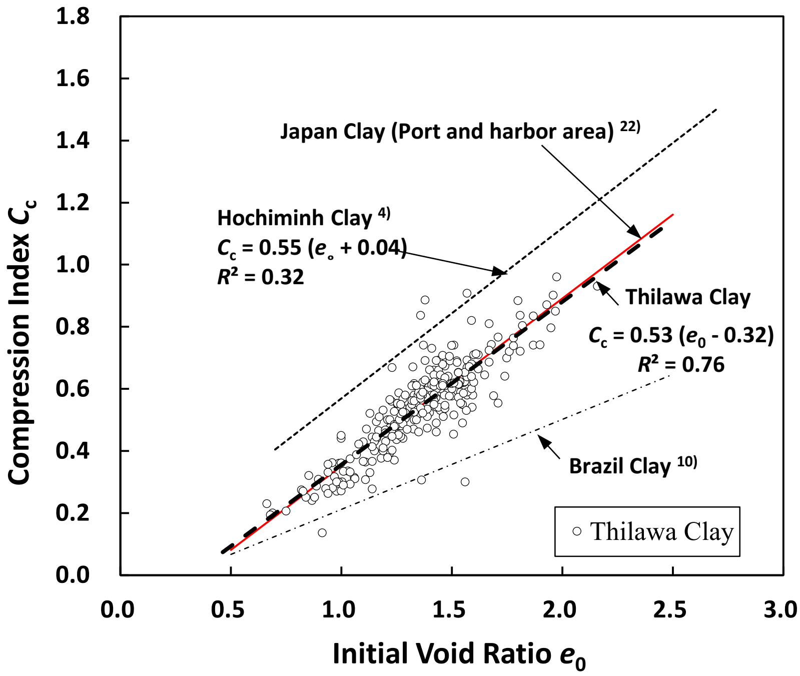


Figure 5-31 Relationship between Compression Index (C_c) and Initial Void Ratio (e_o)

Figure 5-32 shows the relationship between the compression index C_c obtained at the point A out of the five points (Point A to Point E) in the Thilawa area and the swelling index C_s (gradient of the e-log p curve at unloading during the consolidation test). According to the figure, there is a clearly good correlation between compression index C_c and swelling index C_s, and the regression line is as follows.

$$C_s = 0.25\ C_c \text{ (Thilawa clay)} \qquad (21)$$

In other words, it is known that C_s / C_c = 0.1 in Japan [15], but in the case of Thilawa clay, since C_c is small (C_c = 0.25 to 0.75), C_s is about 1/4 of the compression index C_c.

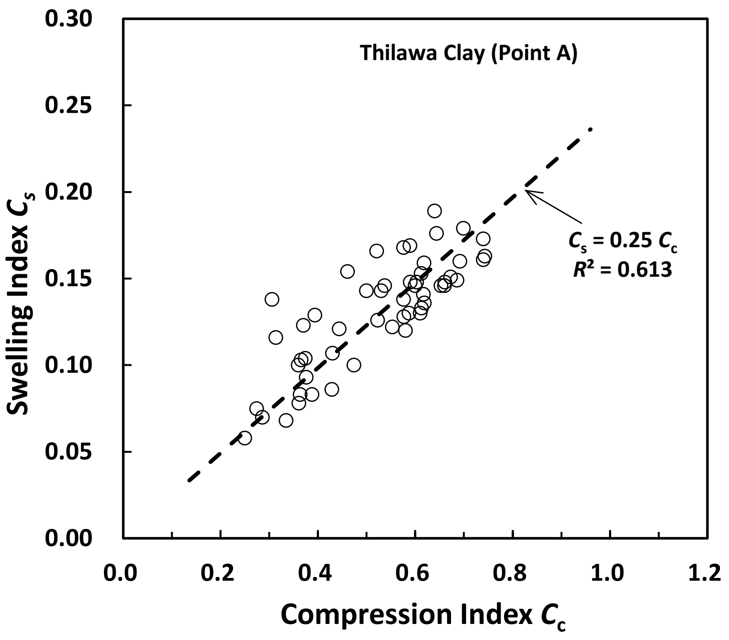


Figure 5-32 Relationship between Swelling Index (C_s) and Compression Index (C_c)

(5) Coefficient of Consolidation (c_v)

The coefficient of consolidation c_v which determines the rate of consolidation settlement is related to the coefficient of permeability of soil and is a very important property for prediction of settlement and eventually for the construction project plan. In the Thilawa area as well, the soil improvement by the Vertical Drain (PVD) method are being implemented, and determination of appropriate c_v is very important. Here, the relationship between c_v and physical properties are described.

Figure 5-33 shows the c_v-log p curves and a representative line in Thilawa area. According to this figure, the typical coefficient of consolidation c_v in the normal consolidation state in Thilawa area is about 50 cm^2/day (red line in the figure) on average. Comparing this with the coefficient of consolidation of clay in Japan and other countries [23)], as shown in Table 5-4, the coefficient of consolidation of Thilawa clay is larger than that of clays in Singapore and Chi Vai (southern part of Vietnam) and smaller than Minami Honmoku and Ariake clay in Japan, and almost similar value as clays in Hai Phong (northern Vietnam).

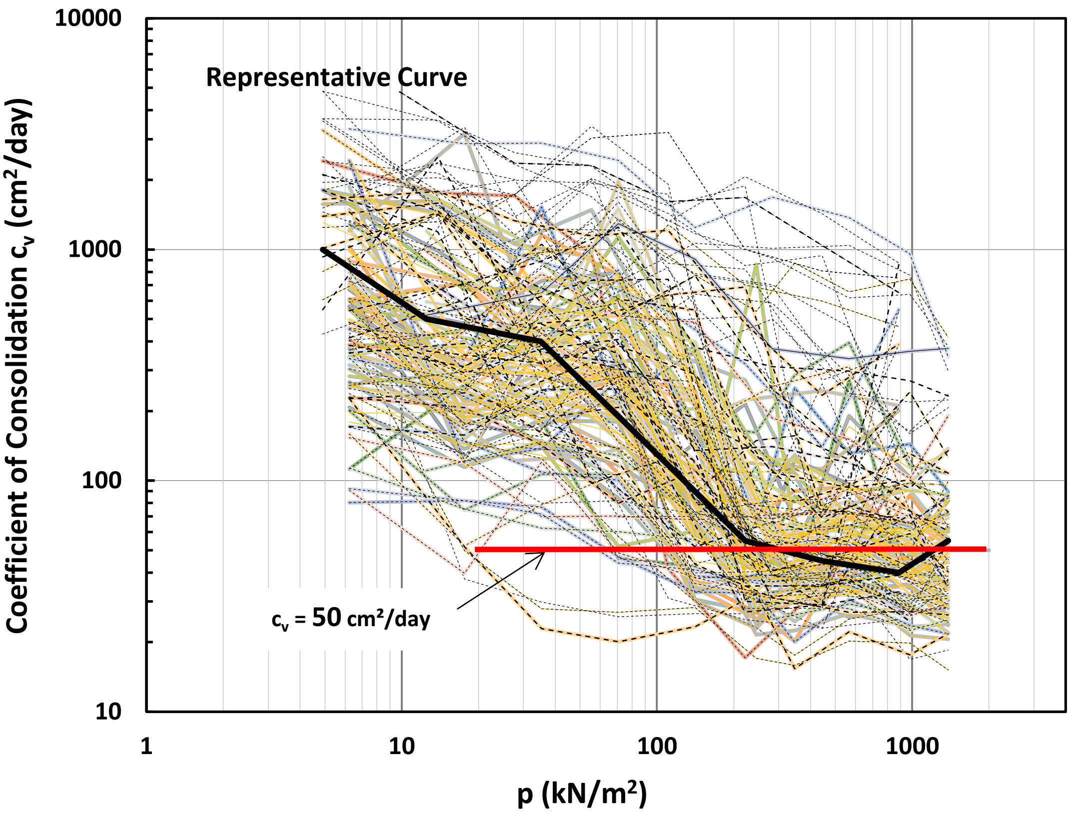


Figure 5-33 c_v – Log p Curves of Thilawa Clay

Table 5-4 Average Coefficient of Consolidation (c_v) of Thilawa Clay and Other Areas (Normally Consolidated State) (Prepared from Reference [23)])

Area	Silt (%)	Clay (%)	Liquid Limit w_L (%)	Coefficient of Consolidation c_v (cm²/day)
Minami Honmoku, Yokohama	44.1	50.5	118	53
Osaka South Port Clay, Japan	40.7	58.2	109	14
Yamashita Park, Yokohama	40.4	54.7	119	48
Hachirogata Clay, Niigata	36.2	63.4	176	60
Ariake Clay, Soga	32.0	68.0	107	40
Bangkok Clay	20.9	77.0	89	6.0
Singapore Clay	28.8	70.0	85	15
Thilawa Clay	49.4	49.1	60	50
Thi Vai Clay	21.5	78.5	82	12
Hai Phong Clay	53.0	38.7	62	50

The relationship between the mean value of coefficient of consolidation c_v under normally consolidated state obtained by the standard consolidation test and the liquid limit w_L is shown in Figure 5-34. As shown in the figure, Thilawa clay has a much lower coefficient of consolidation at the same liquid limit than Japan's marine clay. The regression line formulas of

c_v (cm^2/day) and w_L (%) of Thilawa clay, Myanmar clay, Japan clay, Hochiminh clay are shown as below.

$$c_v = 406 \cdot 10^{-1.22\,(w_L/100)} \qquad \text{(Thilawa clay)} \qquad (22)$$

$$c_v = 2{,}240 \cdot 10^{-2.34\,(w_L/100)} \qquad \text{(Myanmar clay [5])} \qquad (23)$$

$$c_v = 3{,}000 \cdot 10^{-1.59\,(w_L/100)} \qquad \text{(Japan clay, port and harbor area [15])} \qquad (24)$$

$$c_v = 1{,}886 \cdot 10^{-1.61(w_L/100)} \qquad \text{(Hochiminh clay)} \qquad (25)$$

Coefficient of consolidation c_v in normally consolidated state of Thilawa clay is 1/6 when the liquid limit w_L is 40%, 1/4 when w_L is 60%, and about 1/3 when w_L 80% when compared with the Japan clay (port and harbor area). And the time required for the prescribed consolidation will be three to four times longer than that of clay in Japan.

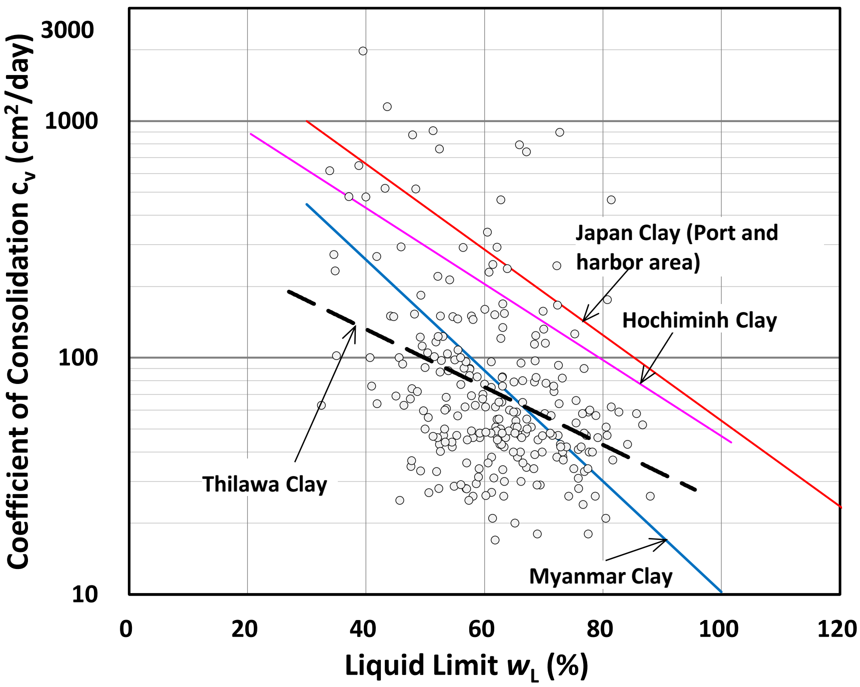


Figure 5-34 Relationship between Coefficient of Consolidation (c_v) and Liquid Limit (w_L)

Figure 5-35 shows the relationship between the coefficient of consolidation c_v under the normally consolidated state and the natural moisture content w_n. The natural moisture content is one of the physical properties of clay which can be measured easily. The following formula was obtained as a regression line of Thilawa clay although the variation was large (coefficient of determination $R^2 = 0.19$).

$$c_v = 517 * 10^{-1.78\,(wn/100)} \quad \text{(Thilawa clay)} \qquad (26)$$

As mentioned above, it can be said that the coefficient of consolidation of clay in Japan is higher than that of clay such as in Myanmar, Vietnam, and Singapore in this comparison.

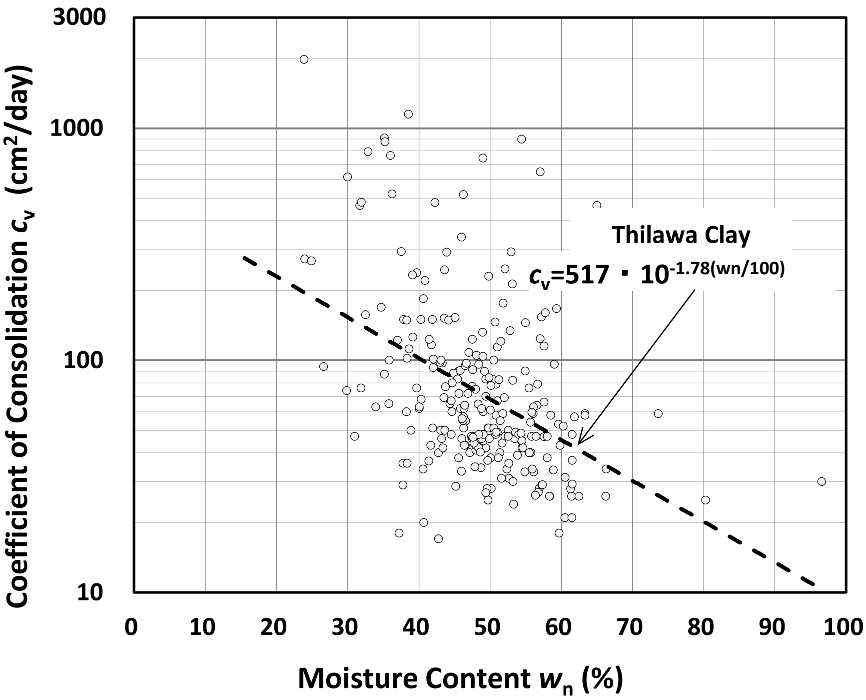


Figure 5-35 Relationship between Coefficient of Consolidation (c_v) and Natural Moisture Content (w_n)

Figure 5-36 is a comparison of the clay content and the coefficient of consolidation c_v for the clay shown in Table 4-4. As shown in the figure, when the clay content increases, the coefficient of consolidation tends to decrease. From the data of Ogawa and Matsumoto [15] summarizing the clays in Japanese port and harbor area, the average and standard deviation of the clay content in Japan were calculated to be 36.7% on average and 52.1% in average standard deviation. The coefficient of consolidation of Japan clay is larger than other areas. (The clay content of Japan clay is less than that of other areas.) However, it is necessary to gather data for further study to clarify it more.

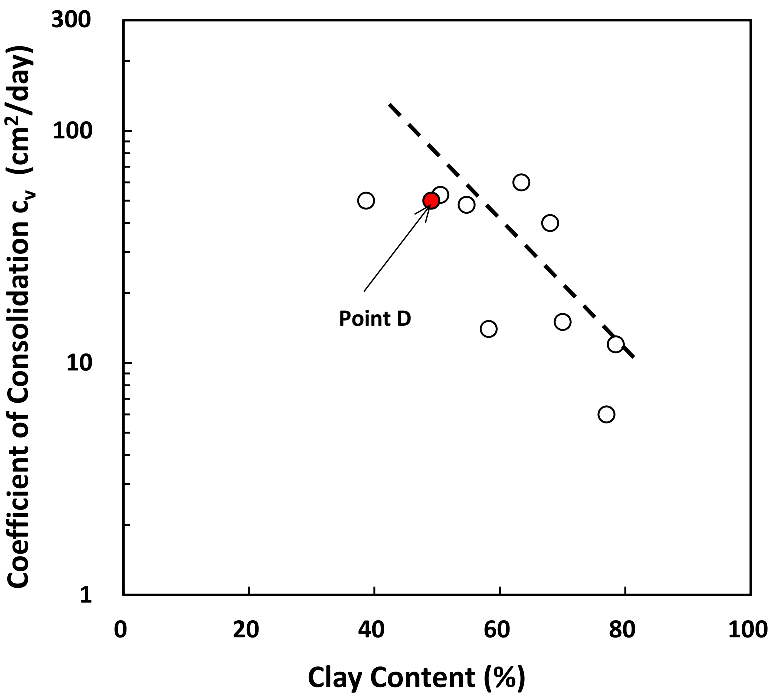


Figure 5-36 Relationship between Coefficient of Consolidation (c_v) and Clay Content

(6) Coefficient of Secondary Consolidation ($C_{\alpha\varepsilon}$)

In construction work on soft clay ground, how to suppress the residual settlement of the ground is one of the most important issue. In overseas construction, when the residual settlement amount including the secondary consolidation settlement is specified in the technical specifications. And there are some cases where the height of the ground raising was determined by the residual settlement which should be within the allowable range. Accordingly, an appropriate coefficient of secondary consolidation is essentially required to accurately predict secondary consolidation settlement and residual consolidation settlement for the practical projects.

Coefficient of secondary consolidation $C_{\alpha\varepsilon}$ (coefficient of secondary consolidation with respect to strain) appears near the end of primary consolidation which is a constant gradient (= $\Delta\varepsilon$ / Δlog t) of the straight-line portion of the curve of ε~log t. Coefficient of secondary consolidation $C_{\alpha\varepsilon}$ obtained from the standard consolidation test conducted at Point A out of 5 points (Point A to Point E) in Thilawa and the $C_{\alpha\varepsilon}$ obtained from the long-term consolidation test conducted at Point D were compared and the secondary consolidation characteristics were studied here.

Figure 5-37 shows the relationship between the coefficient of secondary consolidation $C_{\alpha\varepsilon}$ of the Thilawa clay (Point A) and the initial void ratio e_0. As the initial void ratio becomes larger, the coefficient of secondary consolidation tends to become larger. The regression line of them can be expressed as follows.

$$C_{\alpha\varepsilon} = 0.0045e_0 + 0.003 \text{ (coefficient of determination } R^2 = 0.57) \qquad (27)$$

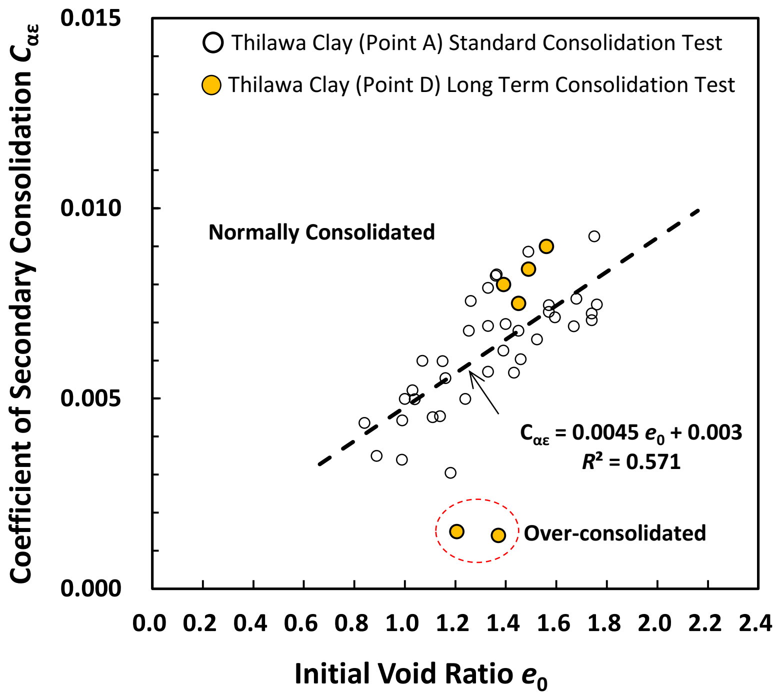


Figure 5-37 Relationship between Coefficient of Secondary Consolidation ($C_{\alpha\varepsilon}$) and Initial Void Ratio (e_o) of Thilawa Clay

In addition, Figure 5-38 shows the relationship between the coefficient of secondary consolidation $C_{\alpha\varepsilon}$ and the compression index C_c of the Thilawa clay (Point A). As the compression index C_c increases, the coefficient of secondary consolidation $C_{\alpha\varepsilon}$ tends to increase, and the regression line of them can be expressed as follows.

$$C_{\alpha\varepsilon} = 0.012\, C_c \qquad \text{(coefficient of determination } R^2 = 0.42) \qquad (28)$$

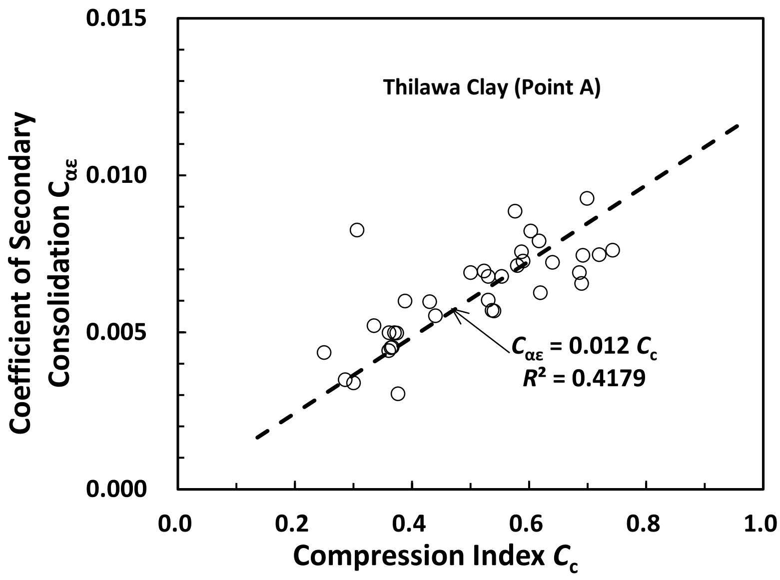


Figure 5-38 Relationship between Coefficient of Secondary Consolidation ($C_{\alpha\varepsilon}$) and Compression Index (C_c) of Thilawa Clay

Furthermore, Figure 5-39 shows the relationship between the coefficient of secondary consolidation $C_{\alpha\varepsilon}$ of Thilawa clay (point A) and the over-consolidation ratio OCR. Although the coefficient of secondary consolidation sharply decreases when the OCR is around 1 to 2, $C_{\alpha\varepsilon}$ gradually approaches a constant value after passing OCR= 2. However, the number of samples is not enough so that further study is necessary to consider about this trend based on sufficient data.

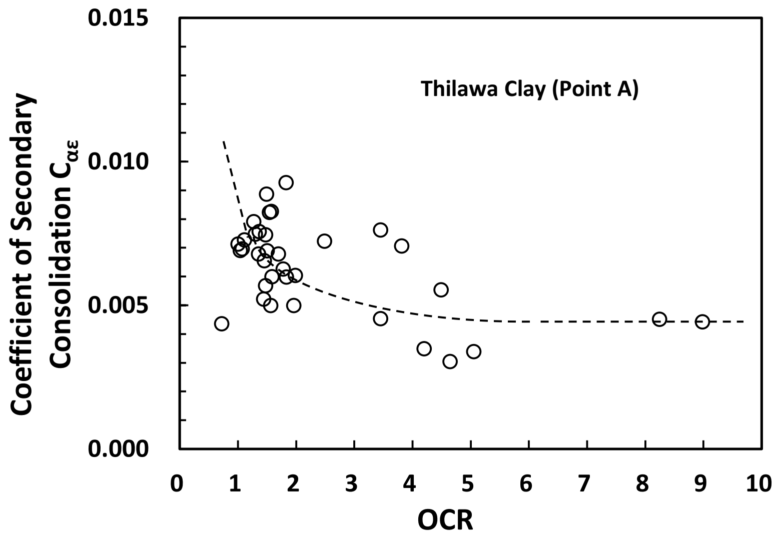


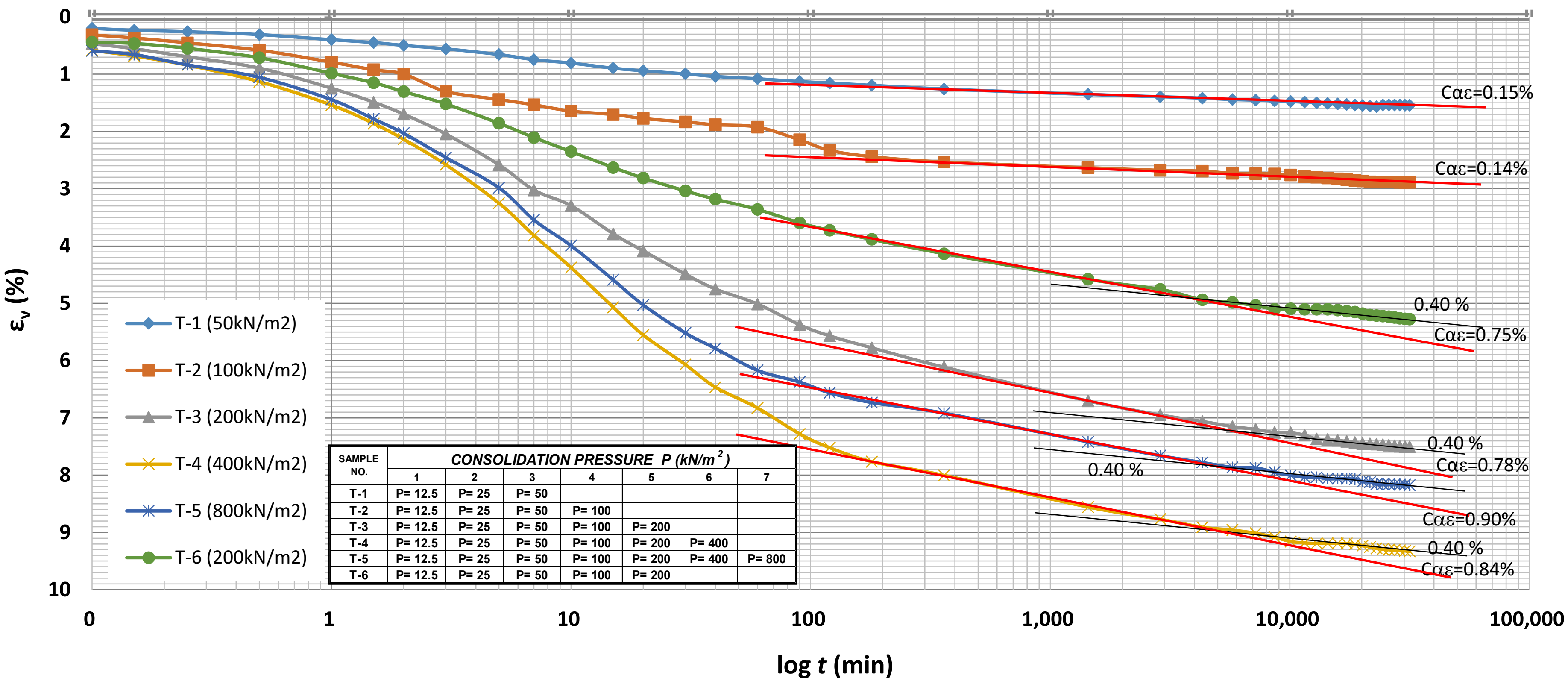
Figure 5-39 Relationship between Coefficient of Secondary Consolidation ($C_{\alpha\varepsilon}$) and Over-consolidation Ratio (OCR) of Thilawa Clay

Figure 5-40 shows the results of long-term consolidation tests carried out on samples taken at Point D in Thilawa area using a standard consolidation test equipment. In order to ascertain the difference of the coefficient of secondary consolidation due to the difference in the load level, six samples with different depths were step-wise consolidated with a load ratio of 2 times and the load shown in Figure 5-40 was applied for 3 weeks (30,240 minutes). It was continuously loaded and the settlement was measured. The coefficient of secondary consolidation obtained from the long-term consolidation test result shows the value of $C_{\alpha\varepsilon}$ = 0.75 to 0.90% (average value = 0.82%) in the strain indication under the normally consolidated state. On the other hand, for the over-consolidated state, $C_{\alpha\varepsilon}$ is ranging from 0.14 to 0.15% and about 1/5 of the normal consolidation state. Although it is not recognized clearly in the over-consolidated state, the gradient of the curve clearly shows a loose slope from the gradient after 10,000 minutes (about 7 days) under the normal consolidation state, and its slope $C_{\alpha\varepsilon}$ is about 0.40%. Compared to the coefficient of secondary consolidation ($C_{\alpha\varepsilon}$) obtained from the settlement after the end of primary consolidation in the standard consolidation test at Point A and $C_{\alpha\varepsilon}$ obtained from long-term consolidation test, both plots are distributed as shown in Figure 5-37. obtained from long term consolidation test at Point D are plotted in the vicinity of upper limit values within the distribution of values obtained from the standard consolidation test at Point A.

The following formula has been proposed for the coefficient of secondary consolidation of inorganic clay (Mesri [24)]).

$$C_{\alpha\varepsilon}/C_R = 0.04 \pm 0.01 \quad \text{(inorganic soil)} \qquad (29)$$

C_R is the compression ratio defined by C_c / (1 + e_0). For the three samples whose C_c is known among the test results obtained by the present long-term consolidation test, it was confirmed by Figure 5-41 whether the above relation holds or not. As stated above, $C_{\alpha\varepsilon}$ is 0.01 (= 1%) under normal consolidation state, then $C_{\alpha\varepsilon}/C_R$ is 0.04 under normal consolidation state. It is almost same value such as average value of $C_{\alpha\varepsilon}/C_R$ = 0.035 of Hiroshima clay [17)]. And the relation of the formula (29) is considered to roughly match with Thilawa clay.



SAMPLE NO.	CONSOLIDATION PRESSURE P (kN/m^2)						
	1	2	3	4	5	6	7
T-1	P= 12.5	P= 25	P= 50				
T-2	P= 12.5	P= 25	P= 50	P= 100			
T-3	P= 12.5	P= 25	P= 50	P= 100	P= 200		
T-4	P= 12.5	P= 25	P= 50	P= 100	P= 200	P= 400	
T-5	P= 12.5	P= 25	P= 50	P= 100	P= 200	P= 400	P= 800
T-6	P= 12.5	P= 25	P= 50	P= 100	P= 200		

Figure 5-40 Settlement Strain (ε_v) – Time (log *t*) Curve by Long Term Consolidation Test of Thilawa Clay (Point D)

In the long-term consolidation test, the gradient from 1,440 minutes (1 day) onwards to 10,000 minutes (about 6 days) is slightly gentle than the gradient from the end of primary consolidation (20 to 30 minutes after loading) to 1,440 minutes (1 day) at the end of one stage loading. The gradient after 10,000 minutes has changed drastically to about half of the previous gradient. In the case of predicting secondary consolidation settlement in practice, it might be the secondary consolidation gradient at the time of 30 to 40 years later after construction. Therefore, there is a possibility to underestimate the total secondary consolidation settlement in that gradient. So, the gradient after 10,000 minutes was not adopted as the coefficient of secondary consolidation in this study.

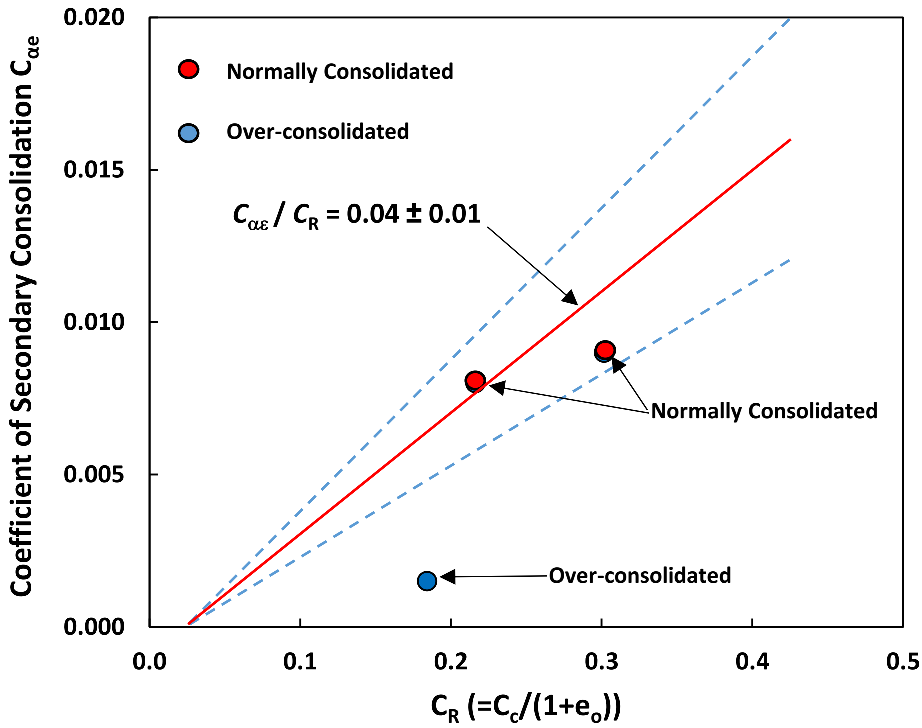


Figure 5-41 Relationship between Coefficient of Secondary Consolidation and Compression Ratio (C_R) of Thilawa Clay (Point D)

5.5 Conclusion of this Chapter

The results of soil test conducted using undisturbed samples taken by the same sampling method as in Japan were analyzed, and the characteristics of soft clay in Thilawa area along the Yangon River were identified and compared with soil properties of clays in Japan and past studies. The results are summarized as follows.

1) The over-consolidation ratio of Thilawa clay is larger than that of Japan clay (port and harbor area). Japan clay has an average over-consolidation ratio of 1.25, but Thilawa clay has an average of about 1.4 when excluding a layer with a large over-consolidation ratio at depths of 5 to 6 meters from the ground surface (equal to shallower than C.D.L. ± 0 m). Even at the same altitude in the river with on-land, the over-consolidation ratio varied in the range of 1 to 6, which was larger than that on-land area. This is probably because the clay of the riverbed has been eroded by river flow and the current overburden pressure is smaller than on-land area.
2) The unconfined compressive strength q_u of Thilawa clay can be expressed by the following formula for both on land and in the river.

$$q_u \text{ (kN/m}^2\text{)} = 45 - 4.0\, z \; (z : \text{C.D.L. m})$$

3) In addition, from the regression line formula qu = 0.445 p_c obtained from the unconfined compressive strength q_u and the consolidation yield stress p_c obtained from standard consolidation test. The normalized shear strength c_u/p_c (equivalent to shear strength ratio: cu/p') of Thilawa clay was about 0.22. This is consistent with the indication by Mesri (1973) [24]. In addition, this is about 2/3 of the value of 0.30 for marine clay in Japan, and therefore, the attention is required when considering the strength obtained by the preloading method or etc. In addition, the normalized shear strength (c_u/p_c) was widely distributed, and no correlation tendency was found in relation to I_P and w_n. Further investigation is required to clarify whether this is due to individuality unique to clay or other factors.

4) The friction angle of the effective stress of Thilawa clay obtained from the triaxial compression test (CIU) is $\phi' = 25°$ and it is smaller than the general value of Japanese marine clays $\phi' = 30°$.
5) The in-situ shear strength τ_v by the field vane shear test in the borehole of Thilawa clay and the undrained shear strength c_u (= $q_u/2$) by the unconfined compression test showed good match each other. The sensitivity ratio S_t of Thilawa clay obtained from those undrained shear strengths was distributed in the range of 1 to 6.
6) Soil properties which shows good correlation with the compression index C_c of Thilawa clay are natural moisture content w_n and the initial void ratio e_0. The following regression formulas were obtained, respectively.

$$C_c = 0.013\, w_n + 0.066 \qquad (R^2=0.53)$$
$$C_c = 0.53\, (e_0 - 0.32) \qquad (R^2=0.76)$$

7) The average coefficient of consolidation (c_v) of Thilawa clay is about 50 cm^2/day under normally consolidated state. The c_v of Thilawa clay are smaller compared to Japanese marine clay under same liquid limit value. However, the average c_v of Thilawa clay and Japanese marine clay are larger than Bangkok clay, Singapore clay and Thi Vai clay (Vietnam). This result suggests that the c_v of Japanese marine clay and Thilawa clay are larger than that of other regions in Asia, and the lower clay content of Japanese marine clay and Thilawa clay possibly be one reason of this.
8) The parameters that have good correlation with the coefficient of secondary consolidation ($C_{\alpha\varepsilon}$) of Thilawa clay are the initial void ratio e_0 and the compression index C_c, and the following regression formulas were obtained, respectively.

$$C_{\alpha\varepsilon} = 0.0045 e_0 + 0.003 \qquad (R^2=0.57)$$
$$C_{\alpha\varepsilon} = 0.012\, C_c \qquad (R^2=0.42)$$

9) In the long-term consolidation test, the coefficient of secondary consolidation ($C_{\alpha\varepsilon}$) under the load in normal consolidated state was about 0.8%. And it becomes smaller with the gradient of about 1/2 apparently from around 10,000 minutes passed. The further study is necessary to clarify how to practically evaluate it.

The correlation formula of the soil parameters shown here is for comparison purpose only. When the formula are used, it is necessary to pay attention to the correlation factors.

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6. CONCLUSION

6.1 Summary of Results

This thesis is the first comprehensive study result in Myanmar on geotechnical engineering characteristics of cohesive soils in Yangon. The results of this Study is summarized as follows.

As for the effects of sample disturbance due to different sampling methods with regard to the quality of undisturbed samples of cohesive soils in Yangon,

(1) The effects of disturbances by using different types of samplers between fixed piston and Shelby tube sampler are recognized in Yangon clay. Even though the differences of degree of disturbance is not so much clear in some of Yangon clays except Thilawa clay, it can be recommended that, for the accurate evaluation of geotechnical data, the fixed piston sampler should be used for taking undisturbed samples of cohesive soils.
(2) As a result, as for the Thilawa clay, it was recognized clearly that the effect of disturbance was smaller incase of fixed piston sampler than Shelby tube sampler. Therefore, in this study, samples taken only with a fixed piston sampler were used.
(3) There existed some differences in the unconfined compressive strength q_u and the compression index C_c of Yangon clay due to the different sampling methods. However, the differences were dependent on the stress-strain property of clay and were not so large as in the more sensitive Thi Vai clays in Vietnam or Hachirogata clays in Japan.

As for the Geotechnical Characteristics of Soft to Firm Clays in Yangon Area,

(1) Based on the soil tests of samples taken by boring works, the soil profiles (cross sections) which covers Yangon area were presented. According to those soil profiles, it was identified that the thickness of the soft to firm clay layers in Yangon ranged from 5 to 25 meters. It was also found that the firm clay of the upper layer of the Tertiary deposit located at the Central Sub-area in Yangon includes lateritic clay, which showed physical and mechanical properties different from other sub-areas in Yangon.
(2) Significant differences of the physical and the mechanical properties were not found between two sedimentary basins (Irrawaddy Delta Sub-basin and Pegu-Yoma Sittaung Basin) and among seven sub-areas in Yangon except for the Central Sub-area.

(3) Yangon clay is the mostly over-consolidated clay. The over-consolidation ratio (OCR) of Yangon clays, which are at depths of shallower than 5 to 6 meters, and above the ground water level, is between 5 and more than 10. For the clays at depths of deeper than 5 to 6 meters, the OCR is ranges from 1.0 to 3.0, with an average of 1.6.

(4) For most clays with depths of shallower than 5 to 6 meters and above ground water level, the unconfined compressive strength is more than 100kN/m2. Although large variation is shown, the unconfined compressive strength (q_u) of clays in Yangon increases with the depth deeper than 5 to 6 meters from ground surface.

(5) Comparing the q_u to the p_c, the consolidation yield stress, q_u of the Yangon clay was 60 % of Japanese marine clays [1)] under the same value of p_c. The strength reduction by swelling due to over-consolidation effect and the strength reduction due to disturbances during sampling and testing procedures might be ones of the reasons for having such a small normalized shear strength.

(6) A good correlation between the compression index, C_c, and the natural moisture content w_n, was found in the Yangon clay. Comparing the C_c value at the median value of natural moisture content of 40 % of Yangon clay, the C_c of Yangon clay is 5/9 that of Japanese marine clay, and 2/3 that of Thi Vai clay in Vietnam.

(7) Under the same liquid limit (w_L), the c_v of Yangon clay is smaller than those of Japanese clays (port and harbor area [1)]) and the Hochiminh clay in Vietnam, and the time required for Yangon clay to complete the required degree of consolidation is from 2 to 4 times that of Japanese marine clays and from 1.3 to 2 times that of the Hochiminh clays.

As for the Geotechnical Characteristics of Soft Clays in Thilawa Area,

(1) The over-consolidation ratio of Thilawa clay is larger than that of Japan clay (port and harbor area [1)]). Japanese marine clay has an average over-consolidation ratio of 1.25, but Thilawa clay has an average of about 1.4 when excluding a layer with a large over-consolidation ratio at depths of 5 to 6 meters from the ground surface (equal to shallower than C.D.L. ± 0 m).

(2) The unconfined compressive strength q_u of Thilawa clay can be expressed by the following formula for both on land and in the river.

$$q_u\,(\mathrm{kN/m2}) = 45 - 4.0\,z \quad (z : \mathrm{C.D.L.\ m})$$

(3) In addition, from the regression line formula "$q_u = 0.445\ p_c$" obtained from the unconfined compressive strength q_u and the consolidation yield stress p_c obtained from

standard consolidation test. The normalized shear strength c_u/p_c (equivalent to shear strength ratio: c_u/p') of Thilawa clay was about 0.22. This is consistent with the indication by Mesri (1973) [2)]. In addition, this is about 2/3 of the value of 0.30 for marine clay in Japan, and therefore, the attention is required when considering the strength obtained of the clay by the preloading method or etc.

(4) The friction angle of the effective stress of Thilawa clay obtained from the triaxial compression test (CIU) is ϕ '= 25 ° and it is a little smaller than the general value of Japanese marine clays ϕ'= 30 °.

(5) The in-situ shear strength τ_v by the field vane shear test in the borehole of Thilawa clay and the undrained shear strength c_u (= q_u /2) by the unconfined compression test showed good match each other. The sensitivity ratio S_t of Thilawa clay obtained from those undrained shear strengths was distributed in the range of 1 to 6.

(6) The average coefficient of consolidation (c_v) of Thilawa clay is about 50 cm^2 /day under normally consolidated state. The c_v of Thilawa clay are smaller compared to Japanese marine clay under same liquid limit value. However, the average c_v of Thilawa clay and Japanese marine clay are larger than Bangkok clay, Singapore clay and Thi Vai clay (Vietnam). This result suggests that the c_v of Japanese marine clay and Thilawa clay are larger than that of other regions in Asia, and the lower clay content of Japanese marine clay and Thilawa clay possibly be one reason of this.

(7) In the long-term consolidation test, the coefficient of secondary consolidation ($C_{\alpha\varepsilon}$) under the load in normal consolidated state was about 0.8%.

6.2 Future Issues

Although some results have been obtained by this study, the issues remaining in the future will be summarized as below.

1) Issues on the effects on sample disturbance due to different sampling methods with regard to the quality of undisturbed samples of cohesive soils in Yangon

In this study, only three locations of samples could be taken at Thilawa, North Dagon and Twantay area, and some influence can be observed due to different type of samplers, fixed piston and Shelby tube sampler. Though the differences of quality of undisturbed samples are clearly observed for Thilawa samples only, but significant differences could not be found with other samples. Accordingly, more data to compare quality of undisturbed samples between two samplers should be collected and analyzed to clarify the differences more for Yangon clays.

2) Issues on Geotechnical Characteristics of Soft to Firm Clays in Yangon Area

In this study, the soil properties of soft to firm clays obtained in Yangon area including Thilawa area is summarized and analyzed, then some points on soil properties are identified and clarified. However, numbers of data obtained in some sub-areas especially in northern sub-areas are still a few to evaluate the characteristics of the soft to firm clays in sub-areas. Accordingly, more data of Yangon area should be collected and analyzed for more accurate evaluation of soil properties in Yangon area. It should be updated when new data is available.

3) Issues on Geotechnical Characteristics of Soft Clays in Thilawa Area

In this study, the soil properties of soft clays obtained in Thilawa area is summarized and analyzed, then several points on geotechnical properties are identified and clarified. However, numbers of data obtained is still a few to evaluate the characteristics of the soft clays in Thilawa area. Accordingly, more data of Thilawa area should be collected and analyzed for more accurate evaluation of geotechnical properties in Thilawa area.

The correlation formula of the soil parameters shown here is for comparison purpose only, when the formula is used, it is necessary to pay attention to the correlation factors.

4) Others

In order to manage the projects smoothly from beginning of the planning to design and construction stage, soil data for project areas is very important for investors and the engineers concerned. Especially, at the initial stage of the project such as feasibility study stage or planning stage, soil data is very useful to estimate the project cost and its feasibility of the project. Therefore, data sharing of soil conditions including laboratory test results are very important to reduce the geotechnical risks, and useful and beneficially for not only the engineers concerned but also the investors and the government sectors concerned issuing construction permissions. Accordingly, we need to consider how to share and utilize the precious existing soil data some of that are protected with Non-disclosure agreement (NDA) without damaging any client’s interests and profits.

Reference

1) Ogawa, F. & Matsumoto, K. (1978): Correlation of Mechanical and Index Properties of Soils in Harbor Districts, Report of the Port and Airport Research Institute, 017-03-01 (in Japanese). https://www.pari.go.jp/en/report_search/detail.php?id=1978090170301
2) Mesri, G. (1973): Coefficient of Secondary Compression, Proc. ASCE, 99, SM1, pp.123-137, 1973.

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APPENDICES

APPENDIX “A”

DISTRIBUTION GRAPHS WITH DEPTH FOR PHYSICAL AND MECHANICAL PROPERTIES OF YANGON CLAYS

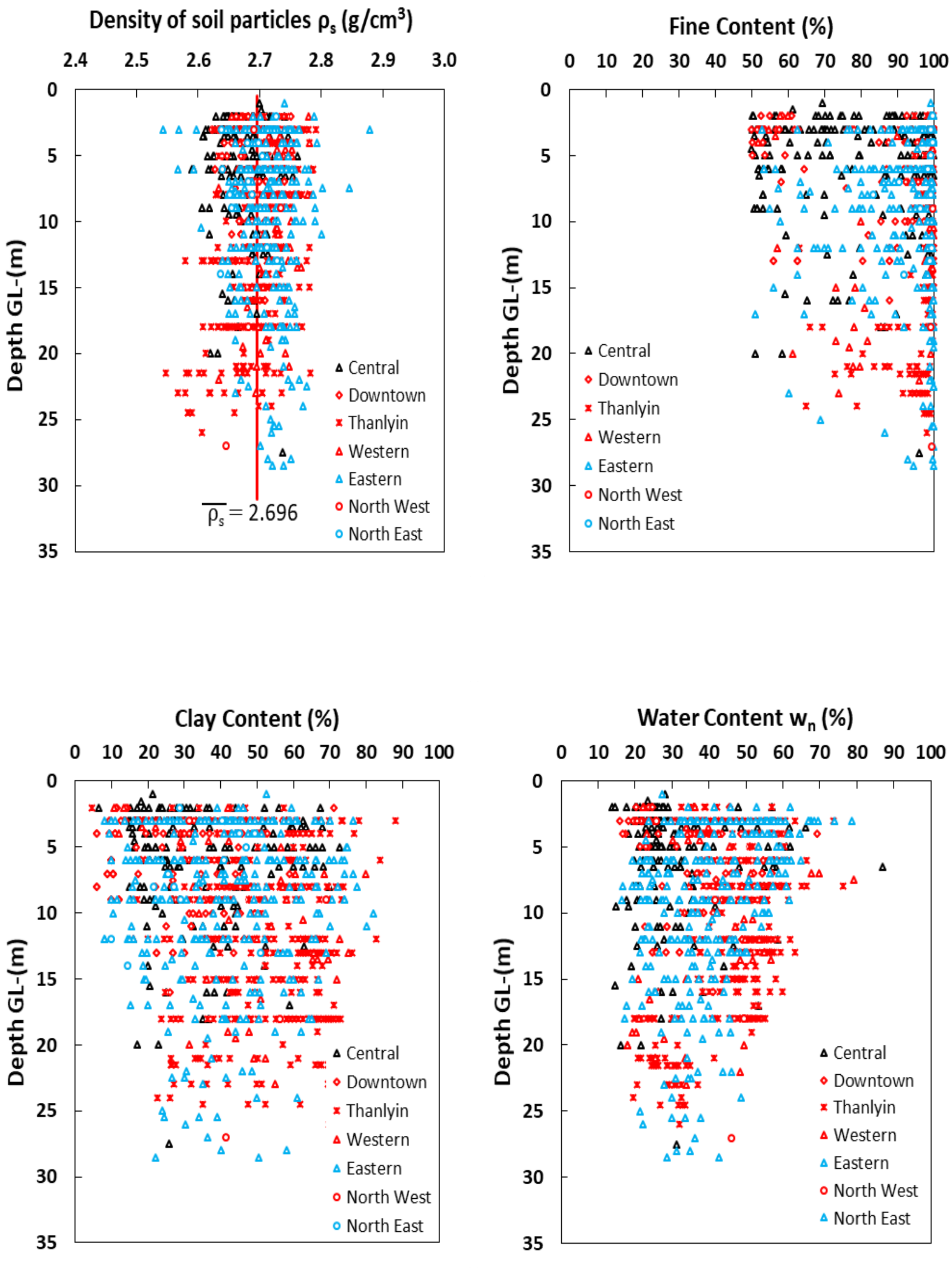
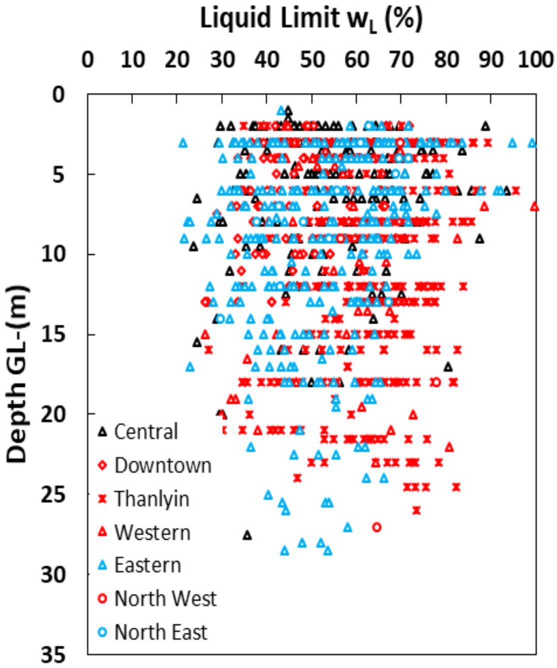
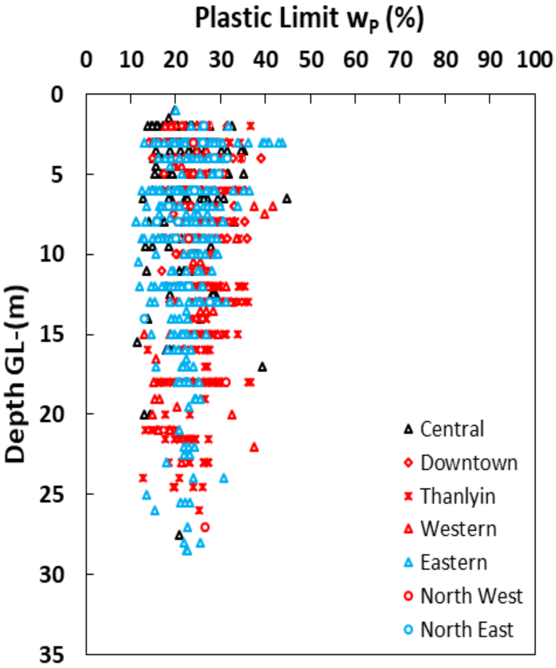
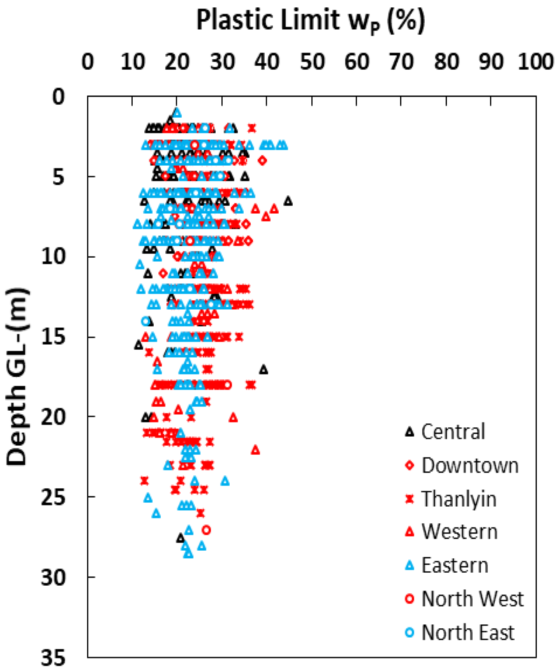


Figure 1 Physical Properties of Yangon Sub-areas Graphs







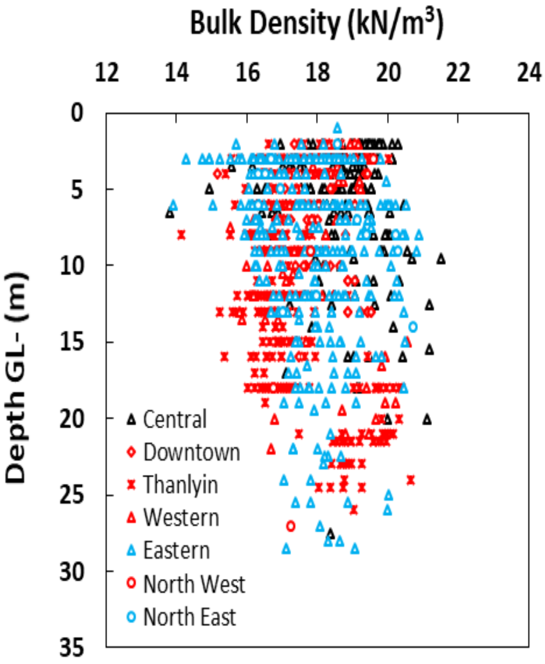


Figure 1 Physical Properties of Yangon Sub-areas Graphs

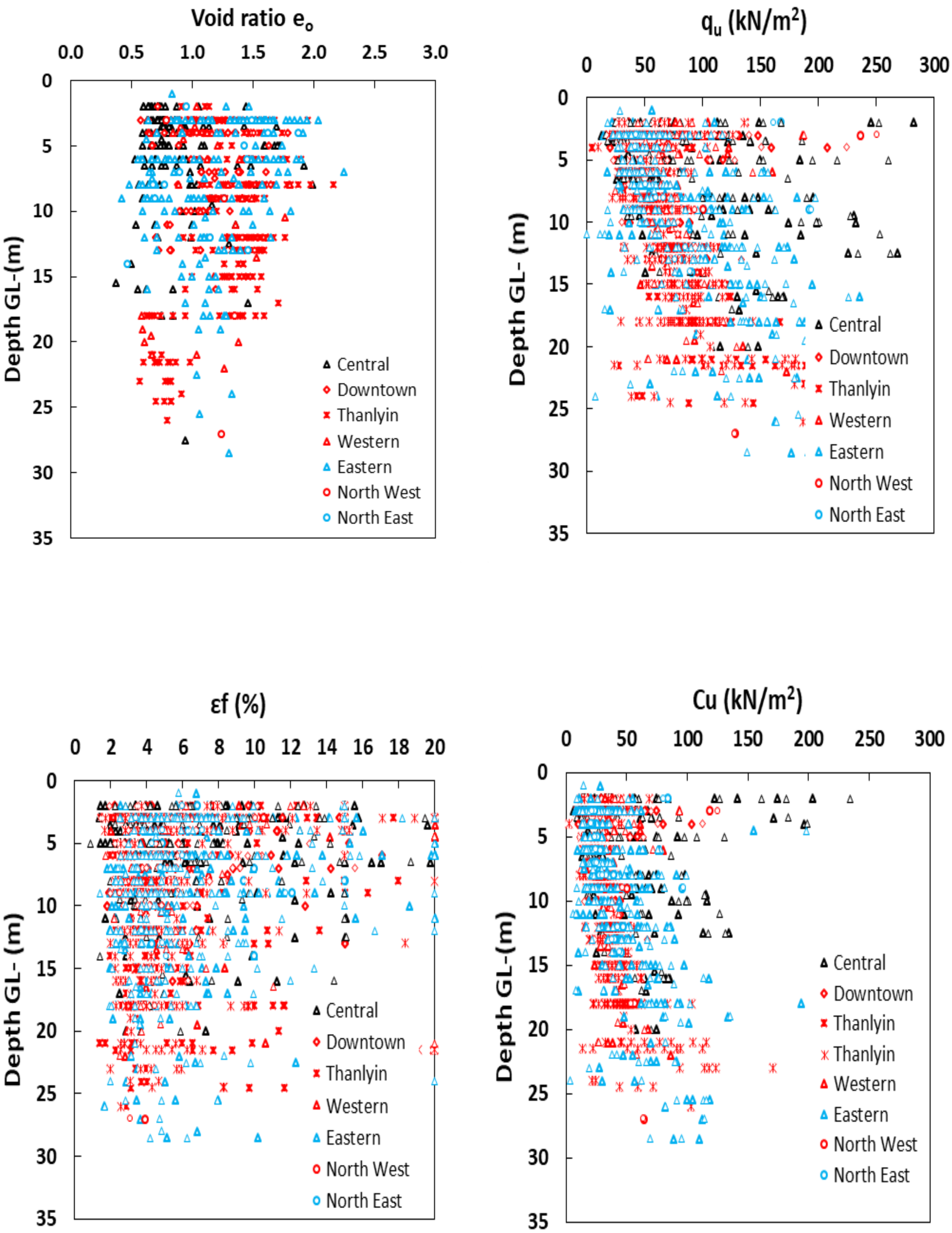


Figure 2 Mechanical Properties of Yangon Sub-areas Graphs

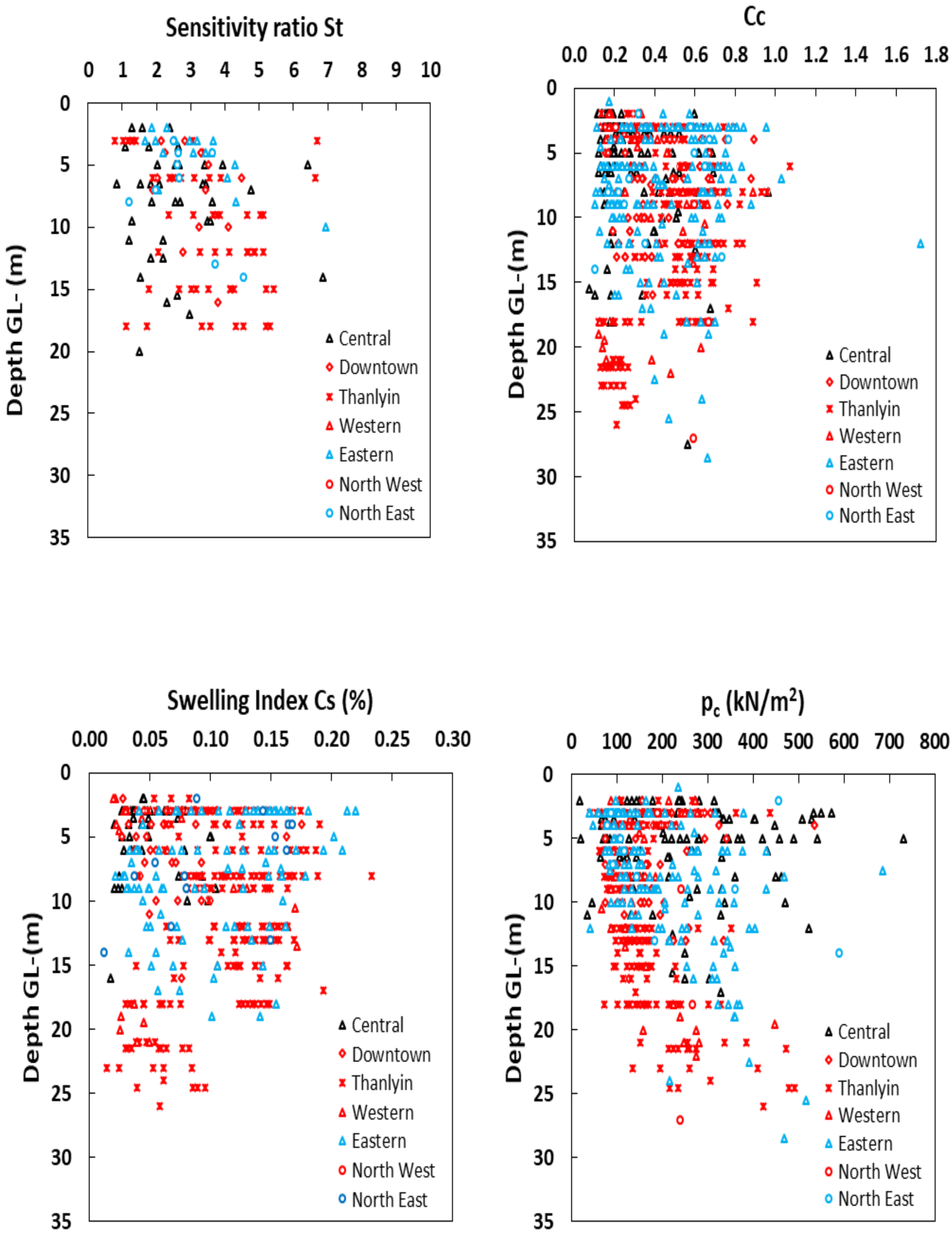


Figure 2 Mechanical Properties of Yangon Sub-areas Graphs

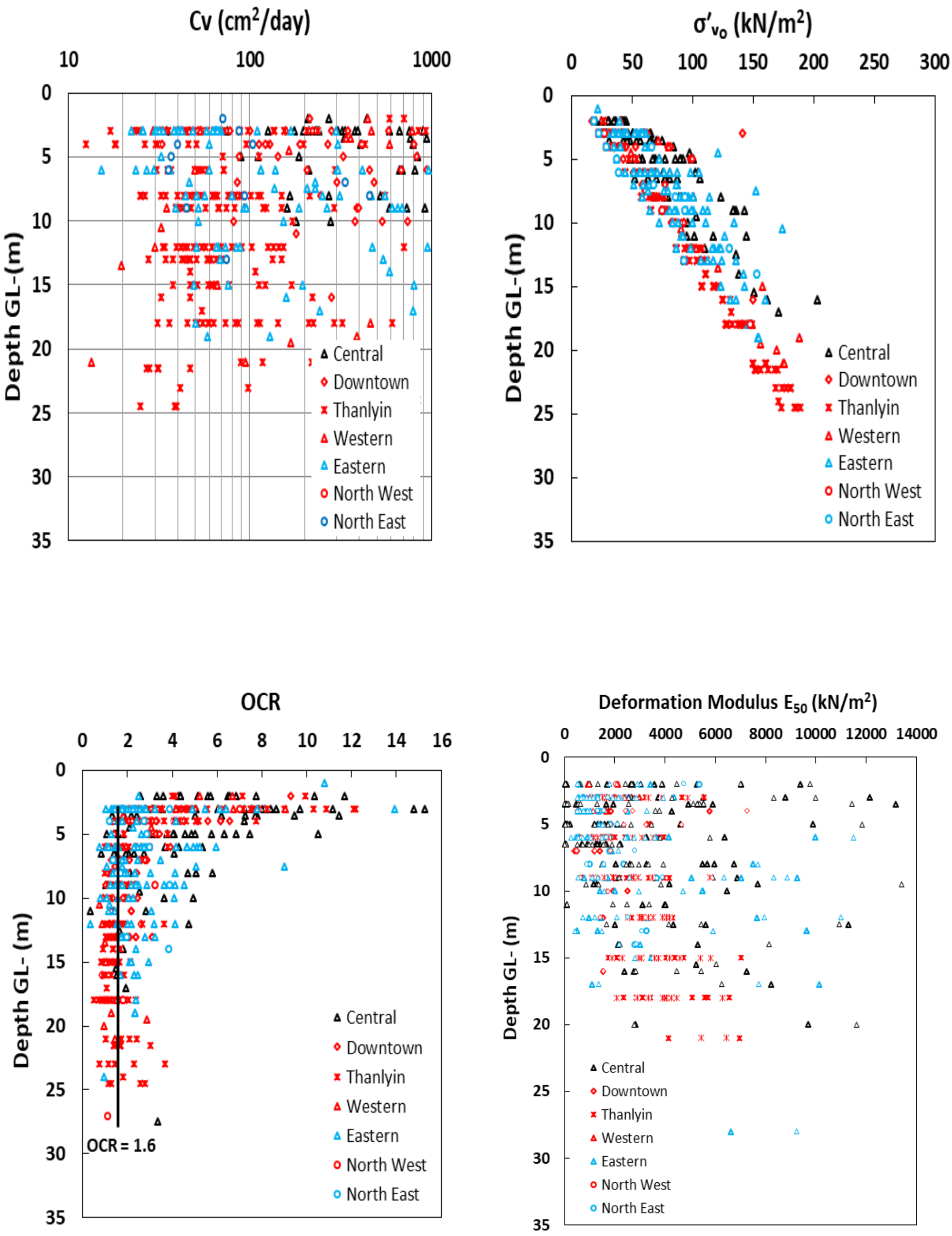
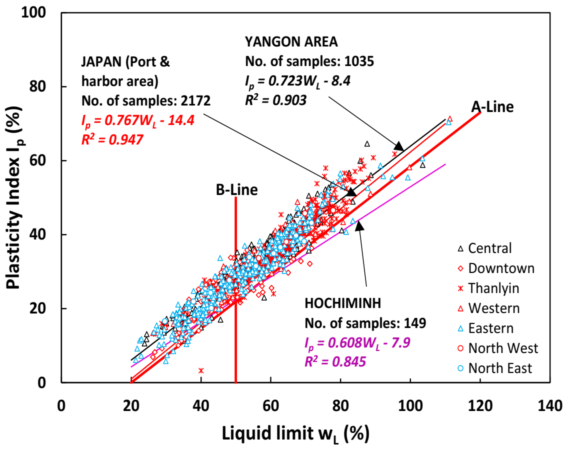
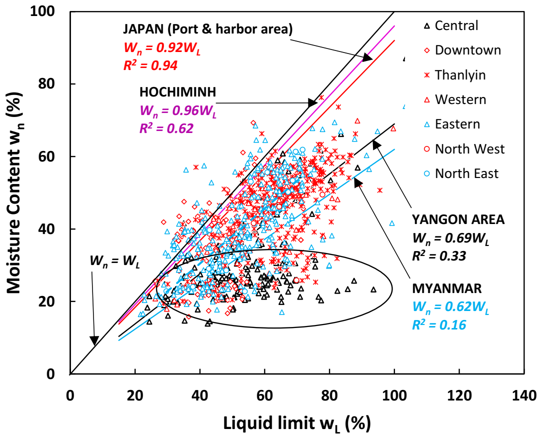


Figure 2 Mechanical Properties of Yangon Sub-areas Graphs

APPENDIX “B”

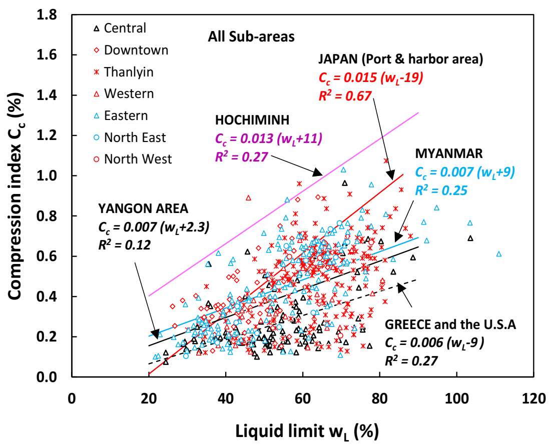
CORRELATION GRAPHS AND DETERMINATION FACTORS BETWEEN PHYSICAL AND MECHANICAL PROPERTIES OF YANGON CLAYS

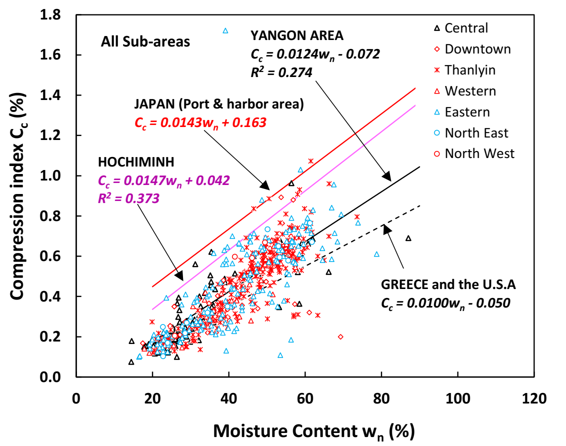




Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	$I_p = 0.656w_L - 3.1$	0.984	4
③ Western	$I_p = 0.671w_L - 5.6$	0.910	60
⑥ Downtown	$I_p = 0.574w_L - 3.7$	0.735	126
⑦ Thanlyin	$I_p = 0.758w_L - 10.7$	0.902	340
② North East	$I_p = 0.611w_L - 2.1$	0.958	13
④ Eastern	$I_p = 0.720w_L - 8.2$	0.924	299
⑤ Central	$I_p = 0.721w_L - 6.8$	0.906	193
Yangon Area	$I_p = 0.723w_L - 8.4$	0.903	1035

Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	$w_n = 0.60w_L$	-0.15	4
③ Western	$w_n = 0.71w_L$	0.66	60
⑥ Downtown	$w_n = 0.79w_L$	0.29	126
⑦ Thanlyin	$w_n = 0.72w_L$	0.20	340
② North East	$w_n = 0.76w_L$	0.64	13
④ Eastern	$w_n = 0.74w_L$	0.39	299
⑤ Central	$w_n = 0.54w_L$	0.25	193
Yangon Area	$w_n = 0.69w_L$	0.33	1035

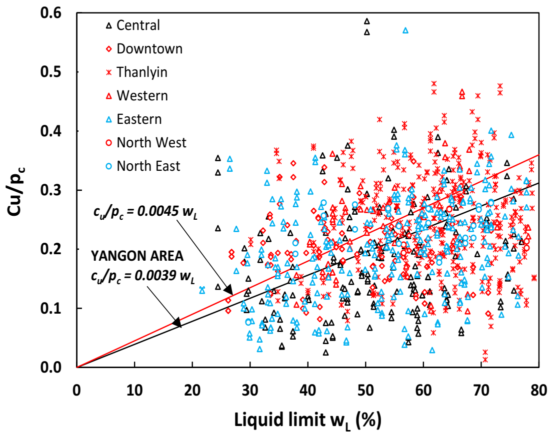


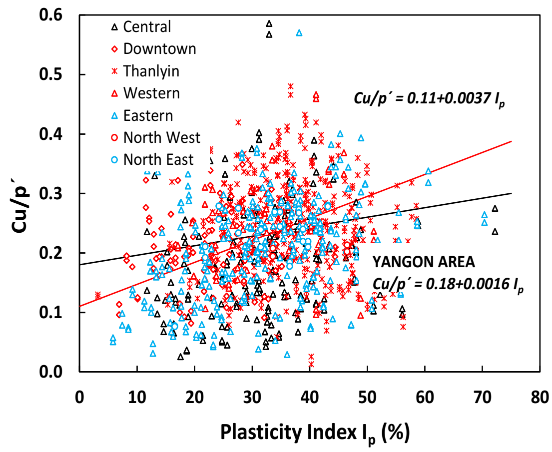


Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	$C_c = -0.001 (w_L - 533.3)$	0.002	4
③ Western	$C_c = 0.009 (w_L - 10.5)$	0.27	41
⑥ Downtown	$C_c = 0.009 (w_L - 1.6)$	0.37	95
⑦ Thanlyin	$C_c = 0.007 (w_L + 9.2)$	0.17	219
② North East	$C_c = 0.015 (w_L - 25.2)$	0.73	13
④ Eastern	$C_c = 0.009 (w_L - 4.1)$	0.39	178
⑤ Central	$C_c = 0.004 (w_L + 24.6)$	0.01	136
Yangon Area	$C_c = 0.007 (w_L + 2.3)$	0.12	686

Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	$C_c = -0.019w_n - 0.284$	0.956	4
③ Western	$C_c = 0.0139w_n - 0.152$	0.708	41
⑥ Downtown	$C_c = 0.012w_n - 0.091$	0.598	95
⑦ Thanlyin	$C_c = 0.0146w_n - 0.185$	0.686	219
② North East	$C_c = 0.0148w_n - 0.175$	0.913	13
④ Eastern	$C_c = 0.0126w_n - 0.080$	0.539	178
⑤ Central	$C_c = 0.0126w_n - 0.018$	0.069	136
Yangon Area	$C_c = 0.0124w_n - 0.072$	0.274	686

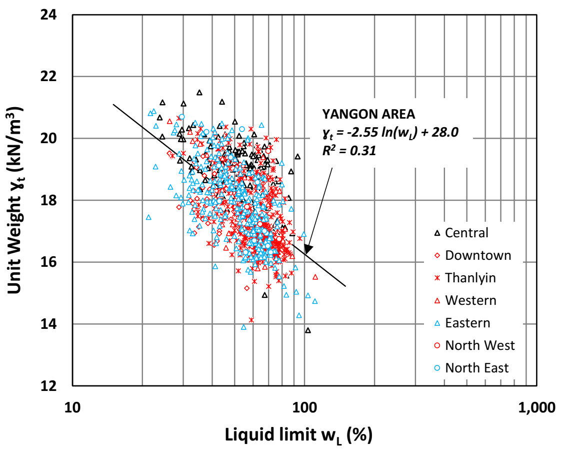
Figure 1 Correlation Graphs and Determination Factors between Physical and Mechanical Properties of Yangon Clays

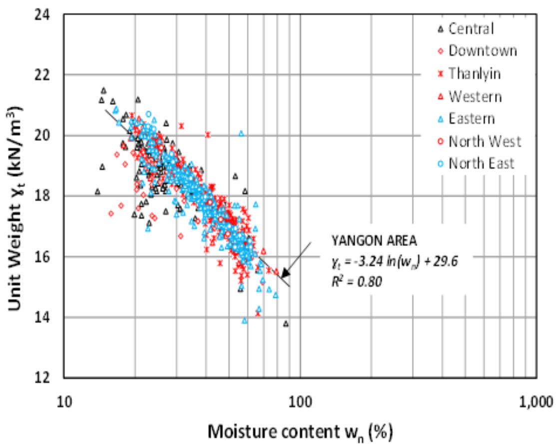




Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	$c_u/p_c = 0.0049w_L$	0.034	8
③ Western	$c_u/p_c = 0.0043w_L$	-0.026	68
⑥ Downtown	$c_u/p_c = 0.0038w_L$	-0.583	151
⑦ Thanlyin	$c_u/p_c = 0.0038w_L$	-0.280	392
② North East	$c_u/p_c = 0.0040w_L$	0.131	24
④ Eastern	$c_u/p_c = 0.0037w_L$	-0.072	301
⑤ Central	$c_u/p_c = 0.0046w_L$	-0.004	192
Yangon Area	$c_u/p_c = 0.0039w_L$	-0.039	1136

Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	$c_u/p_c = 0.0168I_p - 0.36$	0.2436	8
③ Western	$c_u/p_c = 0.19 + 0.0014I_p$	0.0243	68
⑥ Downtown	$c_u/p_c = 0.18 + 0.0008I_p$	0.0113	151
⑦ Thanlyin	$c_u/p_c = 0.29 - 0.0011I_p$	0.0102	392
② North East	$c_u/p_c = 0.09 + 0.0040I_p$	0.3146	24
④ Eastern	$c_u/p_c = 0.14 + 0.0023I_p$	0.0645	301
⑤ Central	$c_u/p_c = 0.19 + 0.0020I_p$	0.0025	192
Yangon Area	$c_u/p_c = 0.18 + 0.0016I_p$	0.0077	1136

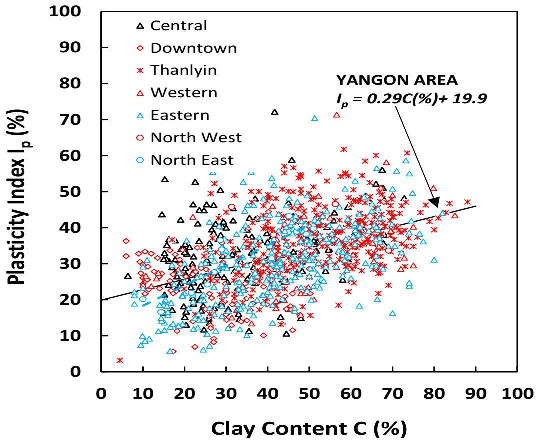




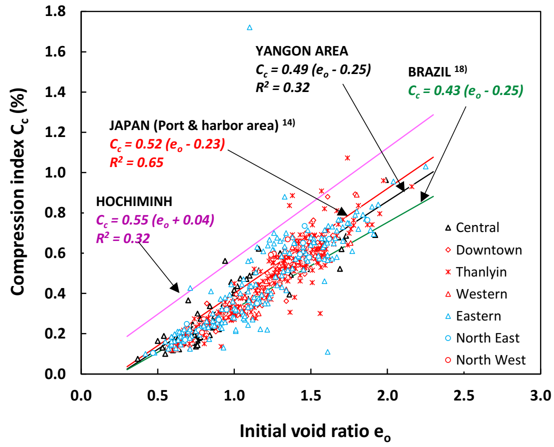
Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	$\gamma_t = -1.11\ln(w_L) + 22.5$	0.01	4
③ Western	$\gamma_t = -3.46\ln(w_L) + 31.7$	0.53	68
⑥ Downtown	$\gamma_t = -2.41\ln(w_L) + 27.1$	0.34	126
⑦ Thanlyin	$\gamma_t = -2.45\ln(w_L) + 27.5$	0.22	339
② North East	$\gamma_t = -4.77\ln(w_L) + 37.1$	0.62	13
④ Eastern	$\gamma_t = -2.59\ln(w_L) + 28.1$	0.36	296
⑤ Central	$\gamma_t = -2.36\ln(w_L) + 28.1$	0.32	186
Yangon Area	$\gamma_t = -2.55\ln(w_L) + 28.0$	0.31	846

Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	$\gamma_t = -4.22\ln(w_n) + 33.4$	0.99	4
③ Western	$\gamma_t = -3.36\ln(w_n) + 30.1$	0.89	68
⑥ Downtown	$\gamma_t = -2.27\ln(w_n) + 25.9$	0.67	126
⑦ Thanlyin	$\gamma_t = -3.96\ln(w_n) + 32.4$	0.87	339
② North East	$\gamma_t = -4.19\ln(w_n) + 33.4$	0.98	13
④ Eastern	$\gamma_t = -3.58\ln(w_n) + 30.9$	0.84	296
⑤ Central	$\gamma_t = -2.81\ln(w_n) + 28.1$	0.64	186
Yangon Area	$\gamma_t = -3.24\ln(w_n) + 29.6$	0.80	1032

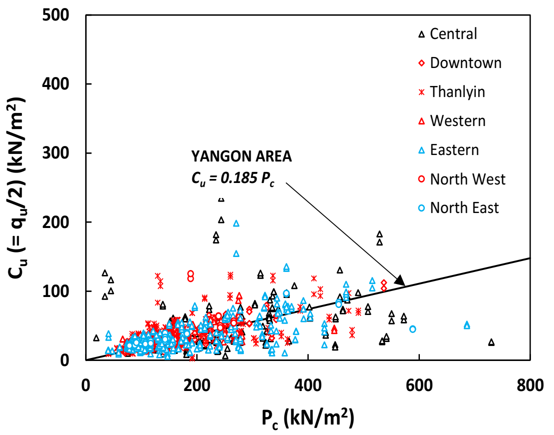
Figure 1 Correlation Graphs and Determination Factors between Physical and Mechanical Properties of Yangon Clays



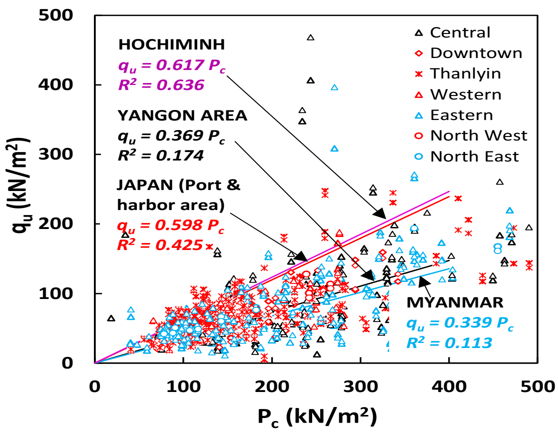
Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	I_p = -0.21C(%) + 51.2	0.29	4
③ Western	I_p = 0.28C(%) + 19.4	0.19	60
⑥ Downtown	I_p = -0.05C(%) + 25.5	0.01	128
⑦ Thanlyin	I_p = 0.22C(%) + 25.8	0.10	340
② North East	I_p = 0.56C(%) + 12.8	0.79	13
④ Eastern	I_p = 0.38C(%) + 14.3	0.32	300
⑤ Central	I_p = 0.22C(%) + 25.1	0.09	193
Yangon Area	I_p = 0.29C(%) + 19.9	0.22	1038



Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	C_c = 0.83 (e_o - 0.54)	0.99	4
③ Western	C_c = 0.54 (e_o - 0.38)	0.89	41
⑥ Downtown	C_c = 0.59 (e_o - 0.44)	0.86	95
⑦ Thanlyin	C_c = 0.59 (e_o - 0.37)	0.83	219
② North East	C_c = 0.52 (e_o - 0.31)	0.94	13
④ Eastern	C_c = 0.49 (e_o - 0.28)	0.65	178
⑤ Central	C_c = 0.48 (e_o - 0.13)	0.07	136
Yangon Area	C_c = 0.49 (e_o - 0.25)	0.32	686

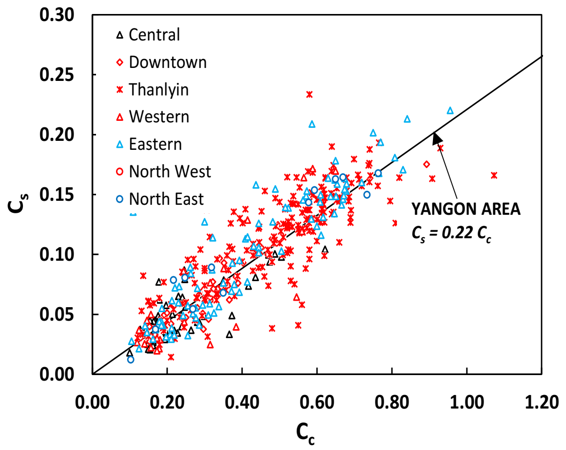


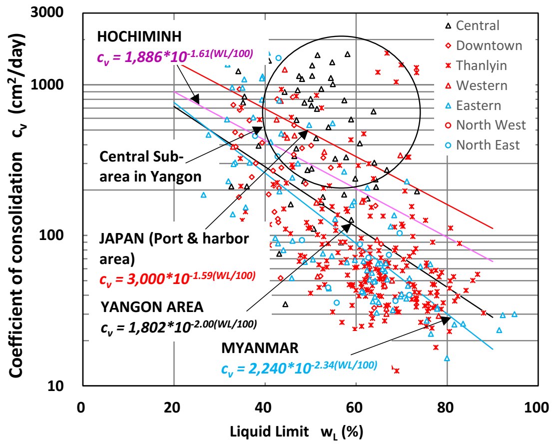
Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	C_u = 0.291P_c	-0.62	4
③ Western	C_u = 0.205 P_c	0.14	39
⑥ Downtown	C_u = 0.189 P_c	0.64	95
⑦ Thanlyin	C_u = 0.211 P_c	0.16	219
② North East	C_u = 0.158 P_c	0.30	12
④ Eastern	C_u = 0.175 P_c	0.33	157
⑤ Central	C_u = 0.169 P_c	-0.02	121
Yangon Area	C_u = 0.185 P_c	0.17	526



Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	q_u = 0.583 P_c	-0.619	8
③ Western	q_u = 0.411 P_c	0.143	68
⑥ Downtown	q_u = 0.379 P_c	0.643	150
⑦ Thanlyin	q_u = 0.421 P_c	0.163	392
② North East	q_u = 0.317 P_c	0.299	24
④ Eastern	q_u = 0.349 P_c	0.326	301
⑤ Central	q_u = 0.337 P_c	-0.021	191
Yangon Area	q_u = 0.369 P_c	0.174	1134

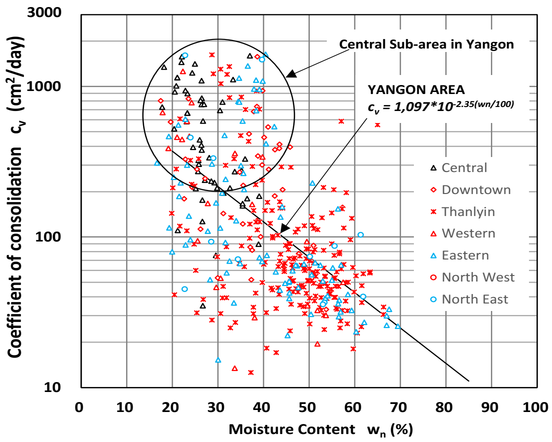
Figure 1 Correlation Graphs and Determination Factors between Physical and Mechanical Properties of Yangon Clays

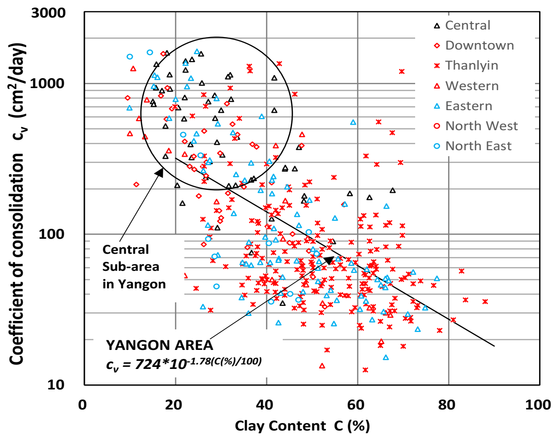




Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	-	-	-
③ Western	$C_s = 0.228\ C_c$	0.747	24
⑥ Downtown	Cs = 0.217 Cc	0.899	39
⑦ Thanlyin	Cs = 0.223 Cc	0.606	213
② North East	Cs = 0.237 Cc	0.917	13
④ Eastern	$C_s = 0.218\ C_c$	0.539	92
⑤ Central	$C_s = 0.196\ C_c$	0.604	53
Yangon Area	$C_s = 0.221\ C_c$	0.700	434

Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	-	-	-
③ Western	$c_v = 10103*10^{-3.39(WL/100)}$	0.5182	24
⑥ Downtown	$c_v = 1368*10^{-1.74(WL/100)}$	0.1883	39
⑦ Thanlyin	$c_v = 577*10^{-1.35(WL/100)}$	0.1484	213
② North East	$c_v = 11058*10^{-3.52(WL/100)}$	0.6659	13
④ Eastern	$c_v = 1498*10^{-2.00(WL/100)}$	0.5364	92
⑤ Central	$c_v = 624*10^{-0.30(WL/100)}$	0.0074	54
Yangon Area	$c_v = 1802*10^{-2.00(WL/100)}$	0.3332	435





Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	-	-	-
③ Western	$c_v = 1954*10^{-3.39(wn/100)}$	0.5187	24
⑥ Downtown	$c_v = 588*10^{-1.04(wn/100)}$	0.0628	39
⑦ Thanlyin	$c_v = 382*10^{-1.52(wn/100)}$	0.1581	213
② North East	$c_v = 878*10^{-2.00(wn/100)}$	0.2666	13
④ Eastern	$c_v = 729*10^{-1.96(wn/100)}$	0.2811	92
⑤ Central	$c_v = 3244*10^{-3.21(wn/100)}$	0.1988	54
Yangon Area	$c_v = 1097*10^{-2.35(wn/100)}$	0.3418	435

Sub-area	Trendline Equation	R^2	Numbers of Data
① North West	-	-	-
③ Western	$c_v = 1049*10^{-2.43(C(\%)/100)}$	0.6624	24
⑥ Downtown	$c_v = 848*10^{-1.78(C(\%)/100)}$	0.2916	39
⑦ Thanlyin	$c_v = 233*10^{-0.91(C(\%)/100)}$	0.1015	213
② North East	$c_v = 1877*10^{-3.47(C(\%)/100)}$	0.6491	13
④ Eastern	$c_v = 1137*10^{-2.26(C(\%)/100)}$	0.5121	92
⑤ Central	$c_v = 1486*10^{-1.74(C(\%)/100)}$	0.3297	54
Yangon Area	$c_v = 724*10^{-1.78(C(\%)/100)}$	0.3702	435

Figure 1 Correlation Graphs and Determination Factors between Physical and Mechanical Properties of Yangon Clays

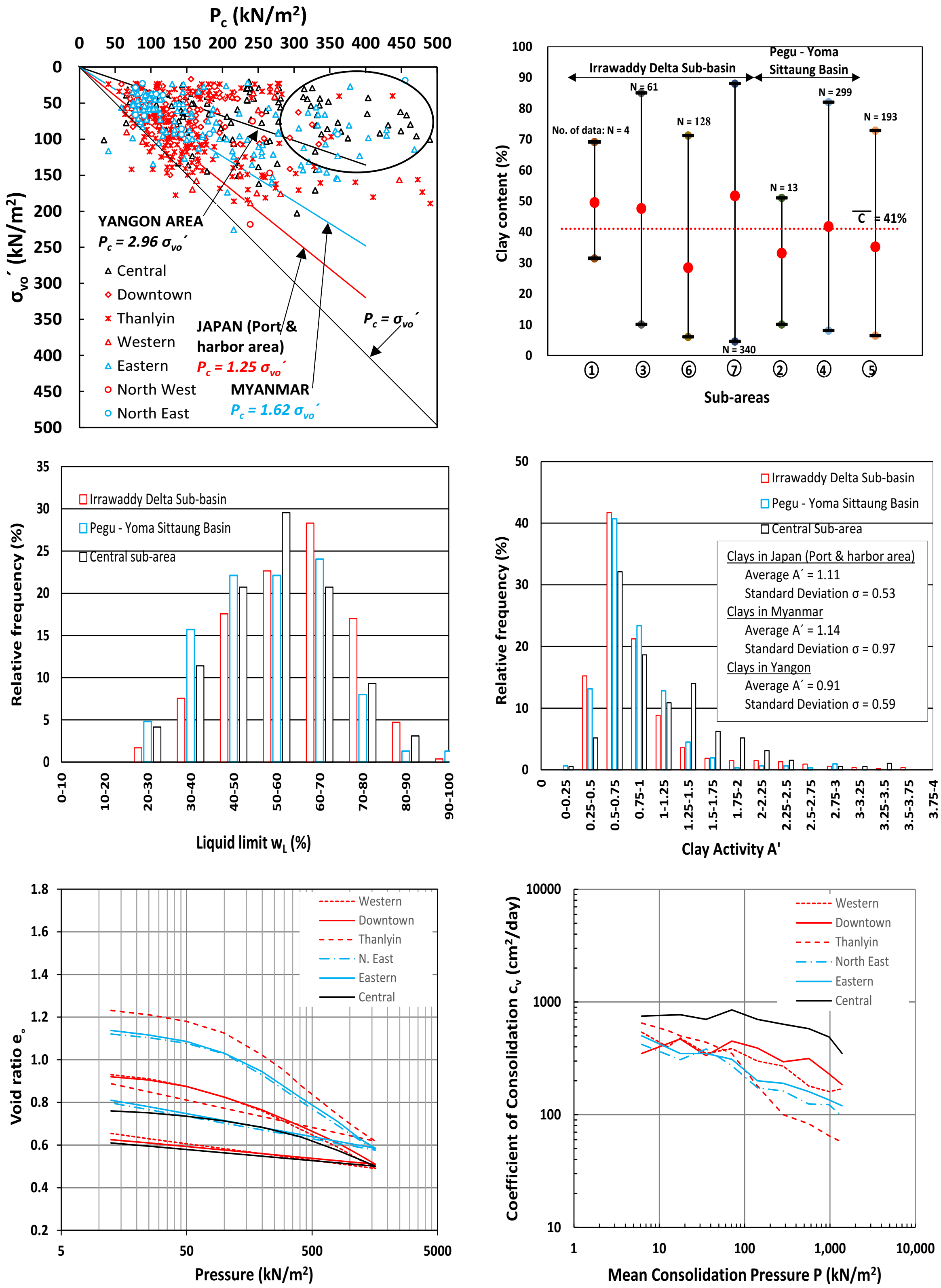
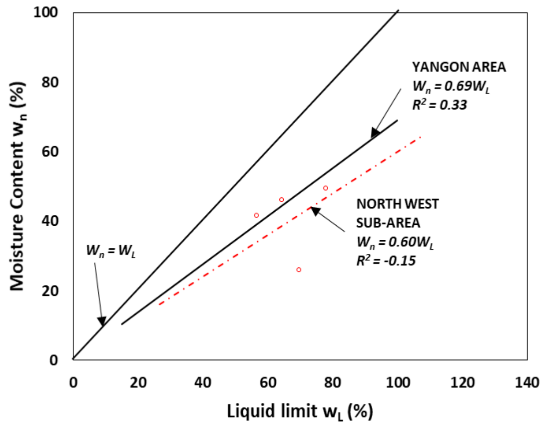
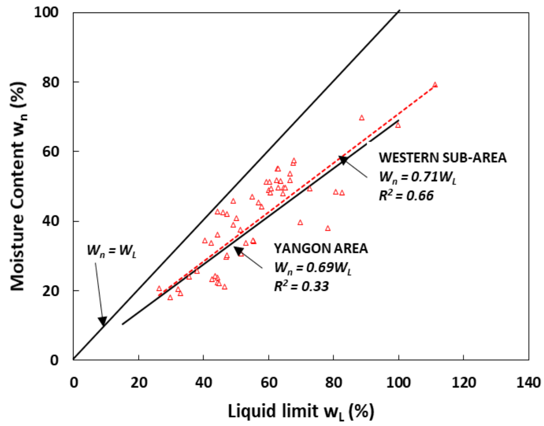


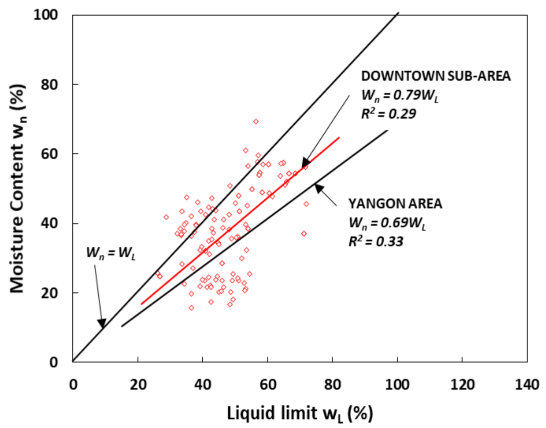
Figure 1 Correlation Graphs and Determination Factors between Physical and Mechanical Properties of Yangon Clays

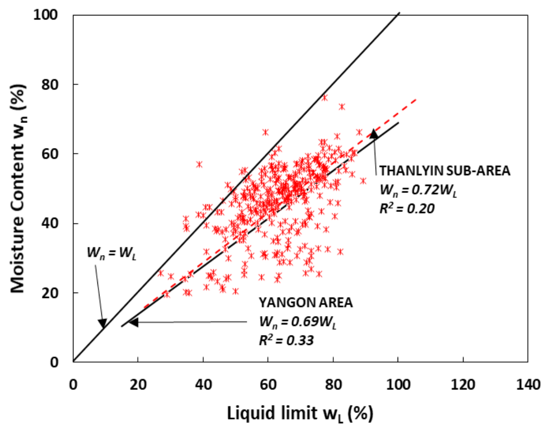
APPENDIX “C”

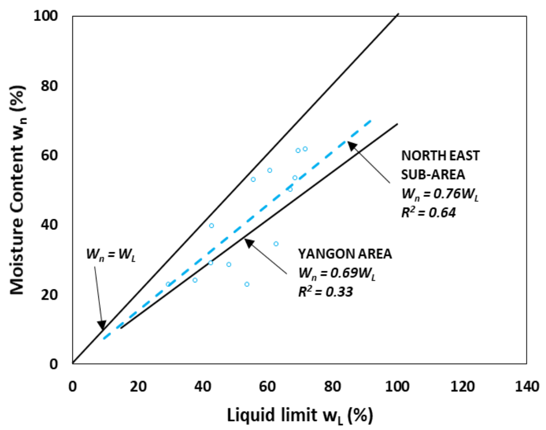
CORRELATION GRAPHS BETWEEN PHYSICAL AND MECHANICAL PROPERTIES OF YANGON CLAYS (SUB-AREA WISE GRAPHS)

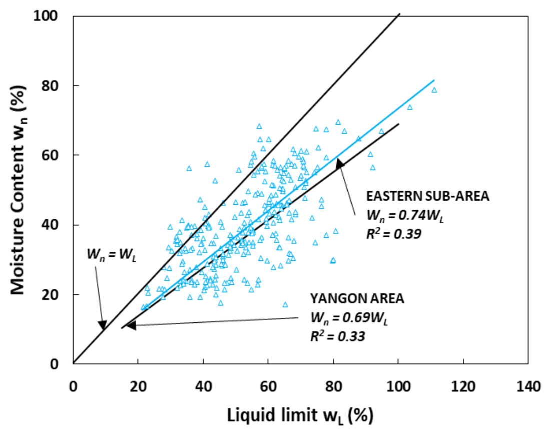

YANGON AREA
$W_n = 0.69W_L$
$R^2 = 0.33$
NORTH WEST
SUB-AREA
$W_n = 0.60W_L$
$R^2 = -0.15$
$W_n = W_L$
Moisture Content w_n (%)
Liquid limit w_L (%)

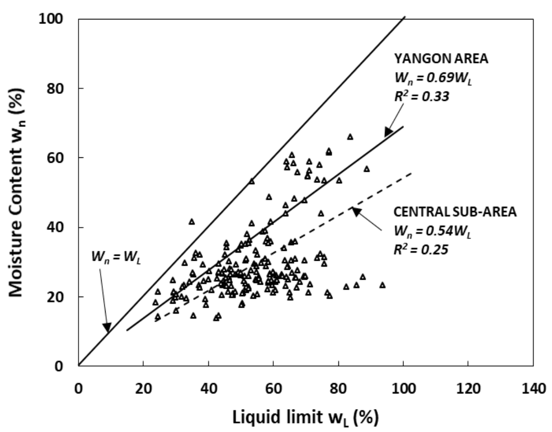

WESTERN SUB-AREA
$W_n = 0.71W_L$
$R^2 = 0.66$
YANGON AREA
$W_n = 0.69W_L$
$R^2 = 0.33$
$W_n = W_L$
Moisture Content w_n (%)
Liquid limit w_L (%)

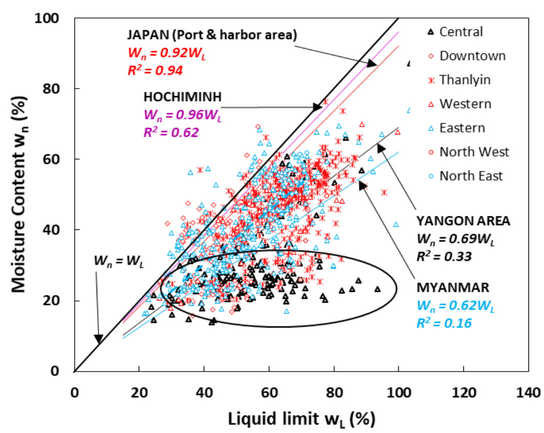

DOWNTOWN SUB-AREA
$W_n = 0.79W_L$
$R^2 = 0.29$
YANGON AREA
$W_n = 0.69W_L$
$R^2 = 0.33$
$W_n = W_L$
Moisture Content w_n (%)
Liquid limit w_L (%)

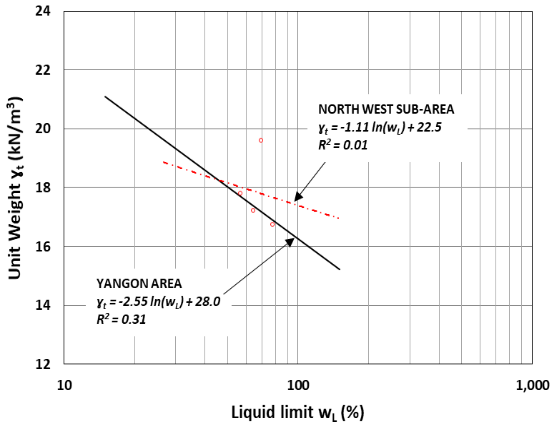

THANLYIN SUB-AREA
$W_n = 0.72W_L$
$R^2 = 0.20$
YANGON AREA
$W_n = 0.69W_L$
$R^2 = 0.33$
$W_n = W_L$
Moisture Content w_n (%)
Liquid limit w_L (%)

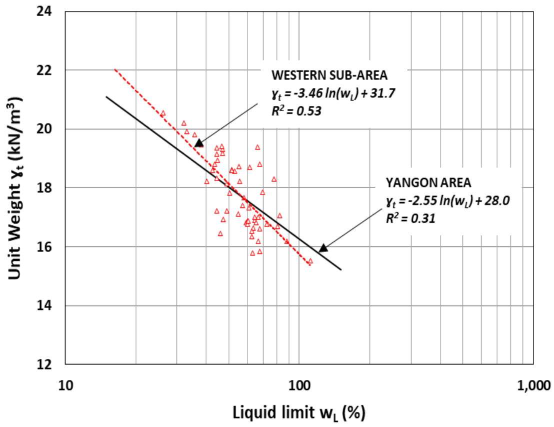

NORTH EAST
SUB-AREA
$W_n = 0.76W_L$
$R^2 = 0.64$
YANGON AREA
$W_n = 0.69W_L$
$R^2 = 0.33$
$W_n = W_L$
Moisture Content w_n (%)
Liquid limit w_L (%)

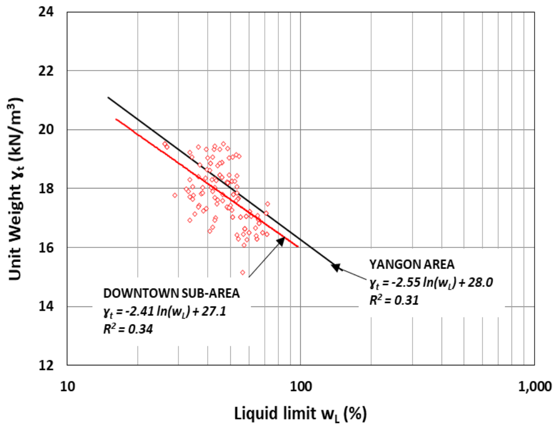

EASTERN SUB-AREA
$W_n = 0.74W_L$
$R^2 = 0.39$
YANGON AREA
$W_n = 0.69W_L$
$R^2 = 0.33$
$W_n = W_L$
Moisture Content w_n (%)
Liquid limit w_L (%)

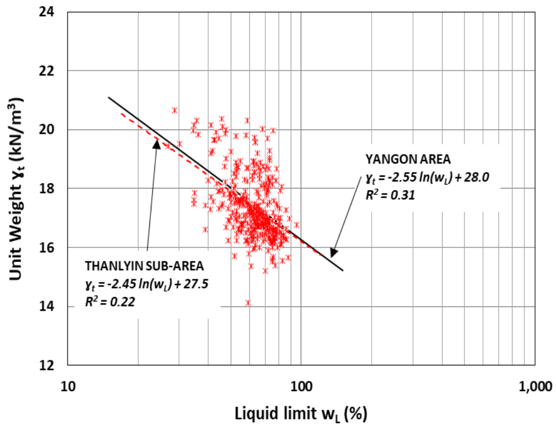

YANGON AREA
$W_n = 0.69W_L$
$R^2 = 0.33$
CENTRAL SUB-AREA
$W_n = 0.54W_L$
$R^2 = 0.25$
$W_n = W_L$
Moisture Content w_n (%)
Liquid limit w_L (%)

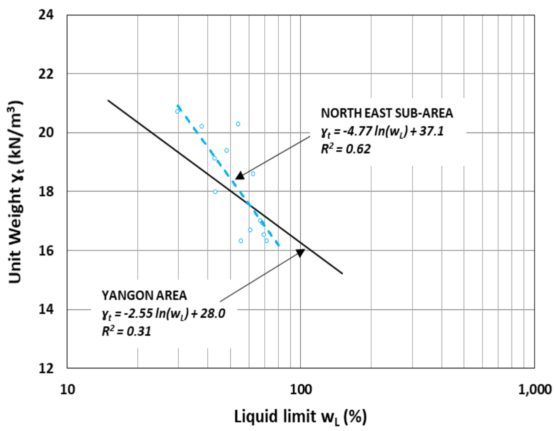

JAPAN (Port & harbor area)
$W_n = 0.92W_L$
$R^2 = 0.94$
HOCHIMINH
$W_n = 0.96W_L$
$R^2 = 0.62$
Central
Downtown
Thanlyin
Western
Eastern
North West
North East
YANGON AREA
$W_n = 0.69W_L$
$R^2 = 0.33$
MYANMAR
$W_n = 0.62W_L$
$R^2 = 0.16$
$W_n = W_L$
Moisture Content w_n (%)
Liquid limit w_L (%)

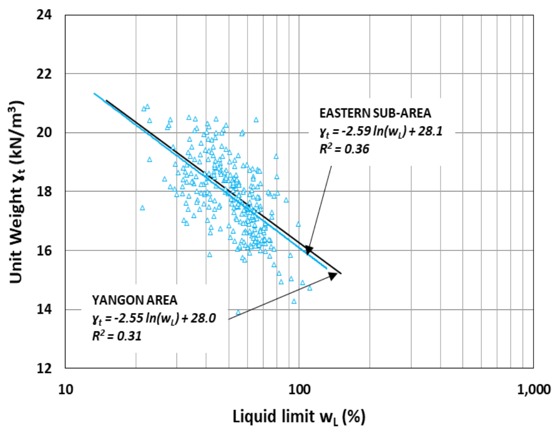

NORTH WEST SUB-AREA
$\gamma_t = -1.11\ln(w_L) + 22.5$
$R^2 = 0.01$
YANGON AREA
$\gamma_t = -2.55\ln(w_L) + 28.0$
$R^2 = 0.31$
Unit Weight γ_t (kN/m^3)
Liquid limit w_L (%)

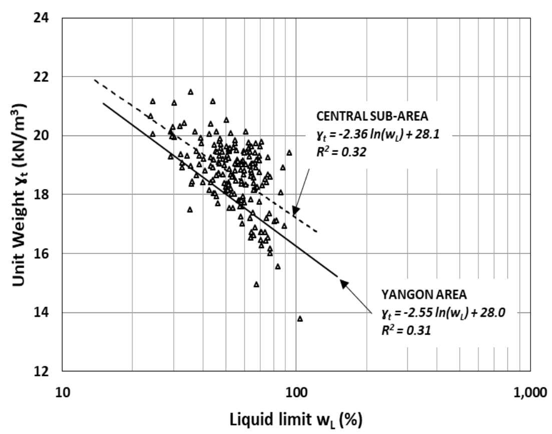

WESTERN SUB-AREA
$\gamma_t = -3.46\ln(w_L) + 31.7$
$R^2 = 0.53$
YANGON AREA
$\gamma_t = -2.55\ln(w_L) + 28.0$
$R^2 = 0.31$
Unit Weight γ_t (kN/m^3)
Liquid limit w_L (%)

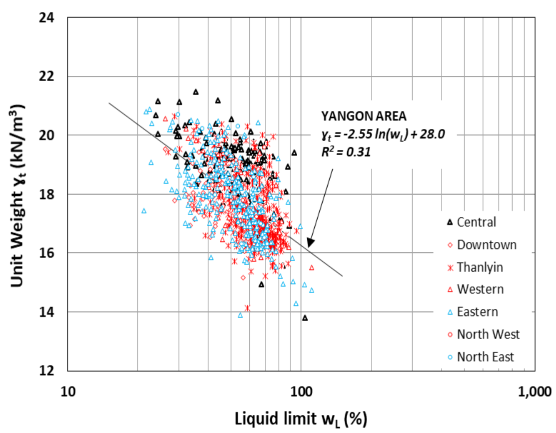

YANGON AREA
$\gamma_t = -2.55\ln(w_L) + 28.0$
$R^2 = 0.31$
DOWNTOWN SUB-AREA
$\gamma_t = -2.41\ln(w_L) + 27.1$
$R^2 = 0.34$
Unit Weight γ_t (kN/m^3)
Liquid limit w_L (%)

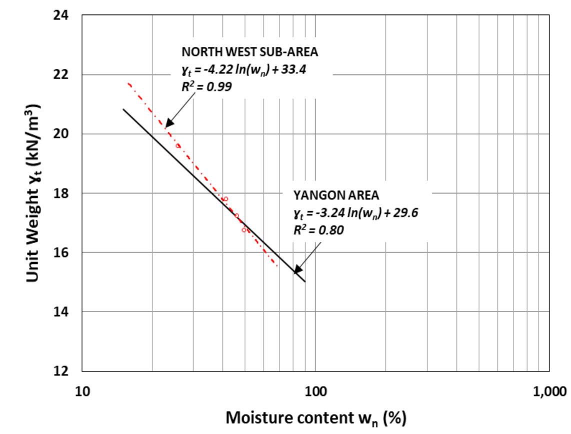

YANGON AREA
$\gamma_t = -2.55\ln(w_L) + 28.0$
$R^2 = 0.31$
THANLYIN SUB-AREA
$\gamma_t = -2.45\ln(w_L) + 27.5$
$R^2 = 0.22$
Unit Weight γ_t (kN/m^3)
Liquid limit w_L (%)

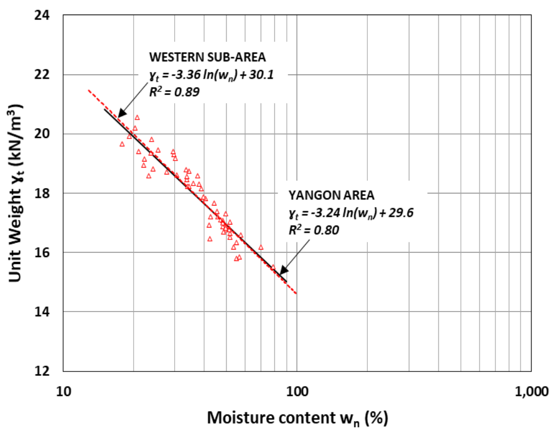

NORTH EAST SUB-AREA
$\gamma_t = -4.77\ln(w_L) + 37.1$
$R^2 = 0.62$
YANGON AREA
$\gamma_t = -2.55\ln(w_L) + 28.0$
$R^2 = 0.31$
Unit Weight γ_t (kN/m^3)
Liquid limit w_L (%)

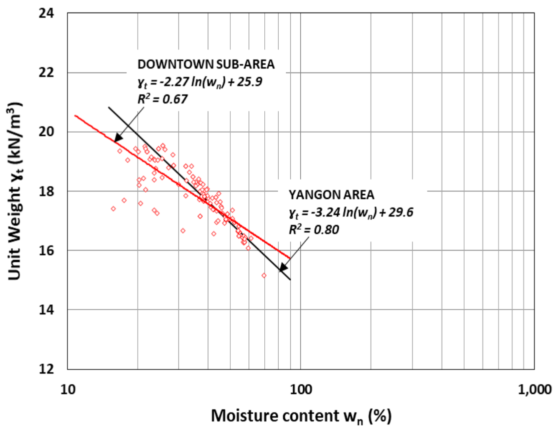

EASTERN SUB-AREA
$\gamma_t = -2.59\ln(w_L) + 28.1$
$R^2 = 0.36$
YANGON AREA
$\gamma_t = -2.55\ln(w_L) + 28.0$
$R^2 = 0.31$
Unit Weight γ_t (kN/m^3)
Liquid limit w_L (%)

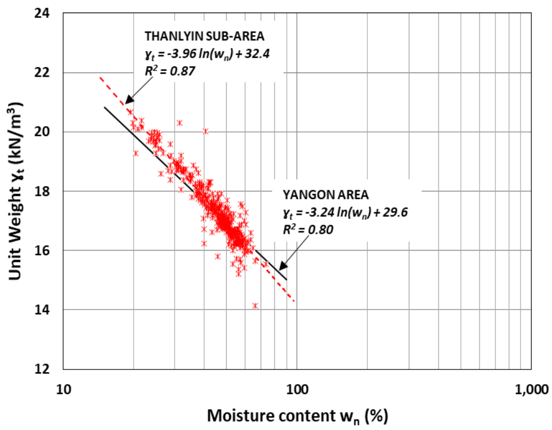

CENTRAL SUB-AREA
$\gamma_t = -2.36\ln(w_L) + 28.1$
$R^2 = 0.32$
YANGON AREA
$\gamma_t = -2.55\ln(w_L) + 28.0$
$R^2 = 0.31$
Unit Weight γ_t (kN/m^3)
Liquid limit w_L (%)

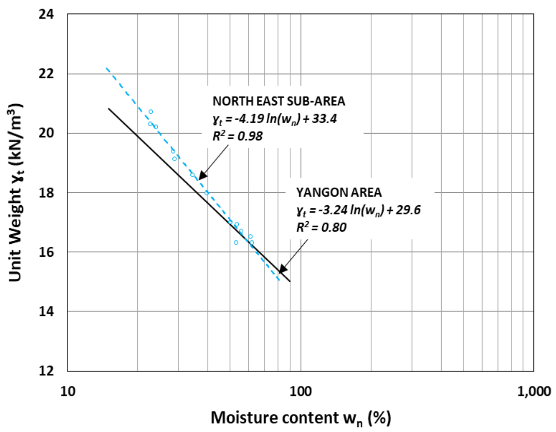

YANGON AREA
$\gamma_t = -2.55\ln(w_L) + 28.0$
$R^2 = 0.31$
Central
Downtown
Thanlyin
Western
Eastern
North West
North East
Unit Weight γ_t (kN/m^3)
Liquid limit w_L (%)

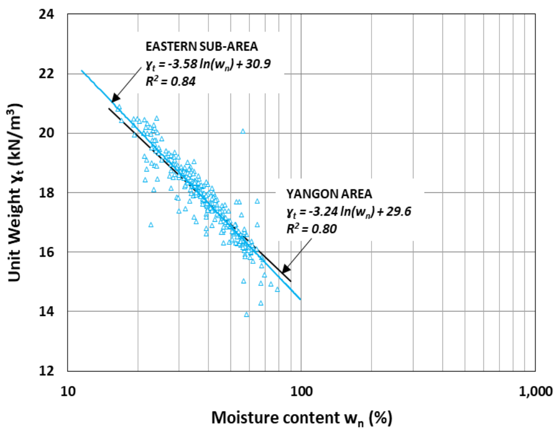

NORTH WEST SUB-AREA
$\gamma_t = -4.22\ln(w_n) + 33.4$
$R^2 = 0.99$
YANGON AREA
$\gamma_t = -3.24\ln(w_n) + 29.6$
$R^2 = 0.80$
Unit Weight γ_t (kN/m^3)
Moisture content w_n (%)

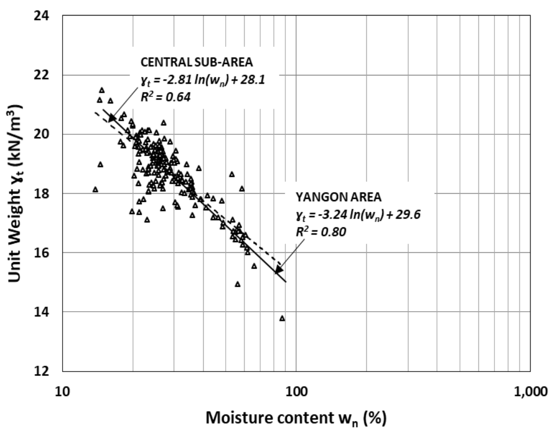

WESTERN SUB-AREA
$\gamma_t = -3.36\ln(w_n) + 30.1$
$R^2 = 0.89$
YANGON AREA
$\gamma_t = -3.24\ln(w_n) + 29.6$
$R^2 = 0.80$
Unit Weight γ_t (kN/m^3)
Moisture content w_n (%)

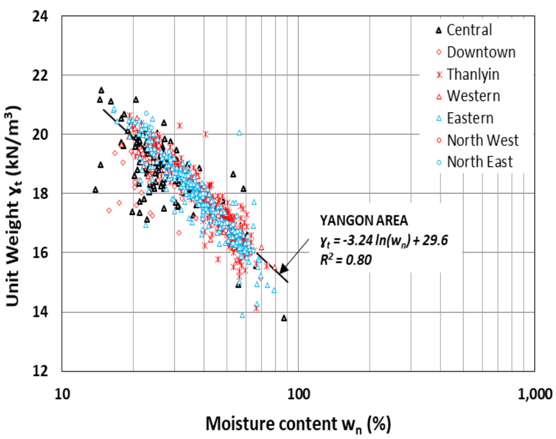

DOWNTOWN SUB-AREA
$\gamma_t = -2.27\ln(w_n) + 25.9$
$R^2 = 0.67$
YANGON AREA
$\gamma_t = -3.24\ln(w_n) + 29.6$
$R^2 = 0.80$
Unit Weight γ_t (kN/m^3)
Moisture content w_n (%)

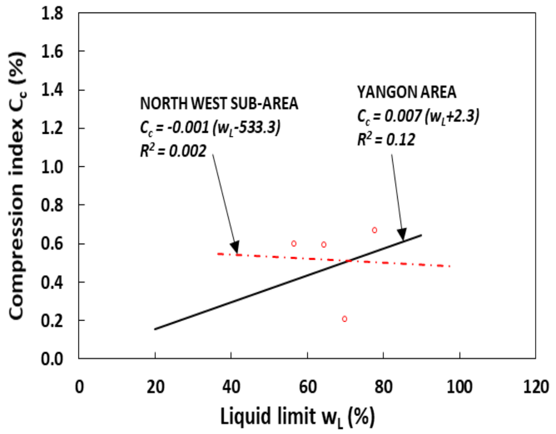

THANLYIN SUB-AREA
$\gamma_t = -3.96\ln(w_n) + 32.4$
$R^2 = 0.87$
YANGON AREA
$\gamma_t = -3.24\ln(w_n) + 29.6$
$R^2 = 0.80$
Unit Weight γ_t (kN/m^3)
Moisture content w_n (%)

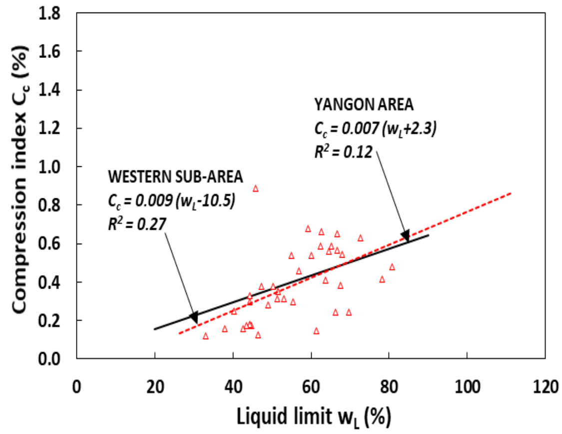

NORTH EAST SUB-AREA
$\gamma_t = -4.19\ln(w_n) + 33.4$
$R^2 = 0.98$
YANGON AREA
$\gamma_t = -3.24\ln(w_n) + 29.6$
$R^2 = 0.80$
Unit Weight γ_t (kN/m^3)
Moisture content w_n (%)

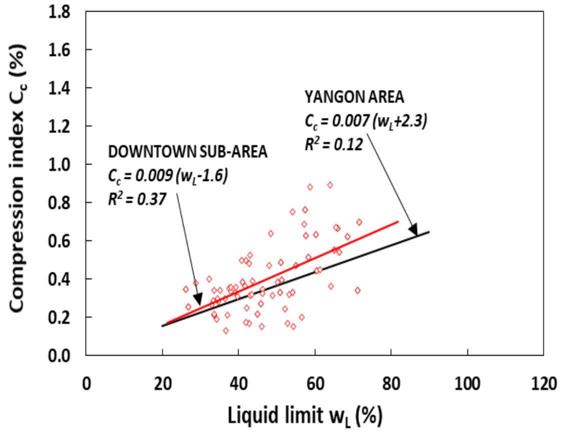

EASTERN SUB-AREA
$\gamma_t = -3.58\ln(w_n) + 30.9$
$R^2 = 0.84$
YANGON AREA
$\gamma_t = -3.24\ln(w_n) + 29.6$
$R^2 = 0.80$
Unit Weight γ_t (kN/m^3)
Moisture content w_n (%)

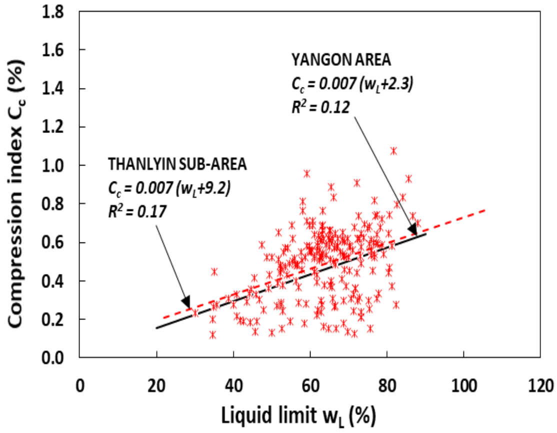

CENTRAL SUB-AREA
$\gamma_t = -2.81\ln(w_n) + 28.1$
$R^2 = 0.64$
YANGON AREA
$\gamma_t = -3.24\ln(w_n) + 29.6$
$R^2 = 0.80$
Unit Weight γ_t (kN/m^3)
Moisture content w_n (%)

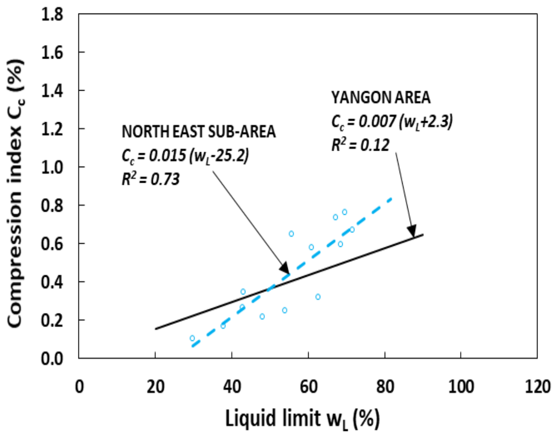

Central
Downtown
Thanlyin
Western
Eastern
North West
North East
YANGON AREA
$\gamma_t = -3.24\ln(w_n) + 29.6$
$R^2 = 0.80$
Unit Weight γ_t (kN/m^3)
Moisture content w_n (%)

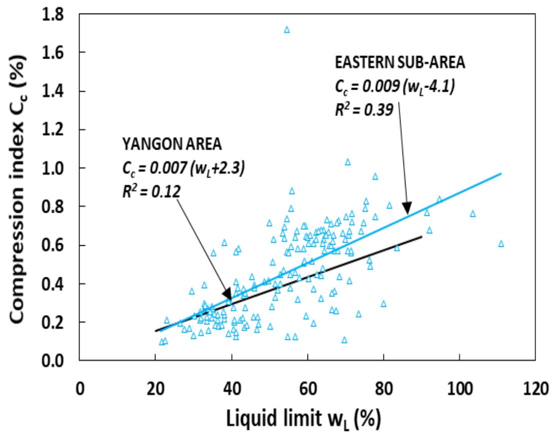

NORTH WEST SUB-AREA
$C_c = -0.001\,(w_L - 533.3)$
$R^2 = 0.002$
YANGON AREA
$C_c = 0.007\,(w_L + 2.3)$
$R^2 = 0.12$
Compression index C_c (%)
Liquid limit w_L (%)

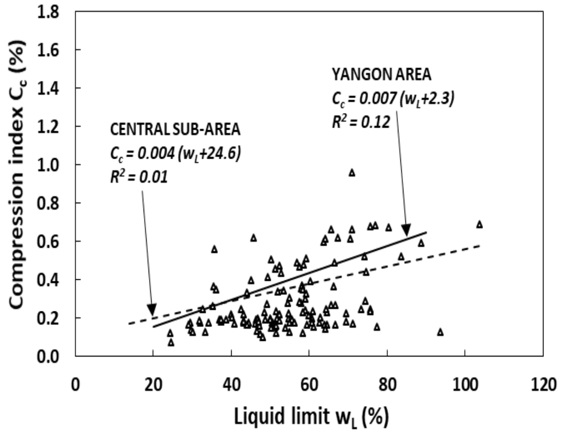

WESTERN SUB-AREA
$C_c = 0.009\,(w_L - 10.5)$
$R^2 = 0.27$
YANGON AREA
$C_c = 0.007\,(w_L + 2.3)$
$R^2 = 0.12$
Compression index C_c (%)
Liquid limit w_L (%)

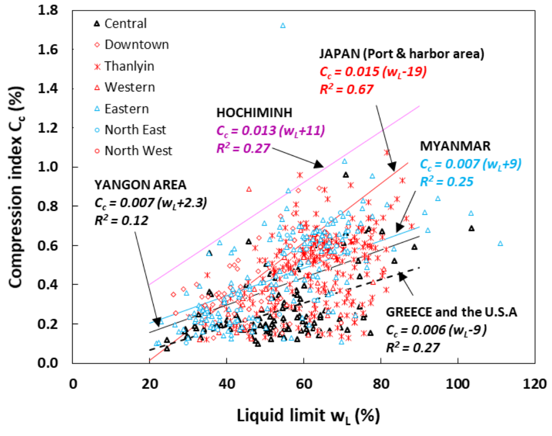

DOWNTOWN SUB-AREA
$C_c = 0.009\,(w_L - 1.6)$
$R^2 = 0.37$
YANGON AREA
$C_c = 0.007\,(w_L + 2.3)$
$R^2 = 0.12$
Compression index C_c (%)
Liquid limit w_L (%)

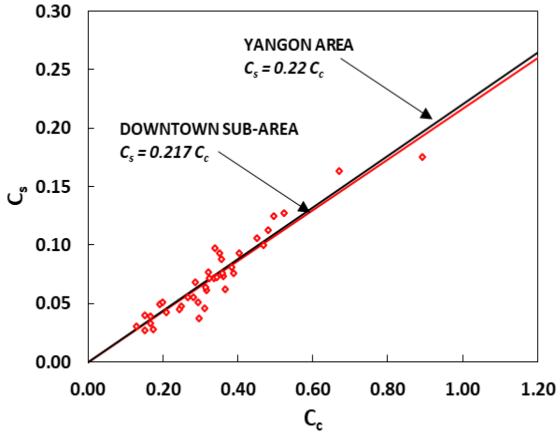

THANLYIN SUB-AREA
$C_c = 0.007\,(w_L + 9.2)$
$R^2 = 0.17$
YANGON AREA
$C_c = 0.007\,(w_L + 2.3)$
$R^2 = 0.12$
Compression index C_c (%)
Liquid limit w_L (%)

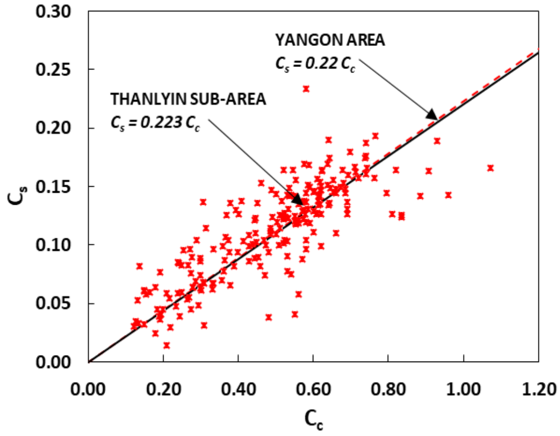

NORTH EAST SUB-AREA
$C_c = 0.015\,(w_L - 25.2)$
$R^2 = 0.73$
YANGON AREA
$C_c = 0.007\,(w_L + 2.3)$
$R^2 = 0.12$
Compression index C_c (%)
Liquid limit w_L (%)

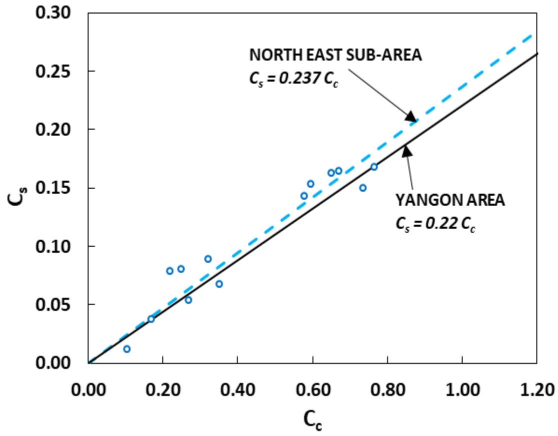

EASTERN SUB-AREA
$C_c = 0.009\,(w_L - 4.1)$
$R^2 = 0.39$
YANGON AREA
$C_c = 0.007\,(w_L + 2.3)$
$R^2 = 0.12$
Compression index C_c (%)
Liquid limit w_L (%)

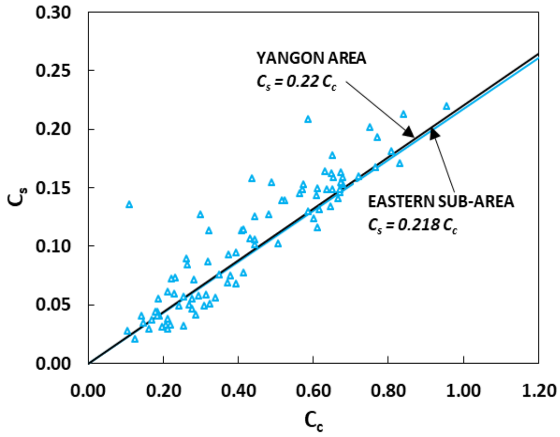

CENTRAL SUB-AREA
$C_c = 0.004\,(w_L + 24.6)$
$R^2 = 0.01$
YANGON AREA
$C_c = 0.007\,(w_L + 2.3)$
$R^2 = 0.12$
Compression index C_c (%)
Liquid limit w_L (%)

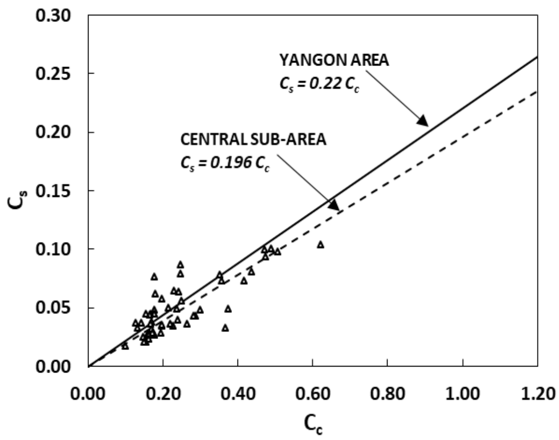

Central
Downtown
Thanlyin
Western
Eastern
North East
North West
JAPAN (Port & harbor area)
$C_c = 0.015\,(w_L - 19)$
$R^2 = 0.67$
HOCHIMINH
$C_c = 0.013\,(w_L + 11)$
$R^2 = 0.27$
MYANMAR
$C_c = 0.007\,(w_L + 9)$
$R^2 = 0.25$
YANGON AREA
$C_c = 0.007\,(w_L + 2.3)$
$R^2 = 0.12$
GREECE and the U.S.A
$C_c = 0.006\,(w_L - 9)$
$R^2 = 0.27$
Compression index C_c (%)
Liquid limit w_L (%)

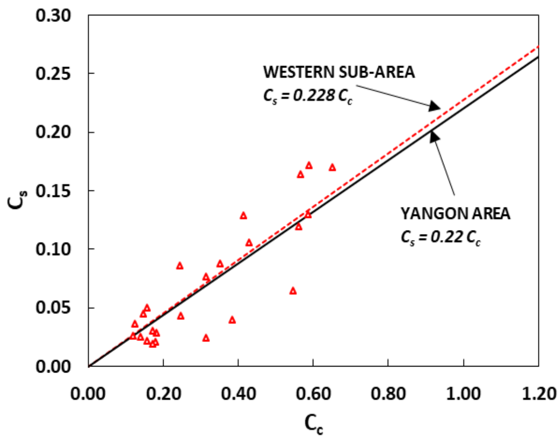

YANGON AREA
$C_s = 0.22\, C_c$
DOWNTOWN SUB-AREA
$C_s = 0.217\, C_c$
C_s
C_c

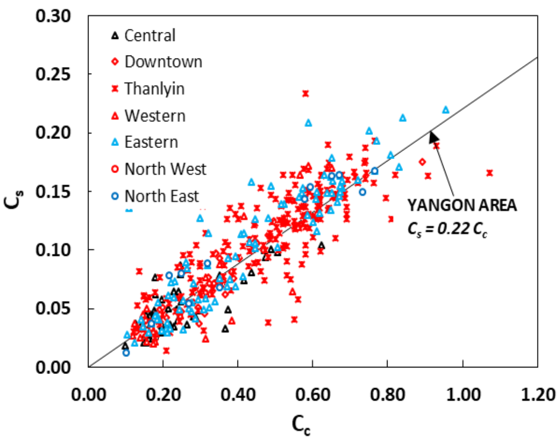

YANGON AREA
$C_s = 0.22\, C_c$
THANLYIN SUB-AREA
$C_s = 0.223\, C_c$
C_s
C_c

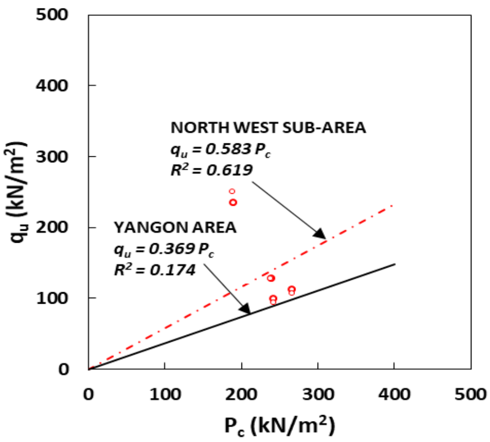

NORTH EAST SUB-AREA
$C_s = 0.237\, C_c$
YANGON AREA
$C_s = 0.22\, C_c$
C_s
C_c

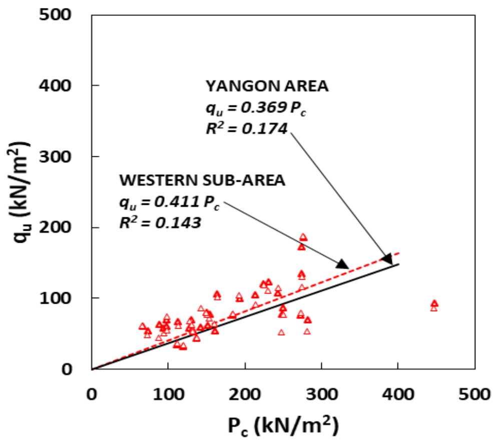

YANGON AREA
$C_s = 0.22\, C_c$
EASTERN SUB-AREA
$C_s = 0.218\, C_c$
C_s
C_c

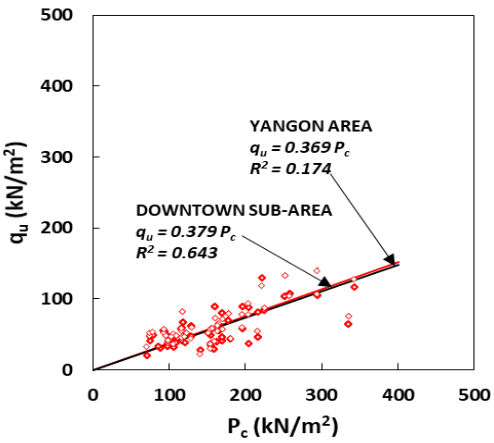

YANGON AREA
$C_s = 0.22\, C_c$
CENTRAL SUB-AREA
$C_s = 0.196\, C_c$
C_s
C_c

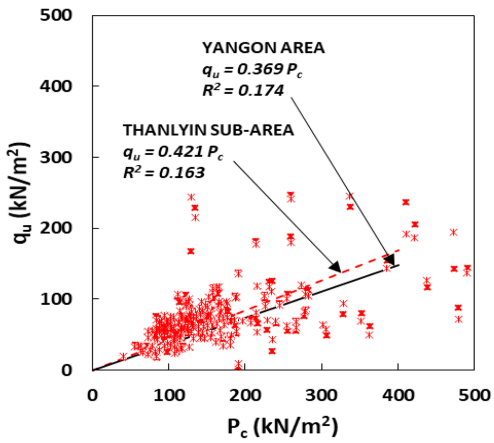

WESTERN SUB-AREA
$C_s = 0.228\, C_c$
YANGON AREA
$C_s = 0.22\, C_c$
C_s
C_c

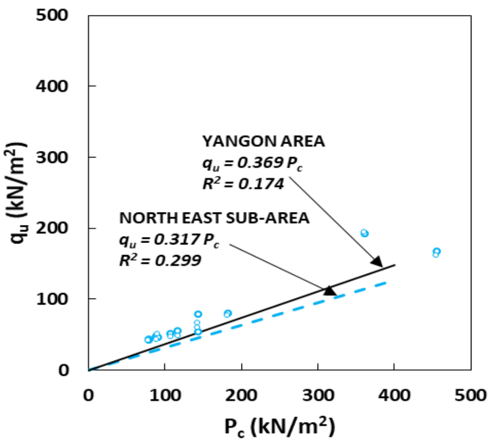

Central
Downtown
Thanlyin
Western
Eastern
North West
North East
YANGON AREA
$C_s = 0.22\, C_c$
C_s
C_c

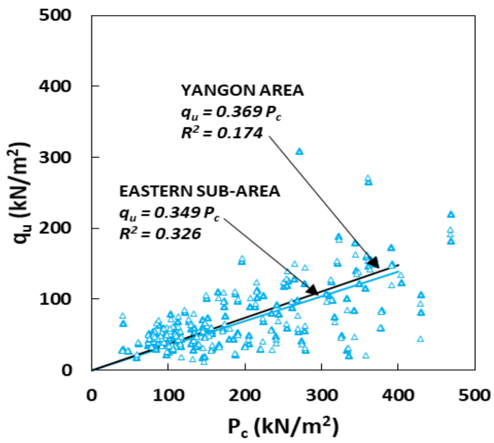

NORTH WEST SUB-AREA
$q_u = 0.583\ P_c$
$R^2 = 0.619$
YANGON AREA
$q_u = 0.369\ P_c$
$R^2 = 0.174$
q_u (kN/m^2)
P_c (kN/m^2)

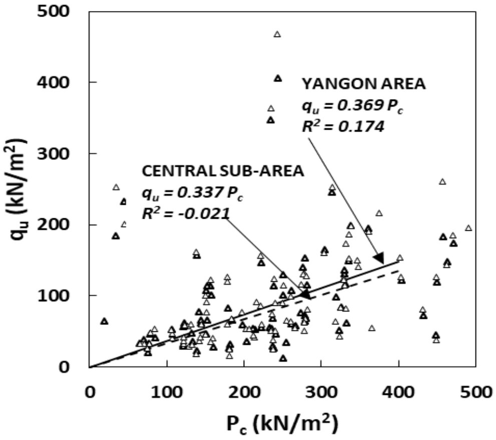

YANGON AREA
$q_u = 0.369\ P_c$
$R^2 = 0.174$
WESTERN SUB-AREA
$q_u = 0.411\ P_c$
$R^2 = 0.143$
q_u (kN/m^2)
P_c (kN/m^2)

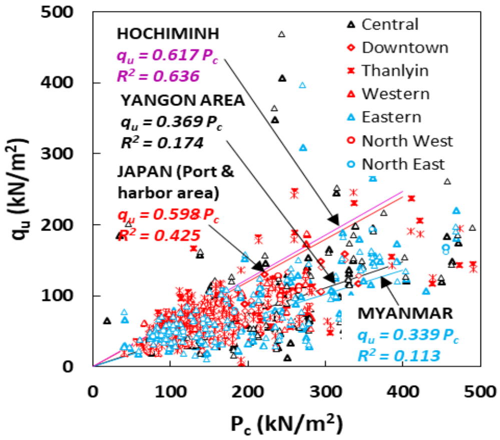

YANGON AREA
$q_u = 0.369\ P_c$
$R^2 = 0.174$
DOWNTOWN SUB-AREA
$q_u = 0.379\ P_c$
$R^2 = 0.643$
q_u (kN/m^2)
P_c (kN/m^2)

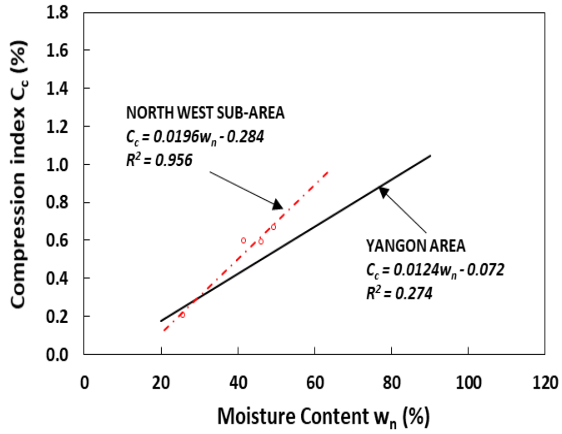

YANGON AREA
$q_u = 0.369\ P_c$
$R^2 = 0.174$
THANLYIN SUB-AREA
$q_u = 0.421\ P_c$
$R^2 = 0.163$
q_u (kN/m^2)
P_c (kN/m^2)

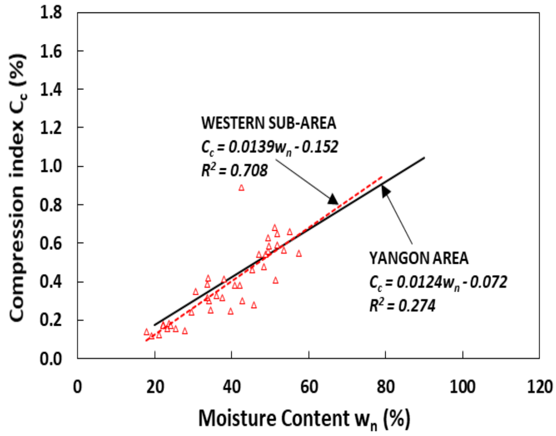

YANGON AREA
$q_u = 0.369\ P_c$
$R^2 = 0.174$
NORTH EAST SUB-AREA
$q_u = 0.317\ P_c$
$R^2 = 0.299$
q_u (kN/m^2)
P_c (kN/m^2)

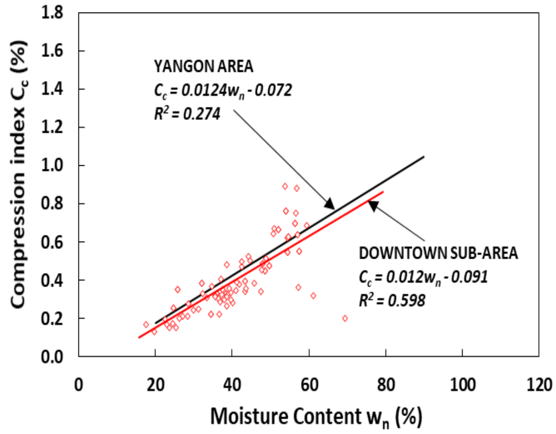

YANGON AREA
$q_u = 0.369\ P_c$
$R^2 = 0.174$
EASTERN SUB-AREA
$q_u = 0.349\ P_c$
$R^2 = 0.326$
q_u (kN/m^2)
P_c (kN/m^2)

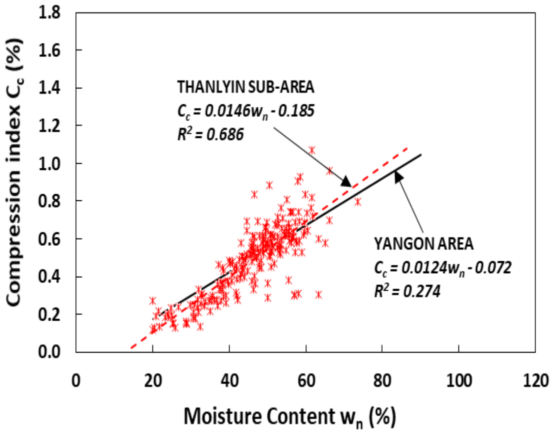

YANGON AREA
$q_u = 0.369\ P_c$
$R^2 = 0.174$
CENTRAL SUB-AREA
$q_u = 0.337\ P_c$
$R^2 = -0.021$
q_u (kN/m^2)
P_c (kN/m^2)

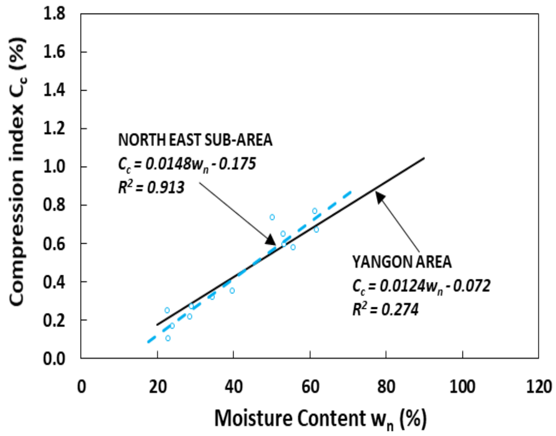

HOCHIMINH
$q_u = 0.617\ P_c$
$R^2 = 0.636$
YANGON AREA
$q_u = 0.369\ P_c$
$R^2 = 0.174$
JAPAN (Port & harbor area)
$q_u = 0.598\ P_c$
$R^2 = 0.425$
MYANMAR
$q_u = 0.339\ P_c$
$R^2 = 0.113$
Central
Downtown
Thanlyin
Western
Eastern
North West
North East
q_u (kN/m^2)
P_c (kN/m^2)

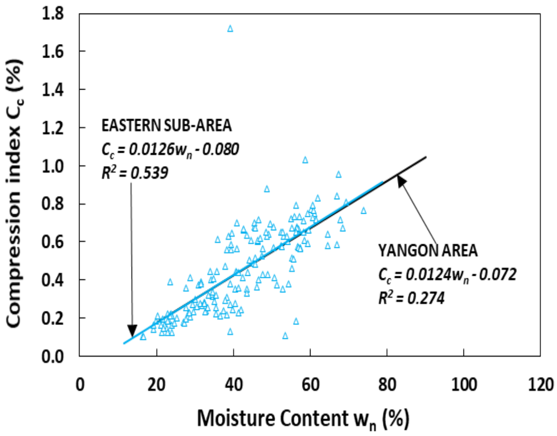

Compression index C_c (%)
Moisture Content w_n (%)
NORTH WEST SUB-AREA
$C_c = 0.0196w_n - 0.284$
$R^2 = 0.956$
YANGON AREA
$C_c = 0.0124w_n - 0.072$
$R^2 = 0.274$

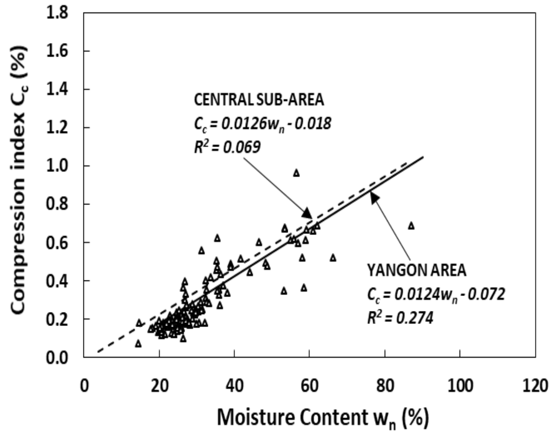

Compression index C_c (%)
Moisture Content w_n (%)
WESTERN SUB-AREA
$C_c = 0.0139w_n - 0.152$
$R^2 = 0.708$
YANGON AREA
$C_c = 0.0124w_n - 0.072$
$R^2 = 0.274$

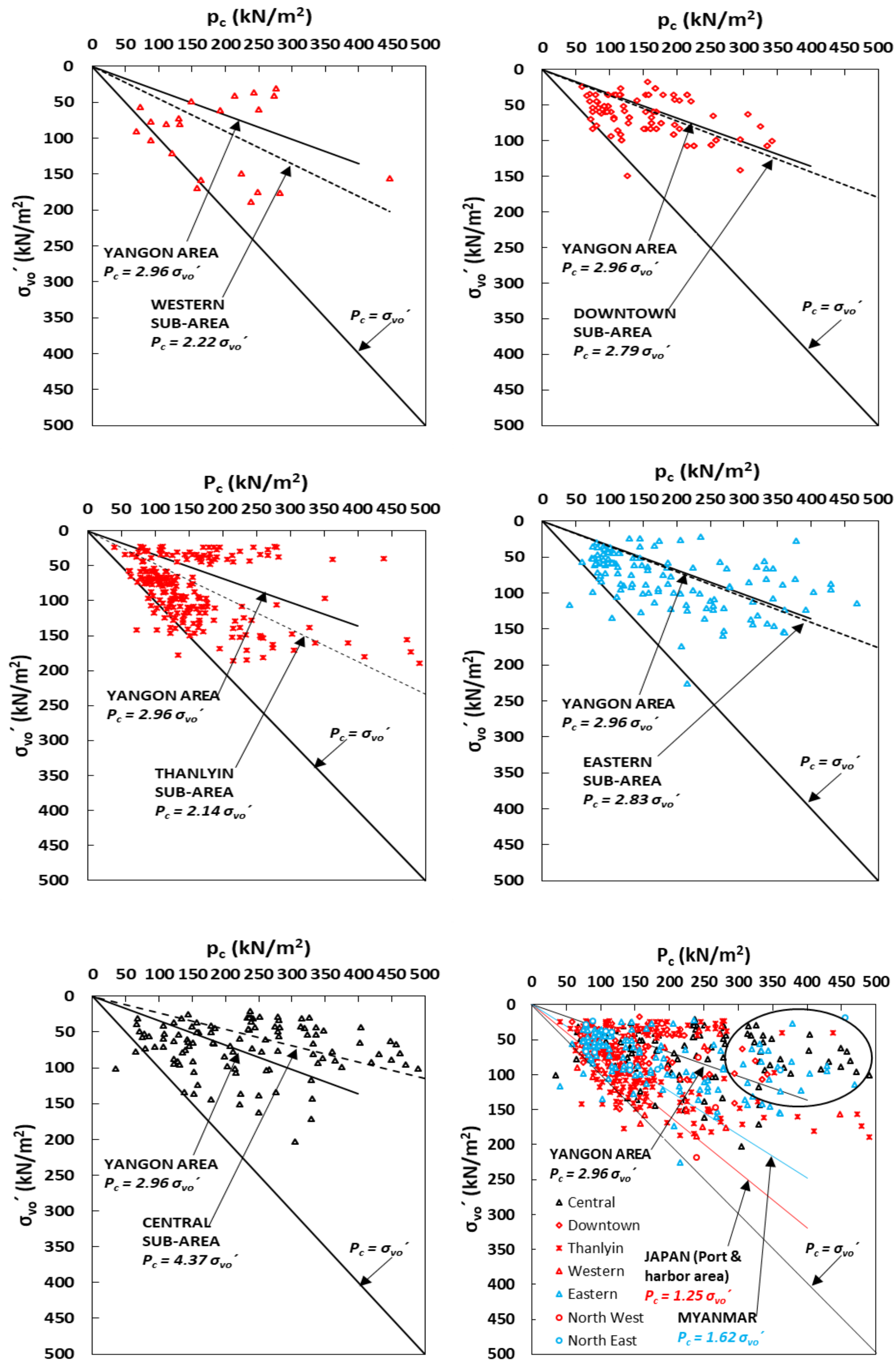

Compression index C_c (%)
Moisture Content w_n (%)
YANGON AREA
$C_c = 0.0124w_n - 0.072$
$R^2 = 0.274$
DOWNTOWN SUB-AREA
$C_c = 0.012w_n - 0.091$
$R^2 = 0.598$


Compression index C_c (%)
Moisture Content w_n (%)
THANLYIN SUB-AREA
$C_c = 0.0146w_n - 0.185$
$R^2 = 0.686$
YANGON AREA
$C_c = 0.0124w_n - 0.072$
$R^2 = 0.274$

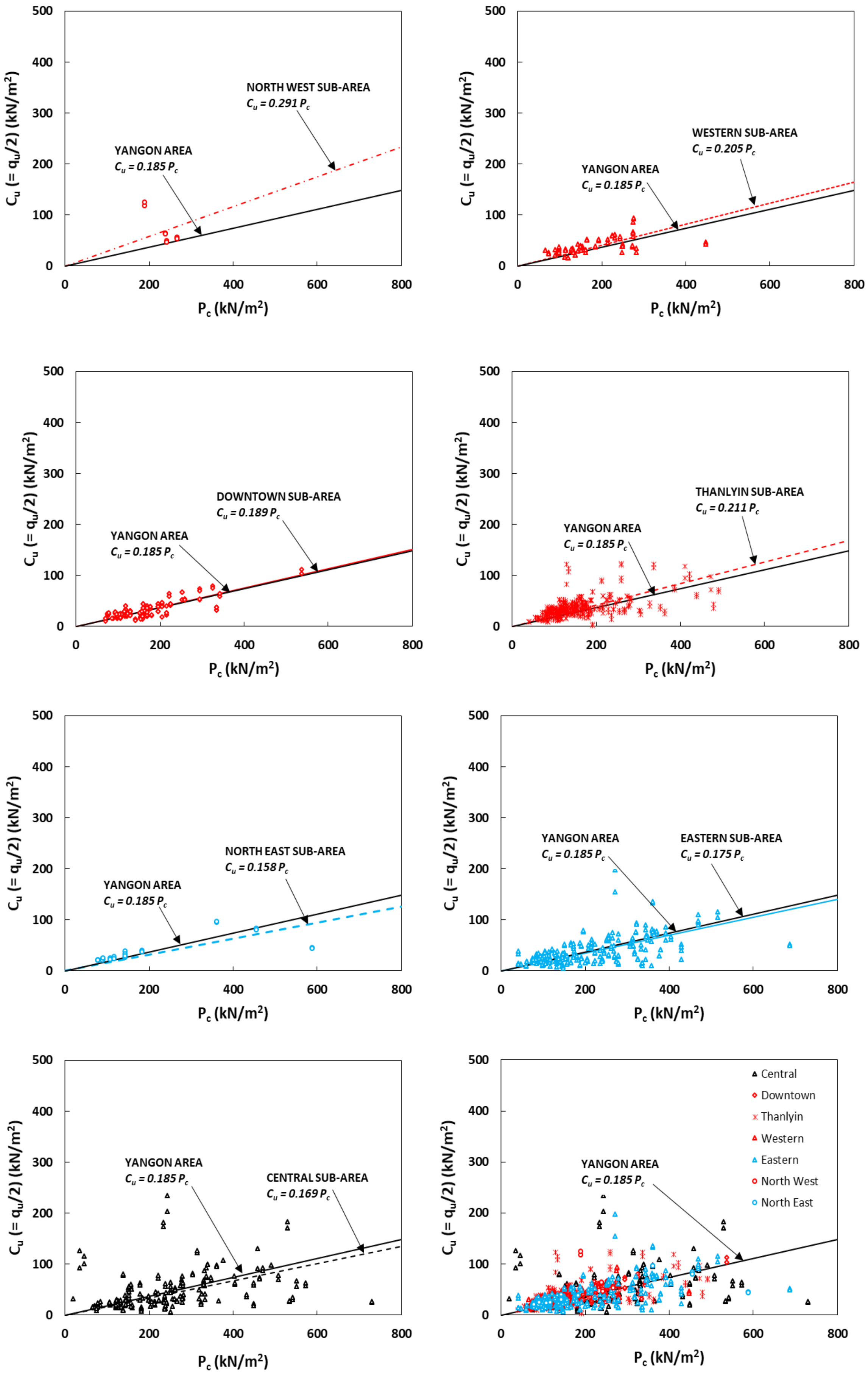

Compression index C_c (%)
Moisture Content w_n (%)
NORTH EAST SUB-AREA
$C_c = 0.0148w_n - 0.175$
$R^2 = 0.913$
YANGON AREA
$C_c = 0.0124w_n - 0.072$
$R^2 = 0.274$

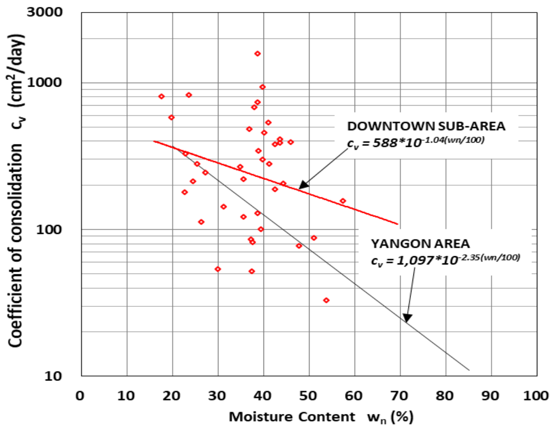

Compression index C_c (%)
Moisture Content w_n (%)
EASTERN SUB-AREA
$C_c = 0.0126w_n - 0.080$
$R^2 = 0.539$
YANGON AREA
$C_c = 0.0124w_n - 0.072$
$R^2 = 0.274$

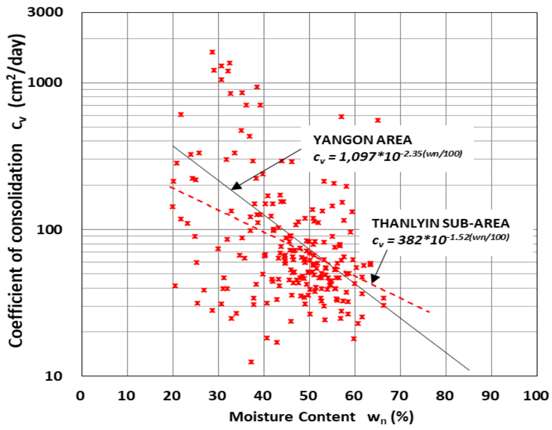

Compression index C_c (%)
Moisture Content w_n (%)
CENTRAL SUB-AREA
$C_c = 0.0126w_n - 0.018$
$R^2 = 0.069$
YANGON AREA
$C_c = 0.0124w_n - 0.072$
$R^2 = 0.274$

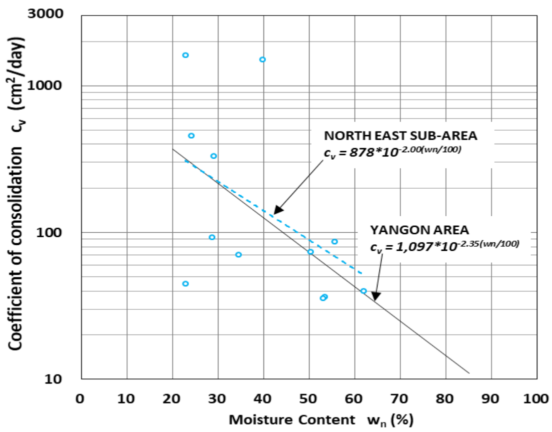


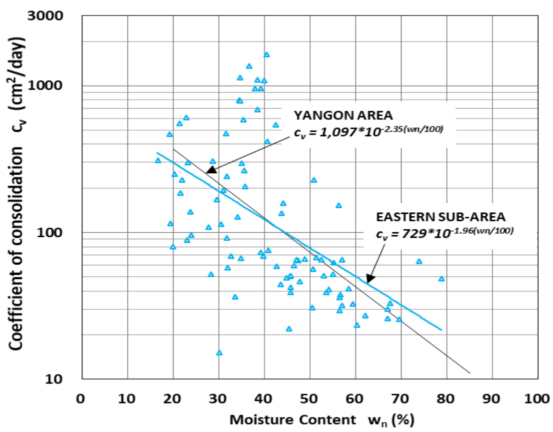
pc (kN/m2)
σvo´ (kN/m2)
YANGON AREA
Pc = 2.96 σvo´
WESTERN
SUB-AREA
Pc = 2.22 σvo´
Pc = σvo´
DOWNTOWN
SUB-AREA
Pc = 2.79 σvo´
Pc (kN/m2)
THANLYIN
SUB-AREA
Pc = 2.14 σvo´
EASTERN
SUB-AREA
Pc = 2.83 σvo´
CENTRAL
SUB-AREA
Pc = 4.37 σvo´
Central
Downtown
Thanlyin
Western
Eastern
North West
North East
JAPAN (Port &
harbor area)
Pc = 1.25 σvo´
MYANMAR
Pc = 1.62 σvo´

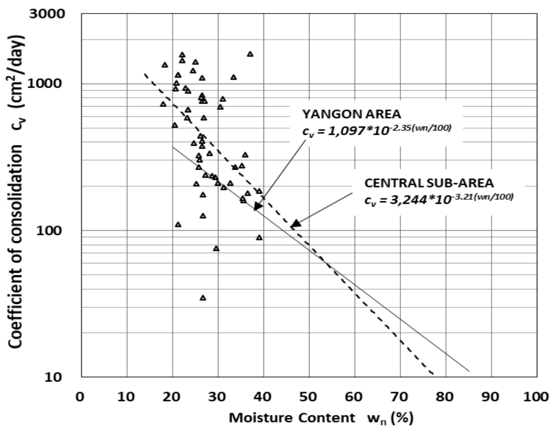

NORTH WEST SUB-AREA
$C_u = 0.291\,P_c$
YANGON AREA
$C_u = 0.185\,P_c$
WESTERN SUB-AREA
$C_u = 0.205\,P_c$
DOWNTOWN SUB-AREA
$C_u = 0.189\,P_c$
THANLYIN SUB-AREA
$C_u = 0.211\,P_c$
NORTH EAST SUB-AREA
$C_u = 0.158\,P_c$
EASTERN SUB-AREA
$C_u = 0.175\,P_c$
CENTRAL SUB-AREA
$C_u = 0.169\,P_c$
Central
Downtown
Thanlyin
Western
Eastern
North West
North East
C_u (= q_u/2) (kN/m^2)
P_c (kN/m^2)

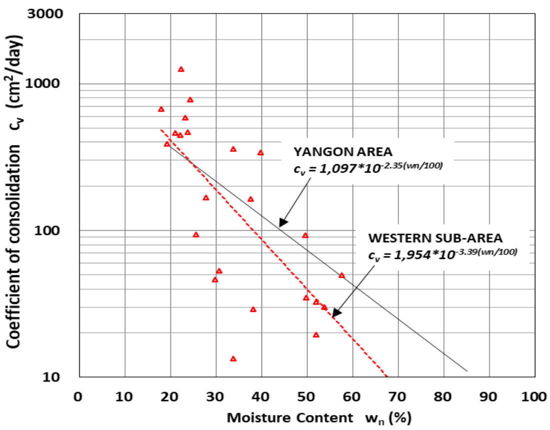

DOWNTOWN SUB-AREA
$c_v = 588*10^{-1.04(wn/100)}$
YANGON AREA
$c_v = 1,097*10^{-2.35(wn/100)}$
Coefficient of consolidation c_v (cm^2/day)
Moisture Content w_n (%)

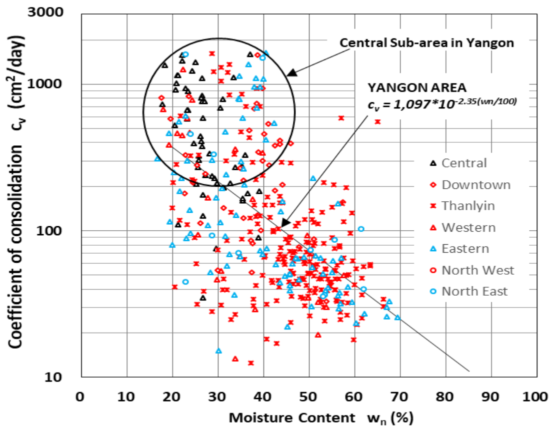

YANGON AREA
$c_v = 1,097*10^{-2.35(wn/100)}$
THANLYIN SUB-AREA
$c_v = 382*10^{-1.52(wn/100)}$
Coefficient of consolidation c_v (cm^2/day)
Moisture Content w_n (%)


NORTH EAST SUB-AREA
$c_v = 878*10^{-2.00(wn/100)}$
YANGON AREA
$c_v = 1,097*10^{-2.35(wn/100)}$
Coefficient of consolidation c_v (cm^2/day)
Moisture Content w_n (%)


YANGON AREA
$c_v = 1,097*10^{-2.35(wn/100)}$
EASTERN SUB-AREA
$c_v = 729*10^{-1.96(wn/100)}$
Coefficient of consolidation c_v (cm^2/day)
Moisture Content w_n (%)


YANGON AREA
$c_v = 1,097*10^{-2.35(wn/100)}$
CENTRAL SUB-AREA
$c_v = 3,244*10^{-3.21(wn/100)}$
Coefficient of consolidation c_v (cm^2/day)
Moisture Content w_n (%)


YANGON AREA
$c_v = 1,097*10^{-2.35(wn/100)}$
WESTERN SUB-AREA
$c_v = 1,954*10^{-3.39(wn/100)}$
Coefficient of consolidation c_v (cm^2/day)
Moisture Content w_n (%)


Central Sub-area in Yangon
YANGON AREA
$c_v = 1,097*10^{-2.35(wn/100)}$
Central
Downtown
Thanlyin
Western
Eastern
North West
North East
Coefficient of consolidation c_v (cm^2/day)
Moisture Content w_n (%)

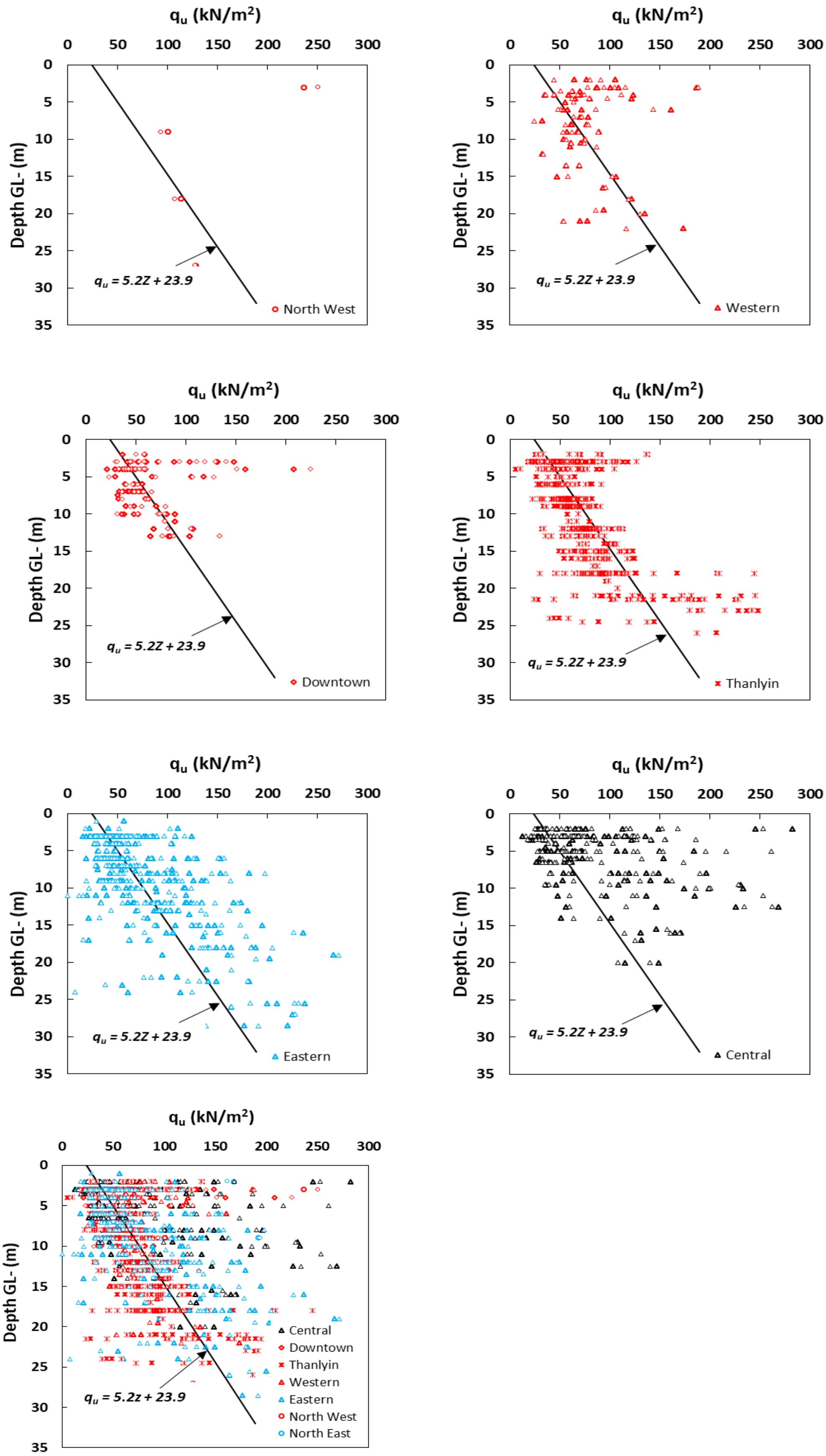


Figure 1 Correlation Graphs between Physical and Mechanical Properties of Yangon Clays (Sub-area wise graphs)

APPENDIX “D”

PHOTOS OF YANGON CLAYS

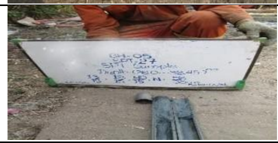
EW-1 Section

Layer Names	Ahlone Area	Kyauktada Area	Thaketa Area
Alluvial CLAY-I (AC-I)			
Alluvial CLAY-II (AC-II)			
Alluvial SAND-I (AS-I)			
Alluvial SAND-II (AS-II)			
Alluvial SAND-III (AS-III)			
CLAY-I (PIC-I)			
CLAY-II (PIC-II)			
SAND-II (PIS-II)			

EW-2 Section

Layer Names	Kamaryut Area	Tamwe Area	Dagon Seikkan Area
Alluvial CLAY-I (AC-I)			
Alluvial CLAY-II (AC-II)			
Alluvial CLAY-III (AC-III)			
Alluvial SAND-I (AS-I)			
Alluvial SAND-II (AS-II)			
CLAY-I (PIC-I)			
CLAY-II (PIC-II)			
SAND-II (PIS-II)			

EW-3 Section

Layer Names	Mayangone Area	Yankin Area	North Dagon Area
Alluvial CLAY-I (AC-I)			
Alluvial CLAY-II (AC-II)			
Alluvial CLAY-III (AC-III)			
Alluvial SAND-I (AS-I)			
Alluvial SAND-II (AS-II)			
CLAY-I (PIC-I)			
SAND-I (PIS-I)			
SAND-II (PIS-II)			

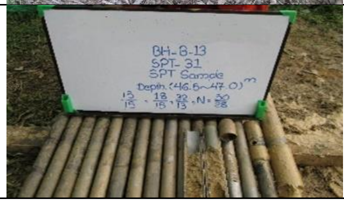
EW-4 Section

Layer Names	Hlaing Tharyar Area	Mayangone Area	North Dagon Area
Alluvial CLAY-I (AC-I)			
Alluvial CLAY-II (AC-II)			
Alluvial SAND-II (AS-II)			
Alluvial SAND-III (AS-III)			
CLAY-II (PIC-II)			
SAND-I (PIS-I)			

EW-5 Section

Layer Names	Hlaing Tharyar Area	Danyingone Area	Mingalardon Area
Alluvial CLAY-I (AC-I)			
Alluvial CLAY-II (AC-II)			
Alluvial SAND-I (AS-I)			
CLAY-I (PIC-I)			
CLAY-II (PIC-II)			
SAND-I (PIS-I)			
SAND-II (PIS-II)			

NS-1 Section

Layer Names	Insein Area	Mayangone Area	Ahlone Area
Alluvial CLAY-II (AC-II)			
Alluvial CLAY-III (AC-III)			
Alluvial SAND-I (AS-I)			
SAND-I (PIS-I)			
SAND-II (PIS-II)			

NS-2 Section

Layer Names	Mingalardon Area	Mayangone Area	Tamwe Area
Alluvial CLAY-I (AC-I)			
Alluvial CLAY-II (AC-II)			
Alluvial CLAY-III (AC-III)			
Alluvial SAND-I (AS-I)			
CLAY-I (PIC-I)			
CLAY-II (PIC-II)			
SAND-I (PIS-I)			
SAND-II (PIS-II)			

APPENDIX “E”

DISTRIBUTION GRAPHS WITH DEPTH FOR PHYSICAL AND MECHANICAL PROPERTIES OF THILAWA CLAYS

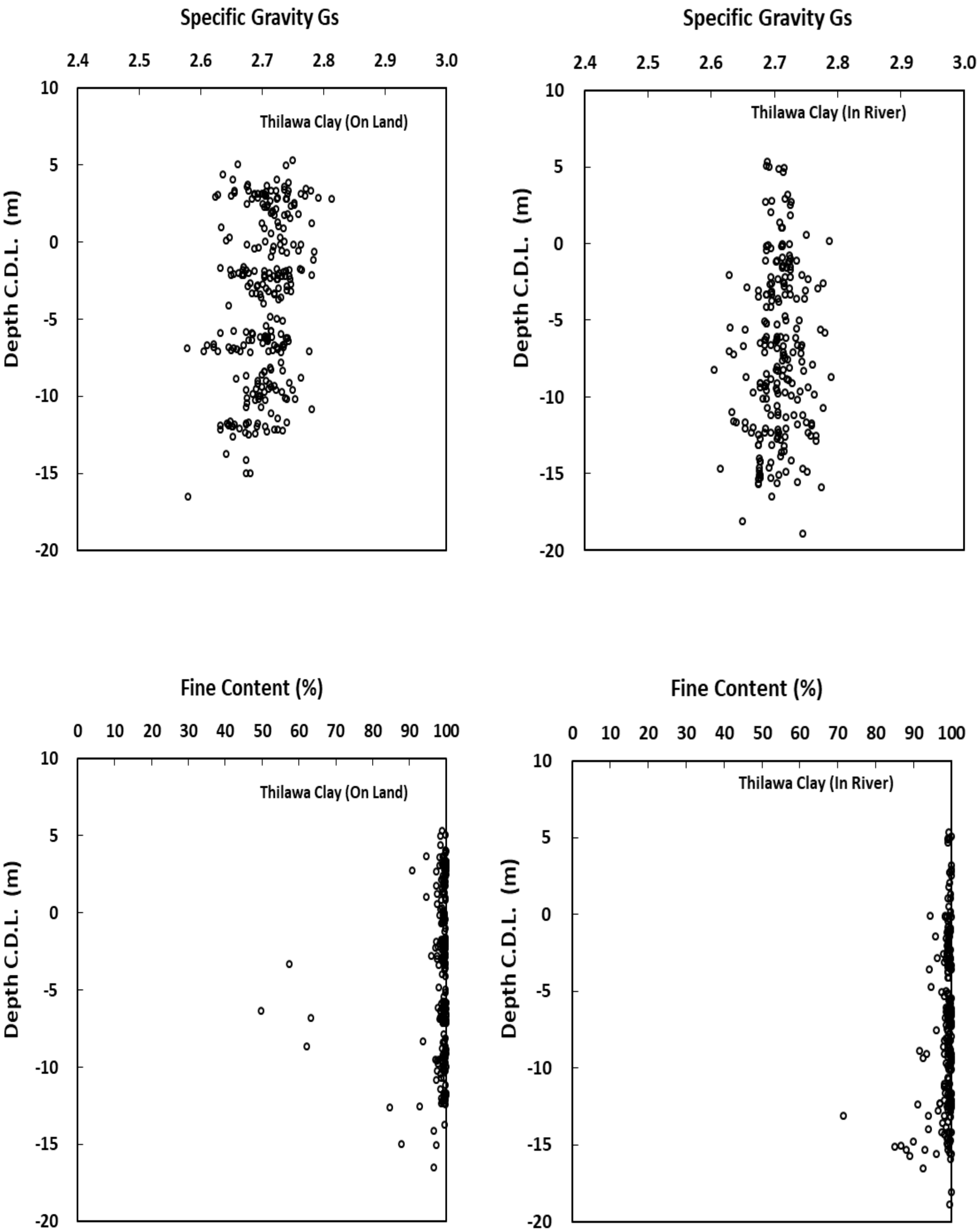


Figure 1 Physical Properties of Thilawa areas Graphs

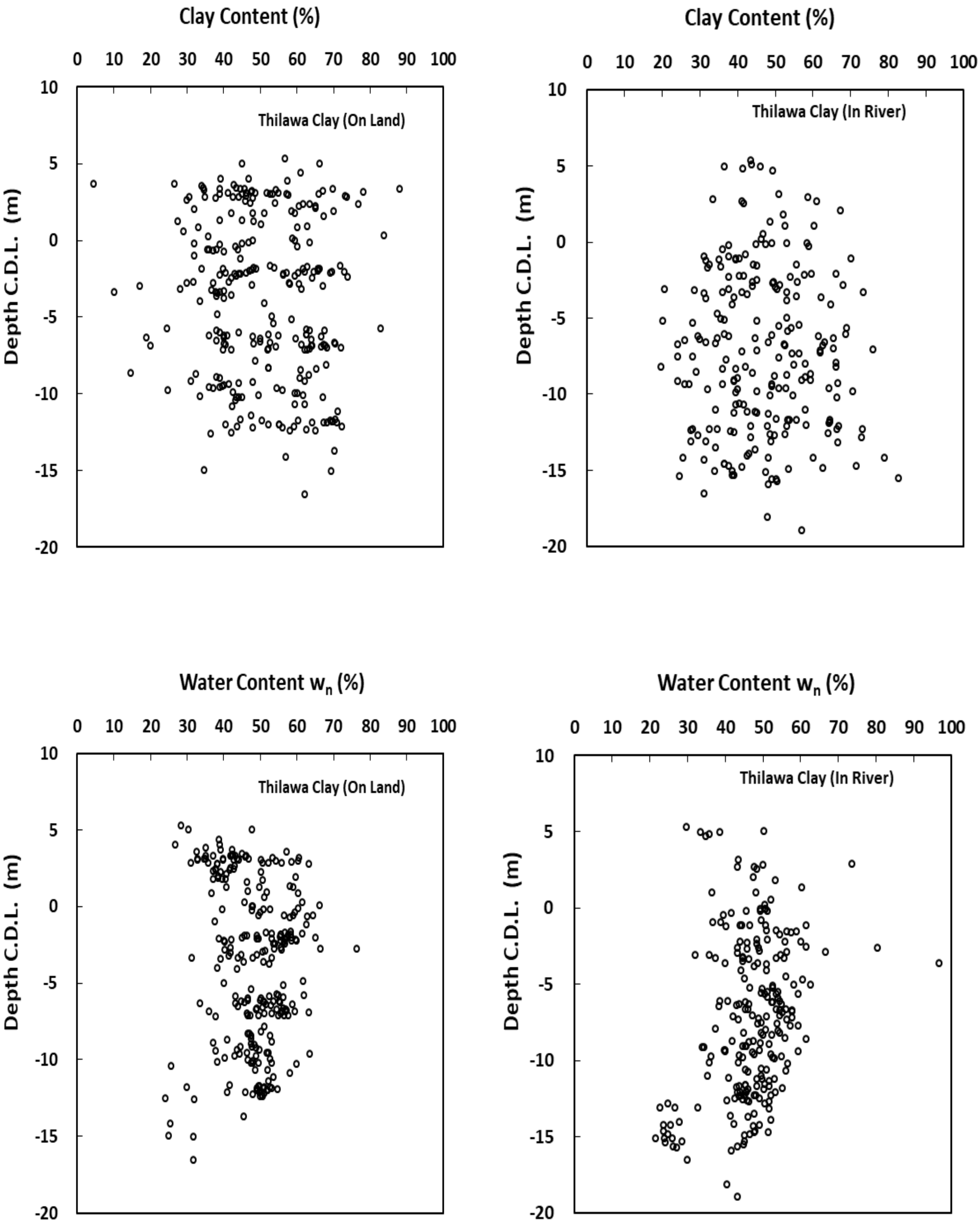


Figure 1 Physical Properties of Thilawa areas Graphs

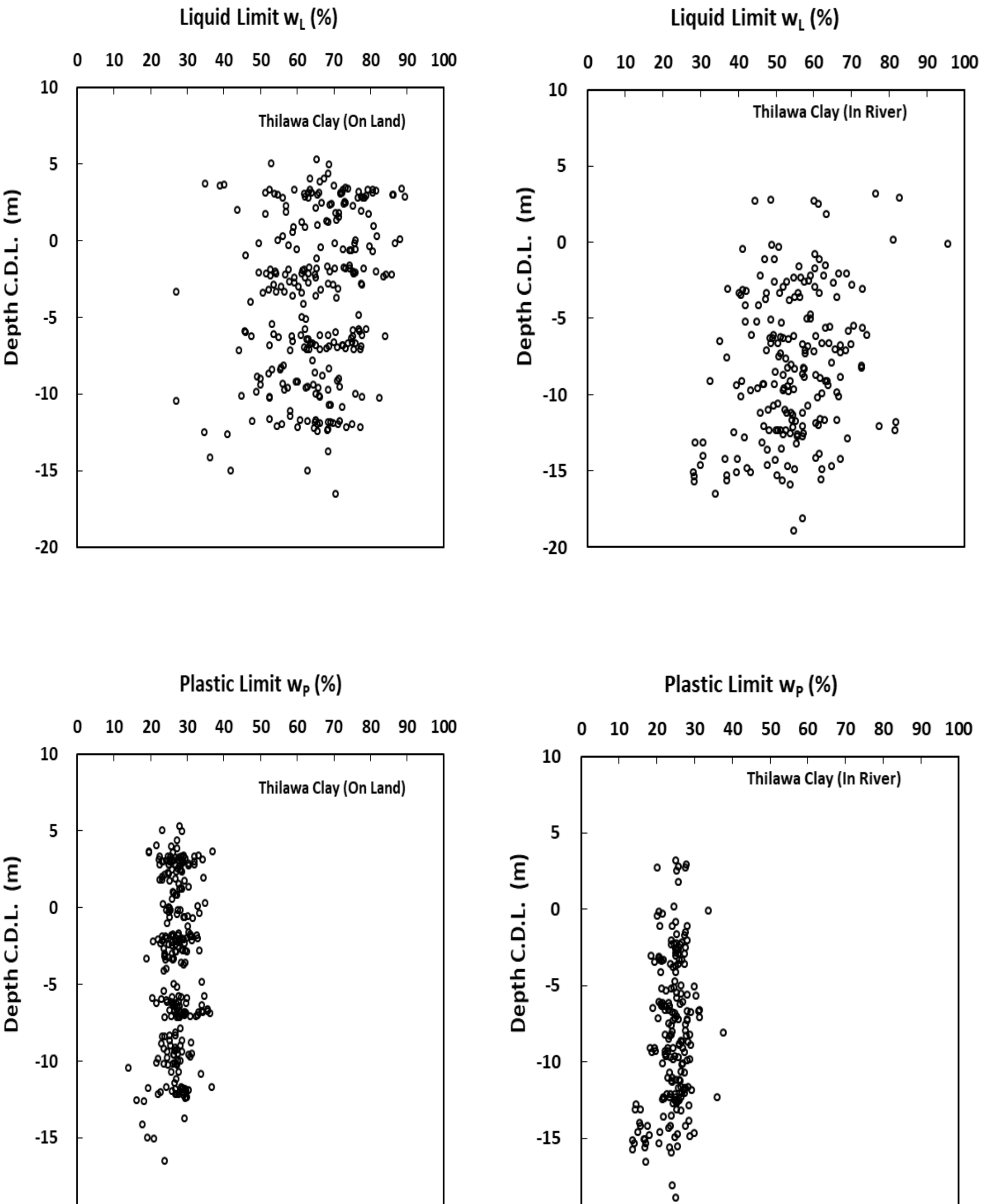


Figure 1 Physical Properties of Thilawa areas Graphs

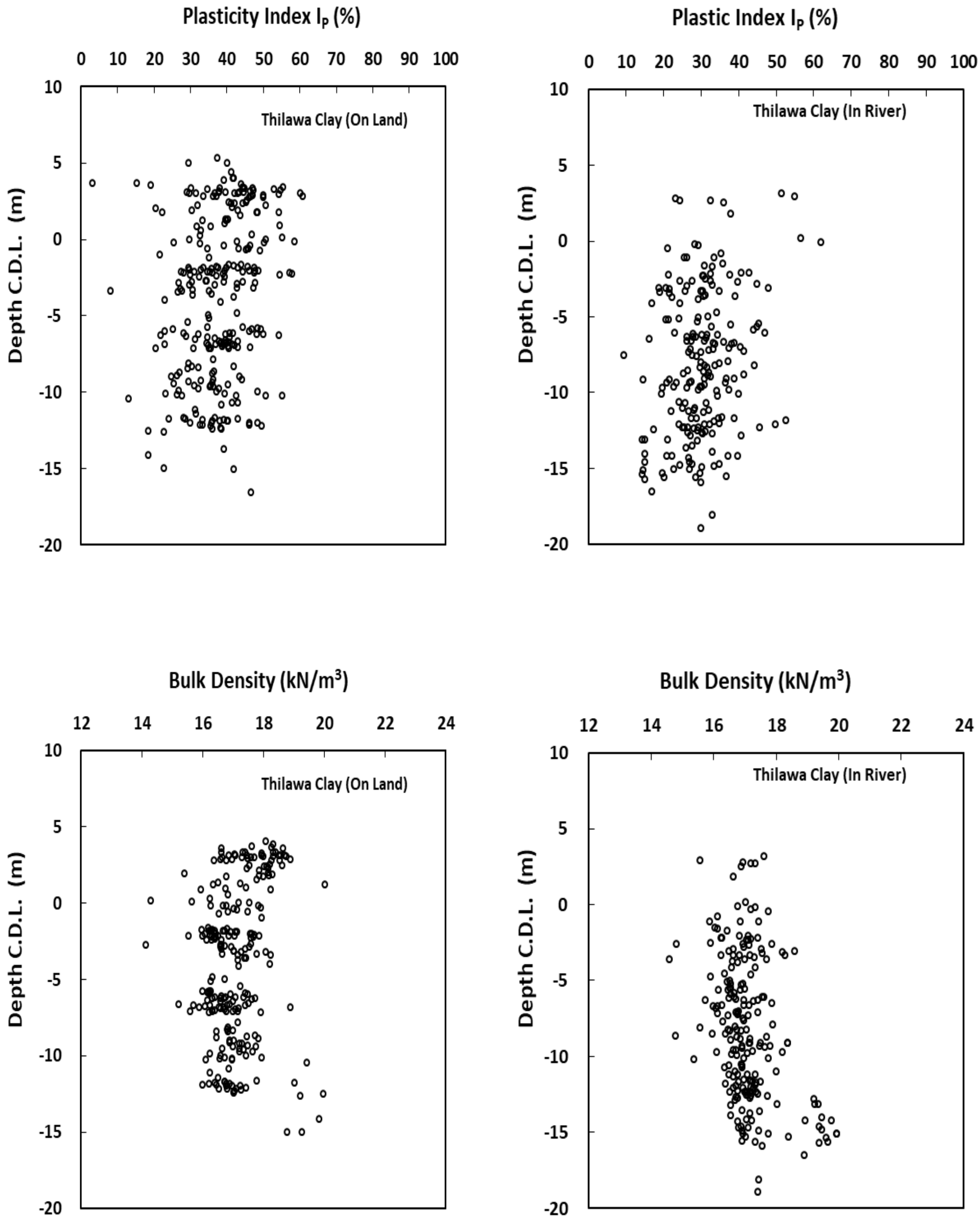


Figure 1 Physical Properties of Thilawa areas Graphs

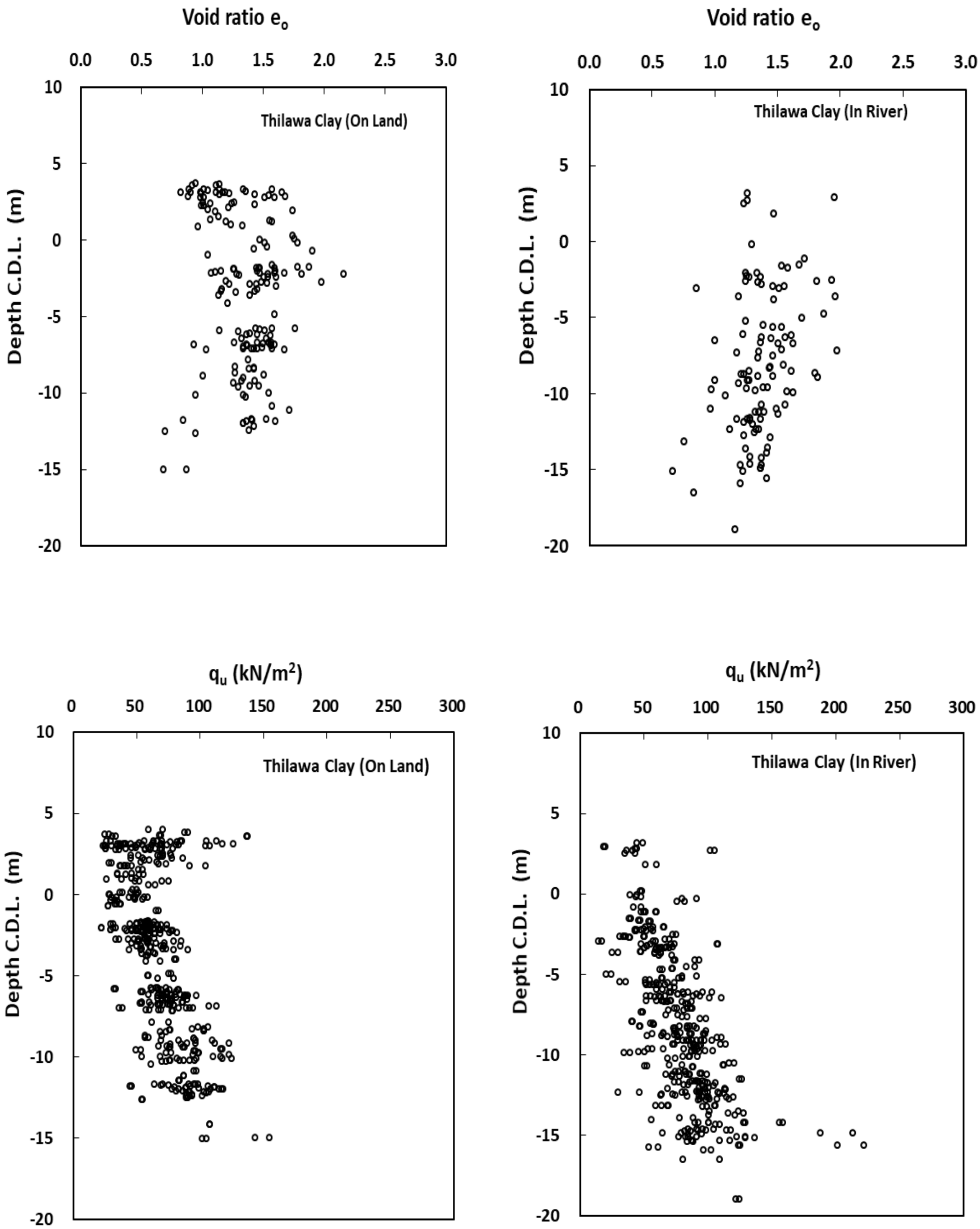


Figure 2 Mechanical Properties of Thilawa areas Graphs

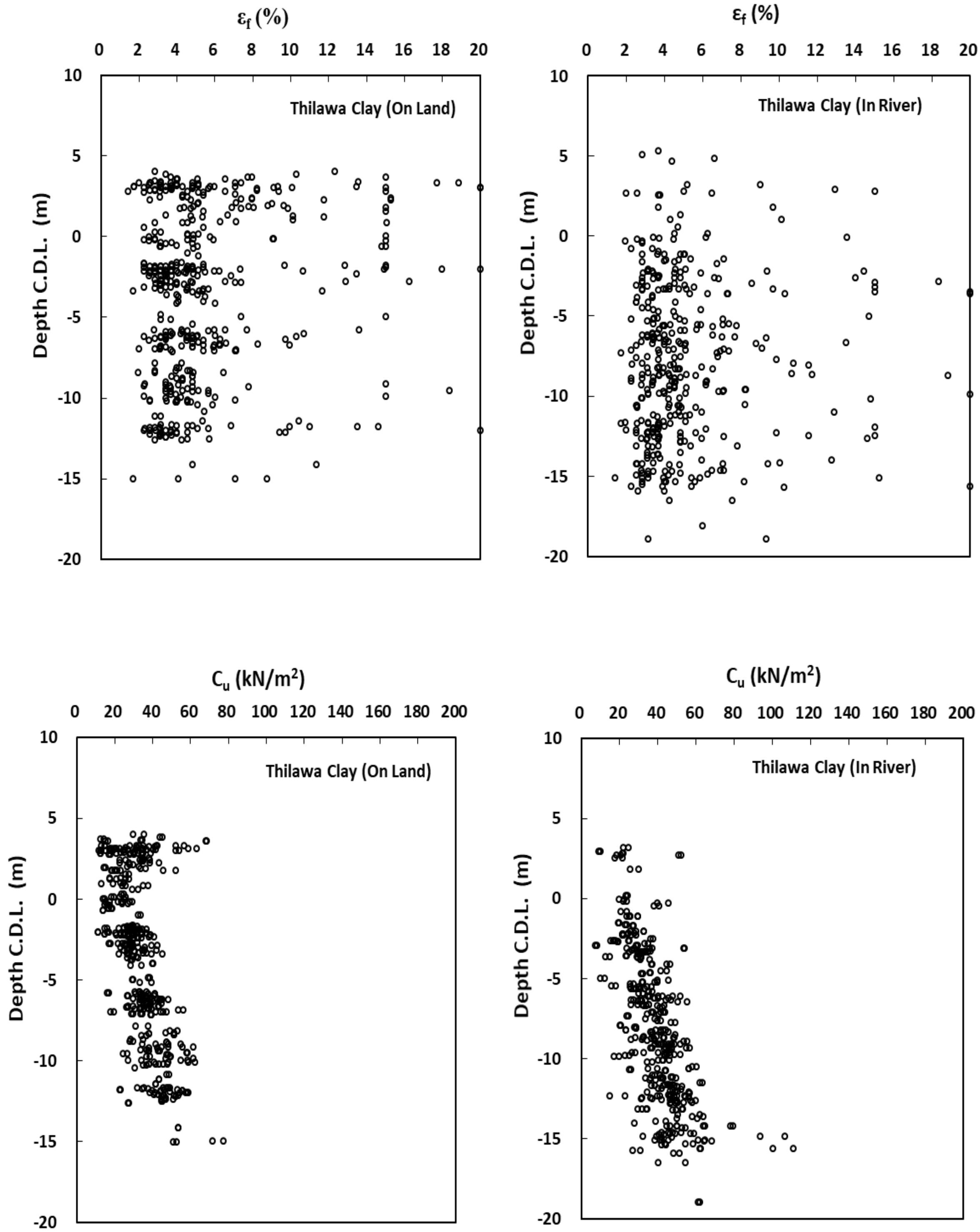


Figure 2 Mechanical Properties of Thilawa areas Graphs

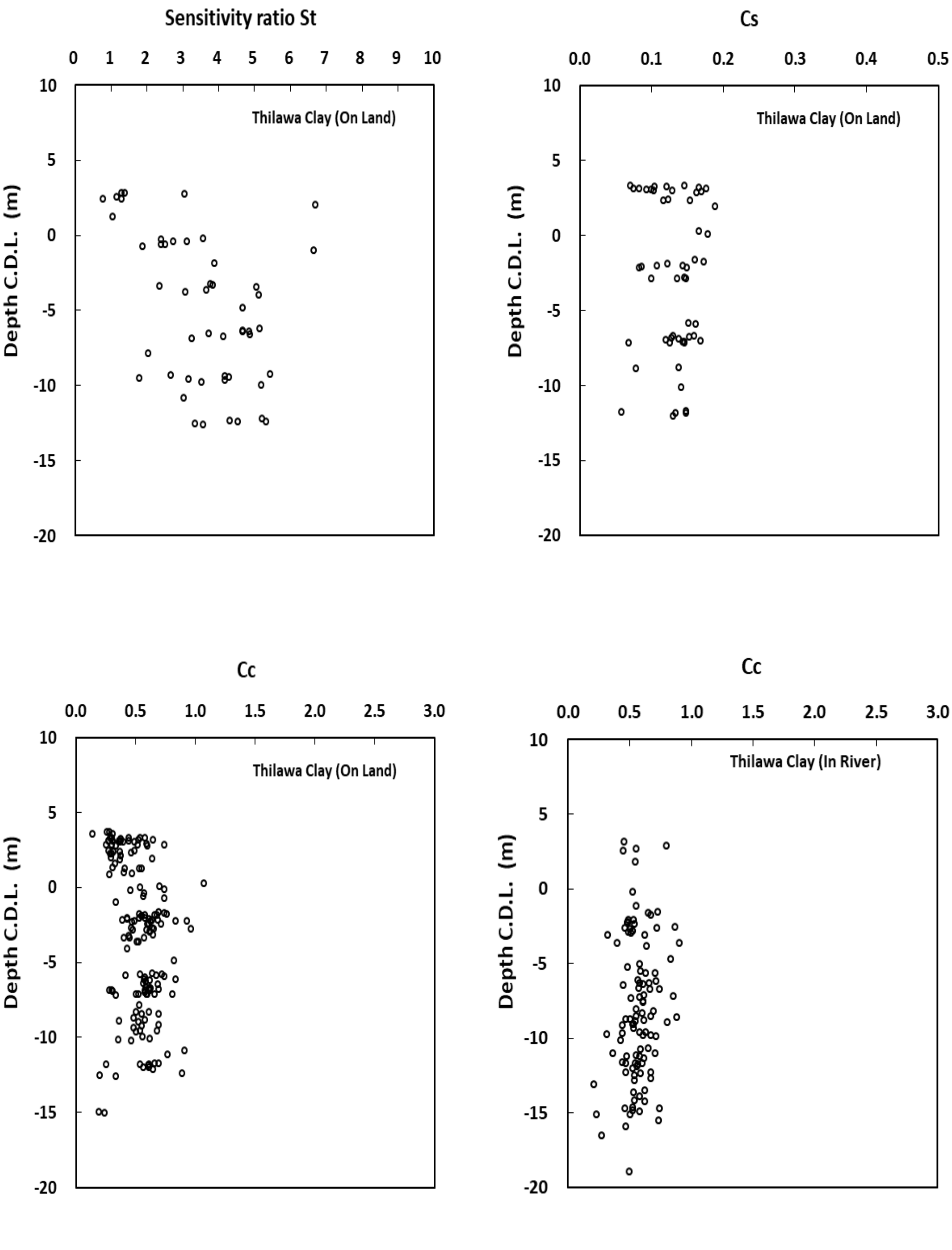


Figure 2 Mechanical Properties of Thilawa areas Graphs

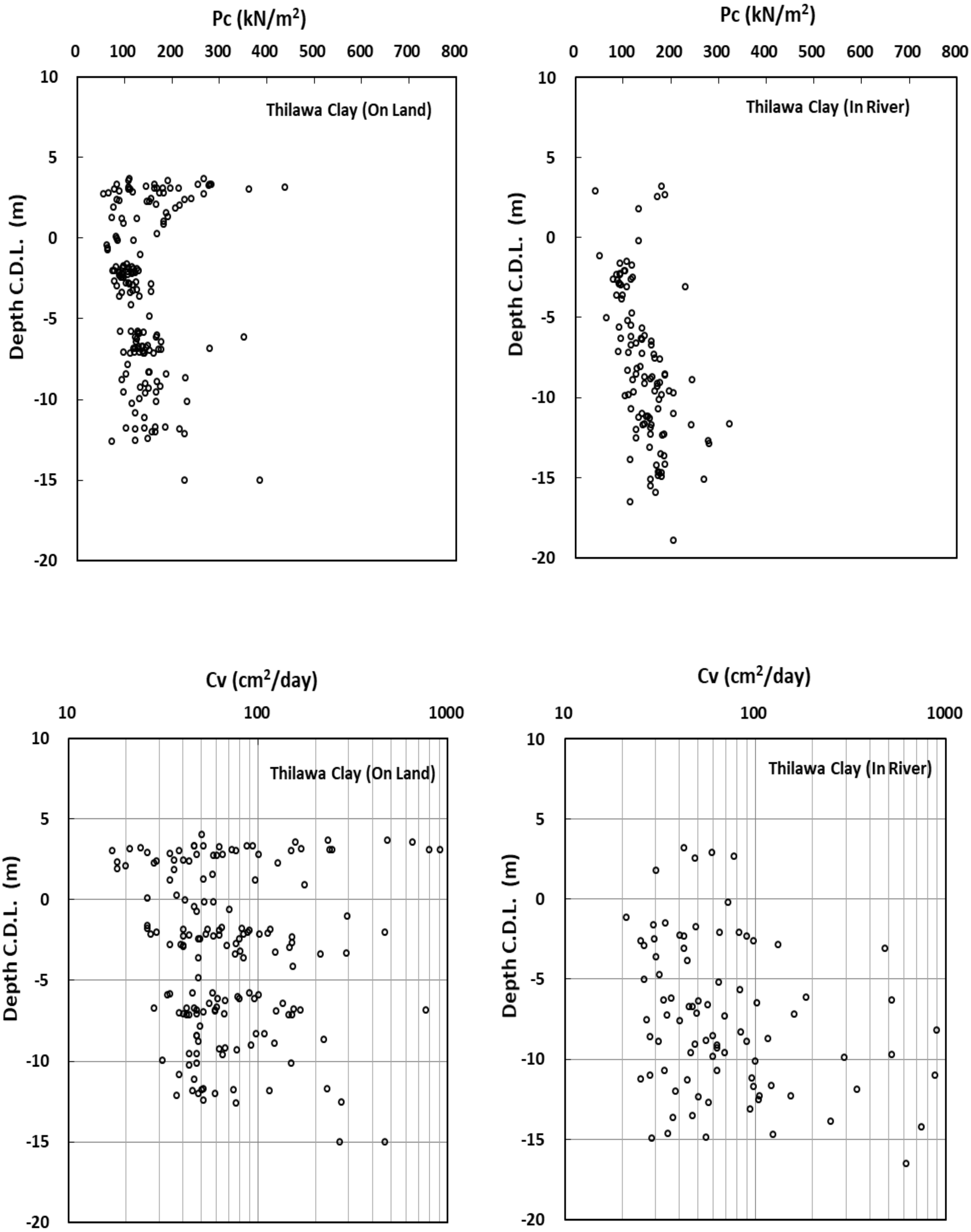


Figure 2 Mechanical Properties of Thilawa areas Graphs

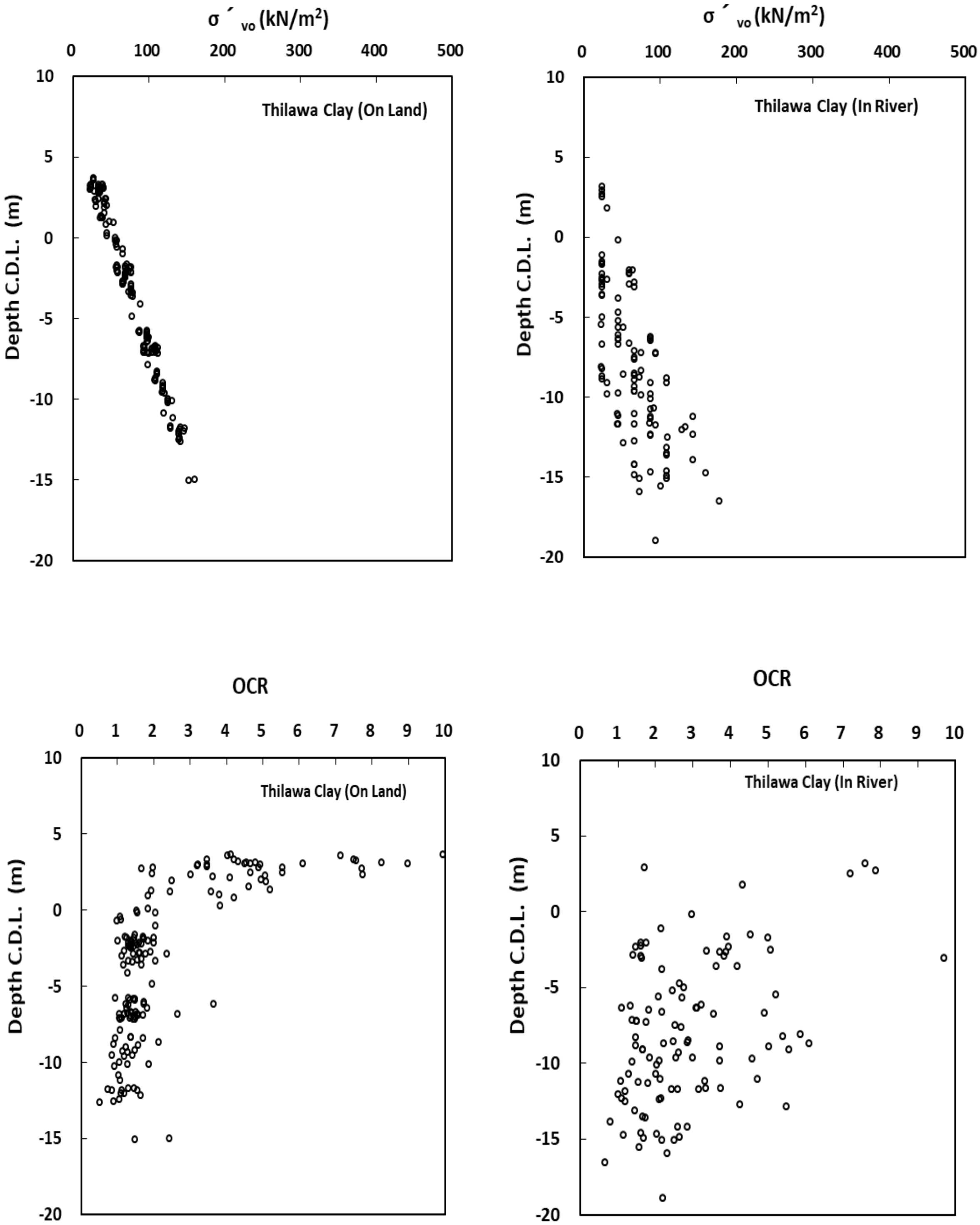
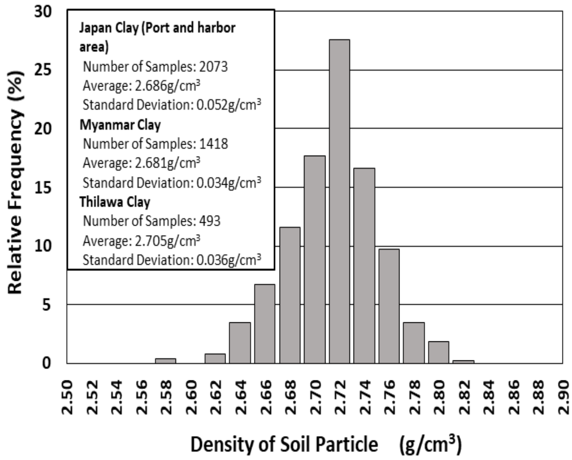
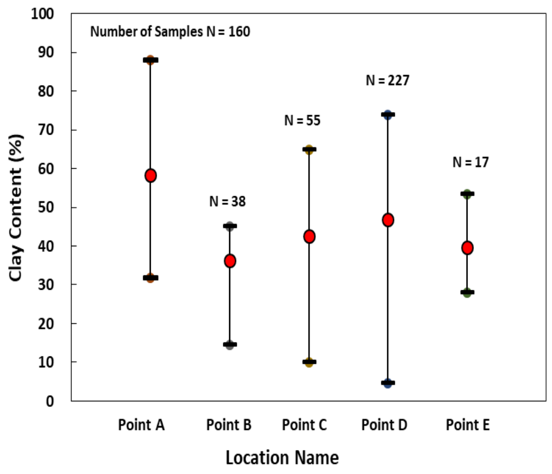


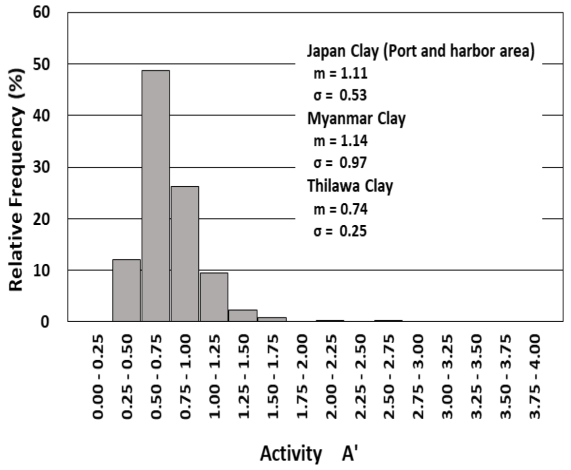
Figure 2 Mechanical Properties of Thilawa areas Graphs

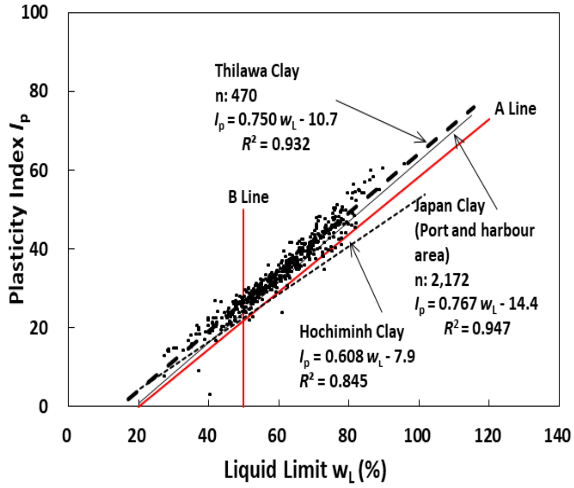
APPENDIX “F”

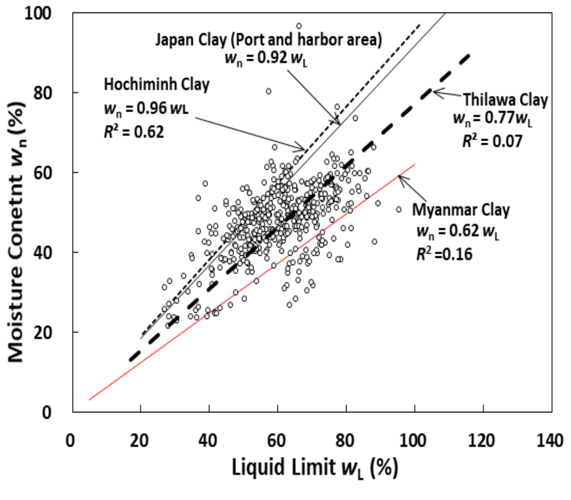
CORRELATION GRAPHS AND DETERMINATION FACTORS BETWEEN PHYSICAL AND MECHANICAL PROPERTIES OF THILAWA CLAYS

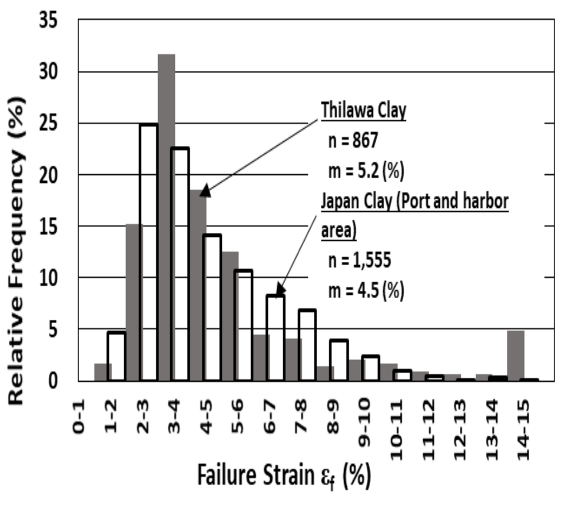

Japan Clay (Port and harbor area)
Number of Samples: 2073
Average: 2.686g/cm3
Standard Deviation: 0.052g/cm3
Myanmar Clay
Number of Samples: 1418
Average: 2.681g/cm3
Standard Deviation: 0.034g/cm3
Thilawa Clay
Number of Samples: 493
Average: 2.705g/cm3
Standard Deviation: 0.036g/cm3
Relative Frequency (%)
Density of Soil Particle (g/cm3)

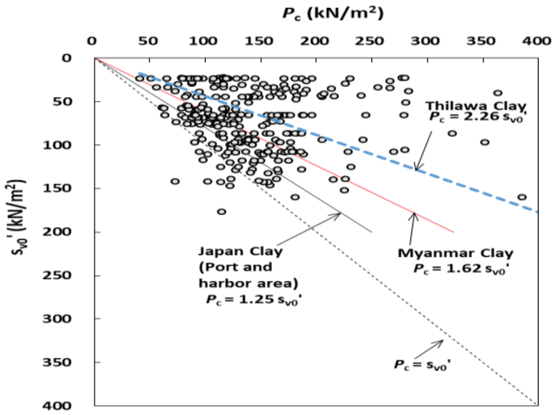

Number of Samples N = 160
N = 38
N = 55
N = 227
N = 17
Clay Content (%)
Point A
Point B
Point C
Point D
Point E
Location Name


Japan Clay (Port and harbor area)
m = 1.11
σ = 0.53
Myanmar Clay
m = 1.14
σ = 0.97
Thilawa Clay
m = 0.74
σ = 0.25
Relative Frequency (%)
Activity A'

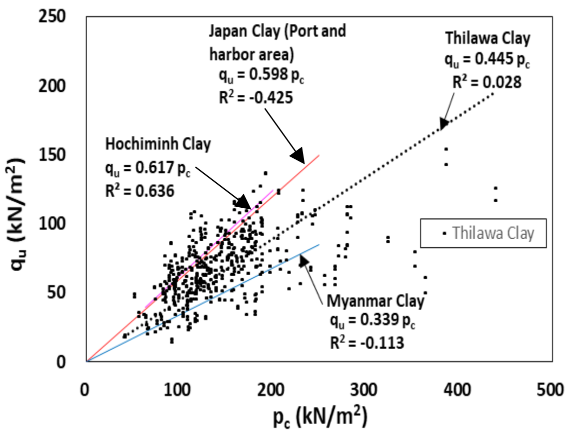

Thilawa Clay
n: 470
Ip = 0.750 wL - 10.7
R2 = 0.932
A Line
B Line
Japan Clay (Port and harbour area)
n: 2,172
Ip = 0.767 wL - 14.4
R2 = 0.947
Hochiminh Clay
Ip = 0.608 wL - 7.9
R2 = 0.845
Plasticity Index Ip
Liquid Limit wL (%)


Japan Clay (Port and harbor area)
wn = 0.92 wL
Hochiminh Clay
wn = 0.96 wL
R2 = 0.62
Thilawa Clay
wn = 0.77wL
R2 = 0.07
Myanmar Clay
wn = 0.62 wL
R2 =0.16
Moisture Conetnt wn (%)
Liquid Limit wL (%)

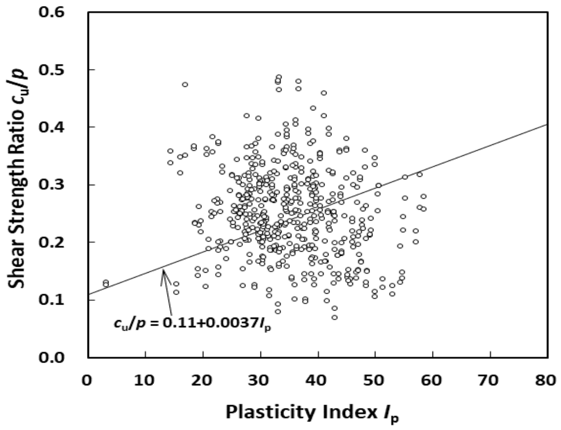

Thilawa Clay
n = 867
m = 5.2 (%)
Japan Clay (Port and harbor area)
n = 1,555
m = 4.5 (%)
Relative Frequency (%)
Failure Strain εf (%)



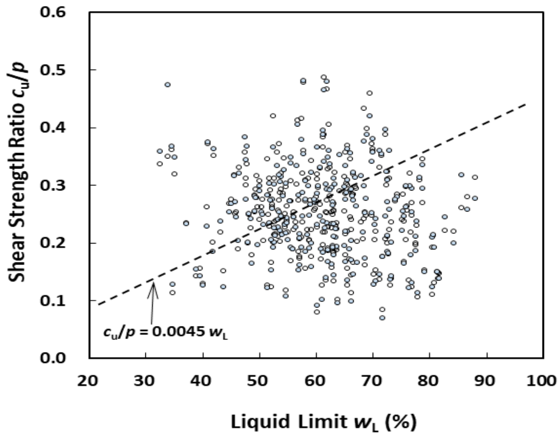
P_c (kN/m²)
s_v0' (kN/m²)
Thilawa Clay
P_c = 2.26 s_v0'
Myanmar Clay
P_c = 1.62 s_v0'
Japan Clay
(Port and
harbor area)
P_c = 1.25 s_v0'
P_c = s_v0'



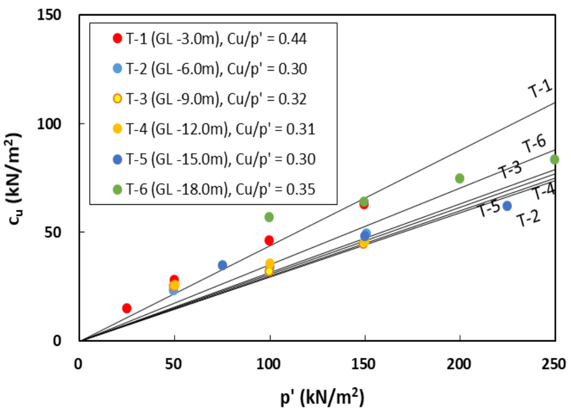
q_u (kN/m²)
p_c (kN/m²)
Japan Clay (Port and
harbor area)
q_u = 0.598 p_c
R² = -0.425
Thilawa Clay
q_u = 0.445 p_c
R² = 0.028
Hochiminh Clay
q_u = 0.617 p_c
R² = 0.636
Myanmar Clay
q_u = 0.339 p_c
R² = -0.113
Thilawa Clay



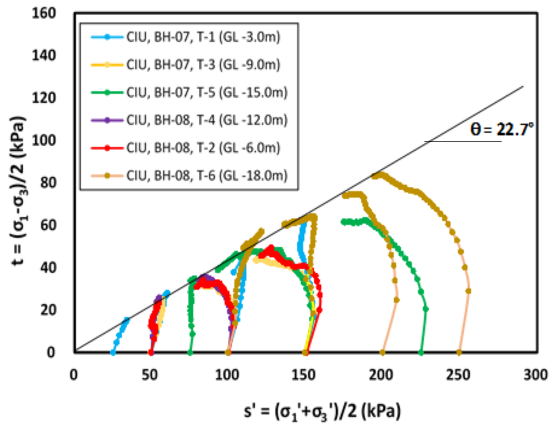
Shear Strength Ratio c_u/p
Plasticity Index I_p
c_u/p = 0.11+0.0037I_p



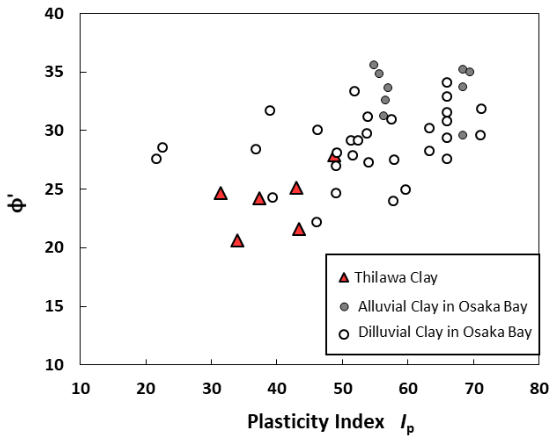
Shear Strength Ratio c_u/p
Liquid Limit w_L (%)
c_u/p = 0.0045 w_L



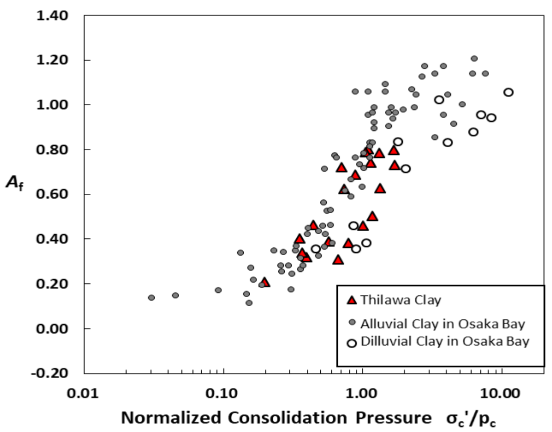
c_u (kN/m²)
p' (kN/m²)
T-1 (GL -3.0m), Cu/p' = 0.44
T-2 (GL -6.0m), Cu/p' = 0.30
T-3 (GL -9.0m), Cu/p' = 0.32
T-4 (GL -12.0m), Cu/p' = 0.31
T-5 (GL -15.0m), Cu/p' = 0.30
T-6 (GL -18.0m), Cu/p' = 0.35
T-1
T-6
T-3
T-4
T-5
T-2



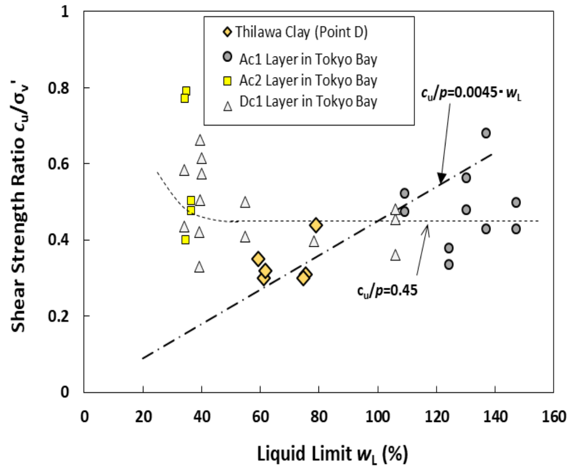
t = (σ1-σ3)/2 (kPa)
s' = (σ1'+σ3')/2 (kPa)
CIU, BH-07, T-1 (GL -3.0m)
CIU, BH-07, T-3 (GL -9.0m)
CIU, BH-07, T-5 (GL -15.0m)
CIU, BH-08, T-4 (GL -12.0m)
CIU, BH-08, T-2 (GL -6.0m)
CIU, BH-08, T-6 (GL -18.0m)
θ = 22.7°

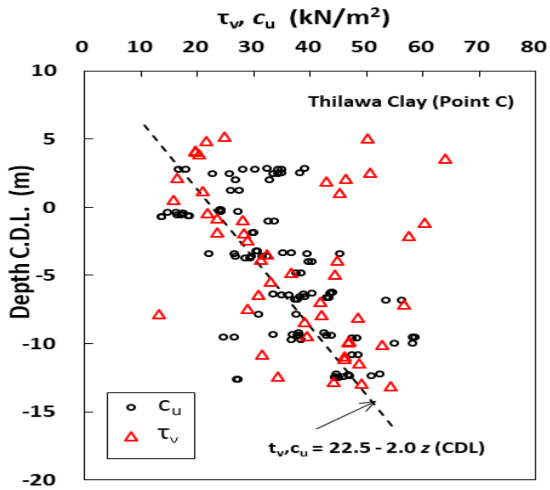


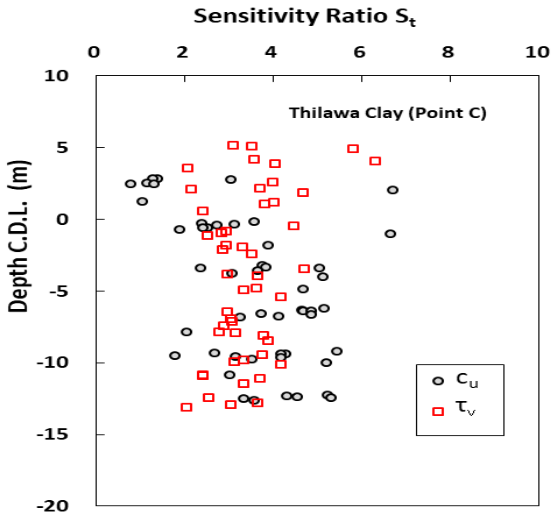
φ'
Plasticity Index I_p
Thilawa Clay
Alluvial Clay in Osaka Bay
Dilluvial Clay in Osaka Bay

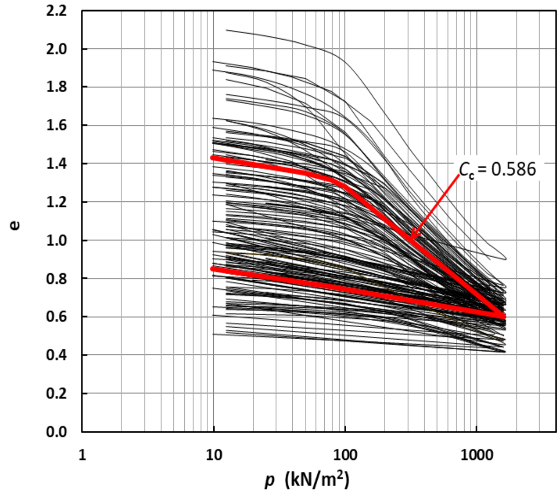


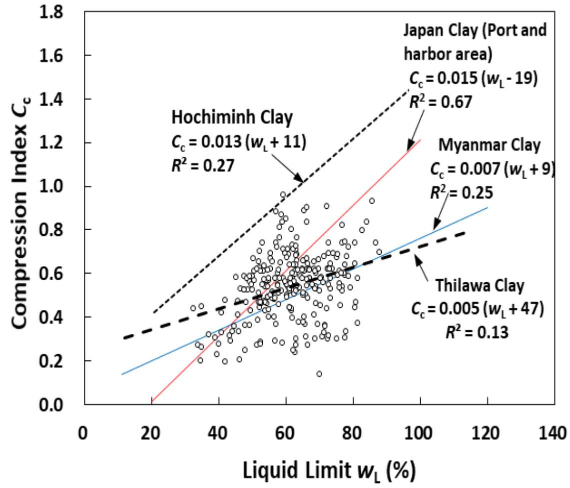
A_f
Normalized Consolidation Pressure σ_c'/p_c
Thilawa Clay
Alluvial Clay in Osaka Bay
Dilluvial Clay in Osaka Bay

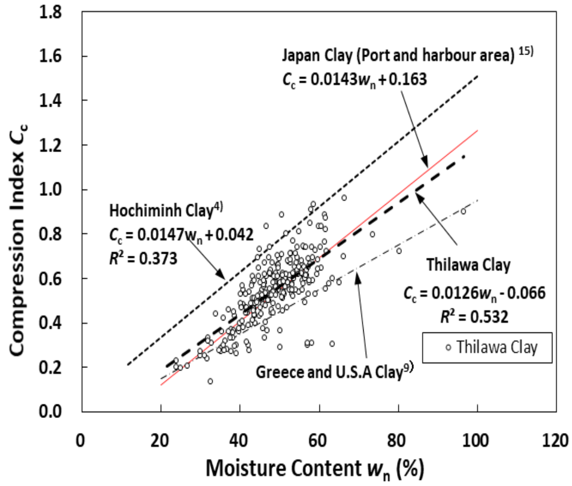

Thilawa Clay (Point D)
Ac1 Layer in Tokyo Bay
Ac2 Layer in Tokyo Bay
Dc1 Layer in Tokyo Bay
c_u/p=0.0045・w_L
c_u/p=0.45
Shear Strength Ratio c_u/σ_v'
Liquid Limit w_L (%)

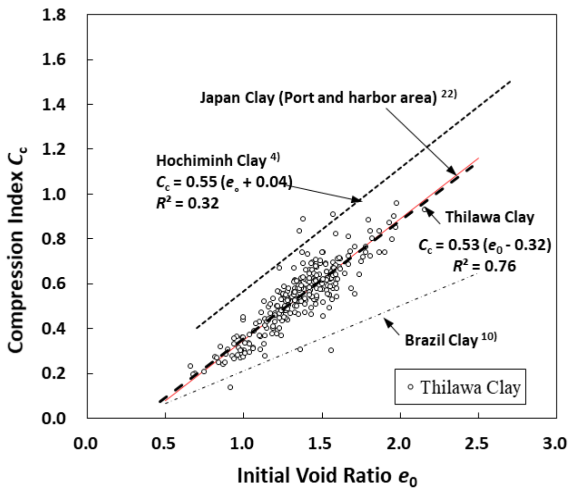

τ_v, c_u (kN/m²)
Thilawa Clay (Point C)
Depth C.D.L. (m)
c_u
τ_v
t_v,c_u = 22.5 - 2.0 z (CDL)

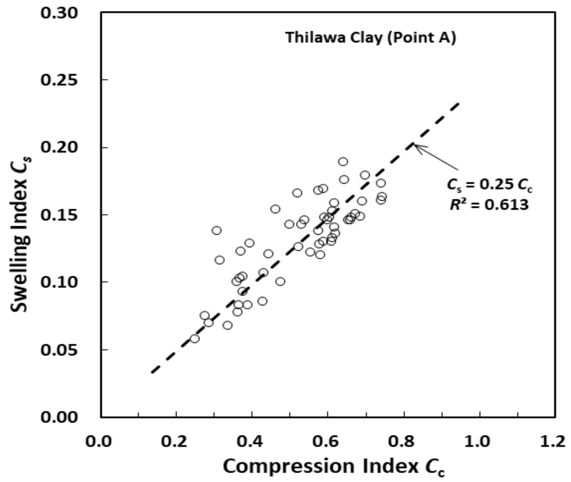

Sensitivity Ratio S_t
Thilawa Clay (Point C)
Depth C.D.L. (m)
c_u
τ_v

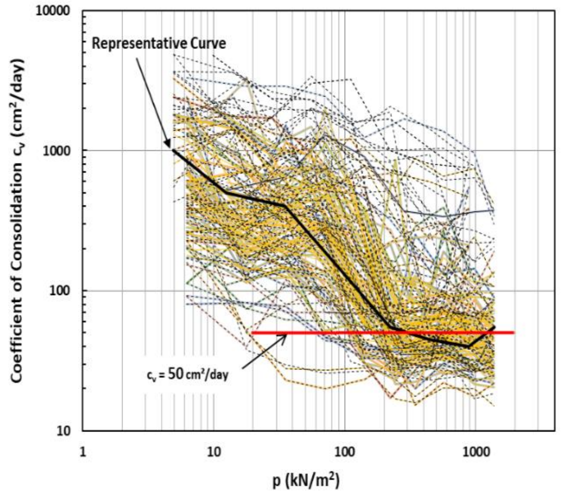

C_c = 0.586
e
p (kN/m²)

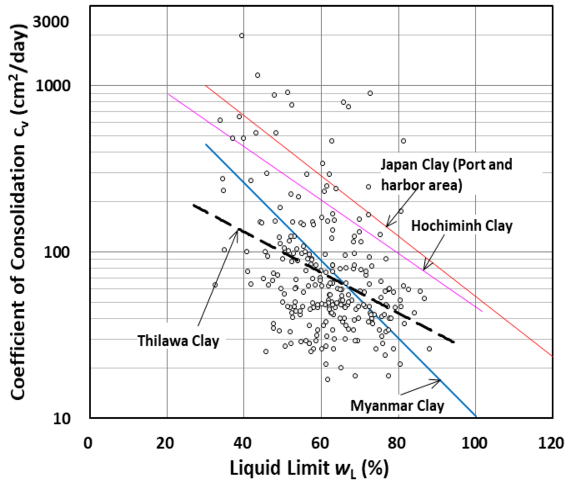

Japan Clay (Port and harbor area)
C_c = 0.015 (w_L - 19)
R^2 = 0.67
Hochiminh Clay
C_c = 0.013 (w_L + 11)
R^2 = 0.27
Myanmar Clay
C_c = 0.007 (w_L + 9)
R^2 = 0.25
Thilawa Clay
C_c = 0.005 (w_L + 47)
R^2 = 0.13
Compression Index C_c
Liquid Limit w_L (%)

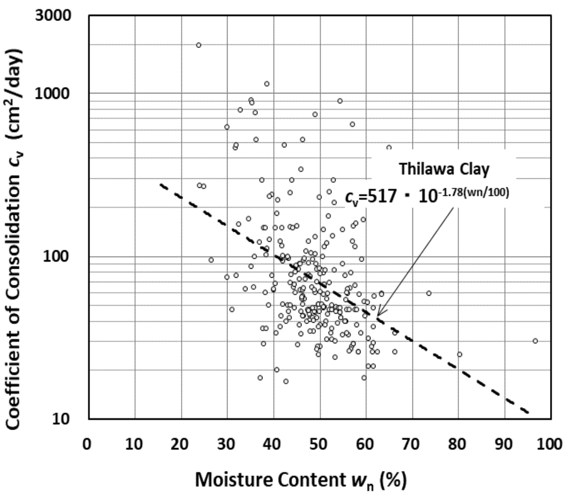

Japan Clay (Port and harbour area) 15)
C_c = 0.0143w_n + 0.163
Hochiminh Clay4)
C_c = 0.0147w_n + 0.042
R^2 = 0.373
Thilawa Clay
C_c = 0.0126w_n - 0.066
R^2 = 0.532
Thilawa Clay
Greece and U.S.A Clay9)
Compression Index C_c
Moisture Content w_n (%)

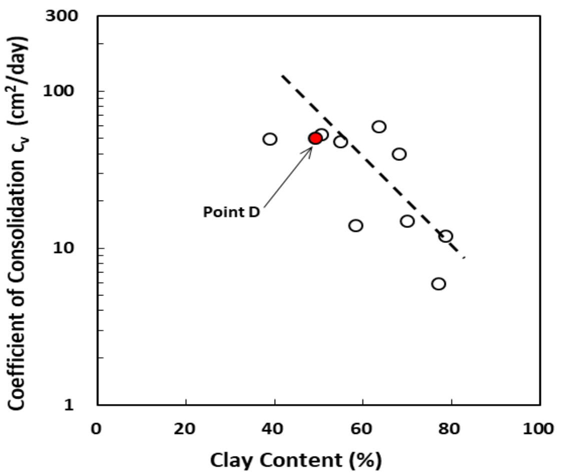

Japan Clay (Port and harbor area) 22)
Hochiminh Clay 4)
$C_c = 0.55 (e_o + 0.04)$
$R^2 = 0.32$
Thilawa Clay
$C_c = 0.53 (e_0 - 0.32)$
$R^2 = 0.76$
Brazil Clay 10)
Thilawa Clay
Compression Index C_c
Initial Void Ratio e_0


Thilawa Clay (Point A)
$C_s = 0.25\ C_c$
$R^2 = 0.613$
Swelling Index C_s
Compression Index C_c


Representative Curve
$c_v = 50$ cm²/day
Coefficient of Consolidation c_v (cm²/day)
p (kN/m²)


Japan Clay (Port and harbor area)
Hochiminh Clay
Thilawa Clay
Myanmar Clay
Coefficient of Consolidation c_v (cm²/day)
Liquid Limit w_L (%)


Thilawa Clay
$c_v = 517 \cdot 10^{-1.78(wn/100)}$
Coefficient of Consolidation c_v (cm²/day)
Moisture Content w_n (%)


Point D
Coefficient of Consolidation c_v (cm²/day)
Clay Content (%)

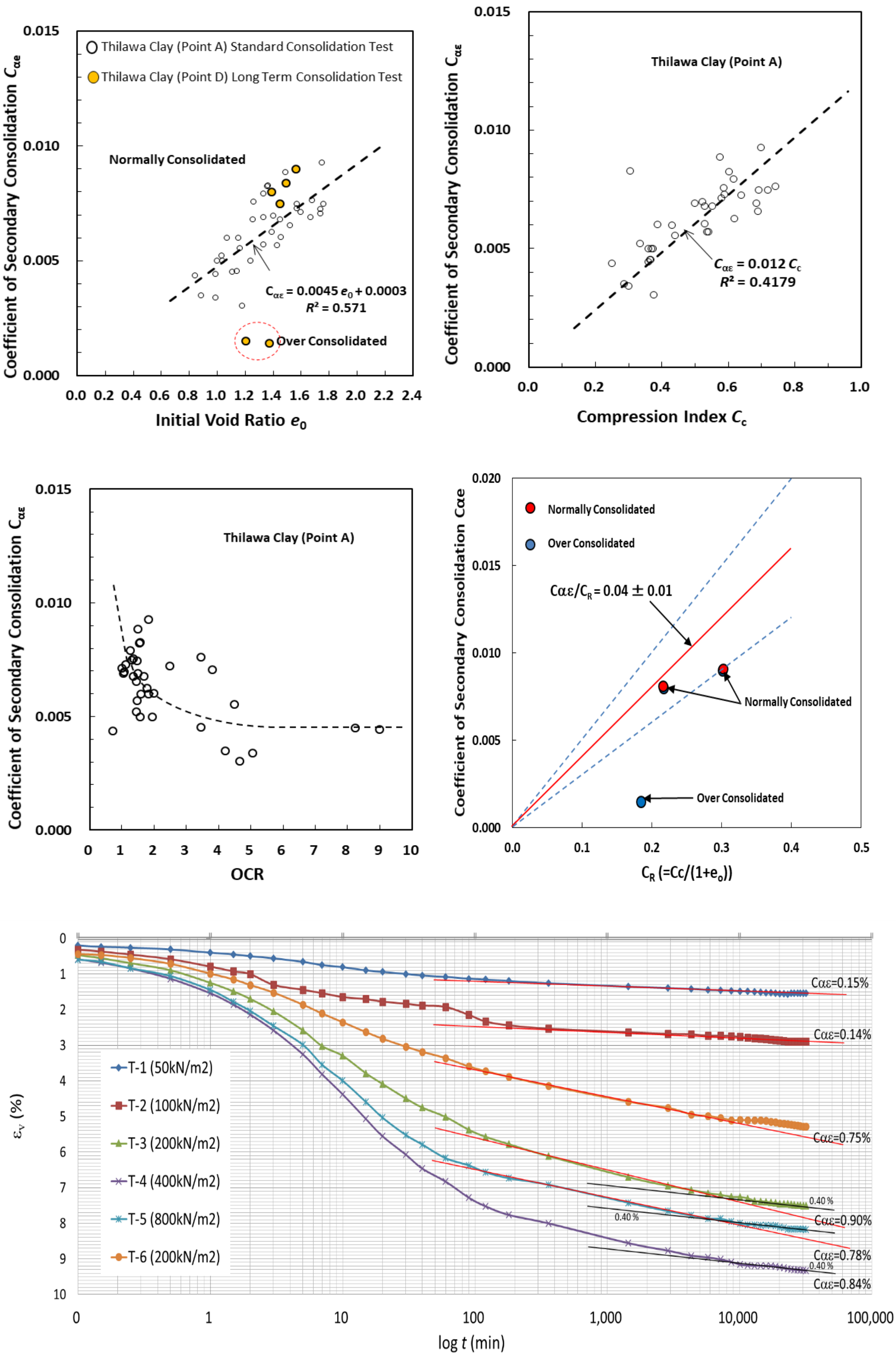


Figure 1 Correlation Graphs and Determination Factors between Physical and Mechanical Properties of Thilawa Clays

APPENDIX “G”

TABLE FOR REGRESSION LINE AND COEFFICIENT OF DETERMINATION R^2 BETWEEN PHYSICAL PROPERTIES AND MECHANICAL PROPERTIES WITH POINT-WISE FOR THILAWA CLAYS

Table for Regression line and Coefficient of Determination R2 between Physical Porperties and Mechanical Properties with Point-wise for Thilawa clays (1/2)

Point Number	Land or River	Formula	Wu ~ Wl	Wu ~ Wp	Wu ~ Ip	Wl ~ Wp	Wl ~ Ip	Wp ~ Ip
A	In the river	y = ax + b	0.3196x + 20.741	1.9737x - 10.791	0.2942x + 29.459	2.9749x - 17.035	1.1815x + 19.402	0.1975x + 18.799
		R^2	0.1995	0.524	0.1209	0.5537	0.9593	0.4051
	On Land	y = ax + b	0.5649x + 6.3487	2.2337x - 15.826	0.5241x + 22.436	2.2513x + 4.4761	1.2191x + 17.687	0.2187x + 17.697
		R^2	0.4085	0.699	0.2633	0.5501	0.9212	0.3272
B	In the river	y = ax + b	0.6044x + 8.8087	2.6723x - 24.879	0.6405x + 22.457	3.0053x - 20.781	1.2856x + 16.41	0.2856x + 16.411
		R^2	0.4025	0.6621	0.267	0.7506	0.9645	0.5727
C	On Land	y = ax + b	0.6691x + 0.5626	2.108x - 14.128	0.6259x + 19.233	2.3694x - 2.6263	1.3315x + 14.751	0.3187x + 15.176
		R^2	0.5502	0.8024	0.3951	0.7133	0.9306	0.5674
D	In the river	y = ax + b	0.4186x + 20.066	2.3823x - 10.602	0.2714x + 33.613	2.1365x + 4.72	1.1653x + 17.037	0.1653x + 17.037
		R^2	0.1883	0.601	0.0536	0.4497	0.92	0.1878
	On Land	y = ax + b	0.3194x + 23.878	2.2421x - 9.873	0.1527x + 37.796	1.5376x + 25.018	1.0569x + 21.684	0.0569x + 21.684
		R^2	0.1442	0.5879	0.0258	0.2052	0.9159	0.0306
E	In the river	y = ax + b	1.173x - 13.938	3.3515x - 30.821	1.1991x + 12.138	1.9648x + 5.9999	1.2957x + 14.56	0.2957x + 14.56
		R^2	0.5396	0.7113	0.2971	0.6234	0.8846	0.2852

Point Number	Land or River	Formula	Cc ~ Wu	Cc ~ Wl	Cc ~ Wp	Cc ~ Ip	Cuu ~ Wu	Cuu ~ Wl
A	In the river	y = ax + b	0.0181x - 0.3084	0.0026x + 0.317	0.0301x - 0.317	-0.0011x + 0.5122	0.5898x + 11.29	0.7483x - 6.9962
		R^2	0.7902	0.0189	0.2939	0.0026	0.1069	0.1127
	On Land	y = ax + b	0.0144x - 0.1929	0.0067x + 0.016	0.0305x - 0.3615	0.0018x + 0.3912	0.2276x + 24.414	0.2968x + 19.877
		R^2	0.719	0.1229	0.4555	0.0063	0.0181	0.0412
B	In the river	y = ax + b	0.0149x - 0.1621	0.0042x + 0.3612	0.0225x + 0.0035	0.0026x + 0.5257	-0.5069x + 45.988	-0.5199x + 49.53
		R^2	0.4386	0.0319	0.1238	0.0053	0.2015	0.2384
C	On Land	y = ax + b	0.0152x - 0.185	0.0115x - 0.2292	0.0333x - 0.4139	0.0141x - 0.0201	-	-
		R^2	0.8043	0.5159	0.6473	0.3622	-	-
D	In the river	y = ax + b	0.0104x + 0.0417	0.0073x + 0.1295	0.0313x - 0.1998	0.0077x + 0.2847	-0.6396x + 60.417	0.2375x + 16.497
		R^2	0.7497	0.26	0.3742	0.1822	0.1177	0.0592
	On Land	y = ax + b	0.0173x - 0.2984	0.0085x + 0.0252	0.0178x + 0.0901	0.0085x + 0.2361	0.0712x + 19.778	0.1277x + 15.204
		R^2	0.5259	0.309	0.088	0.2676	0.0056	0.0763
E	In the river	y = ax + b	0.0106x + 0.0752	0.0206x - 0.4856	0.0459x - 0.4965	0.0314x - 0.3009	1.1082x + 87.529	12.047x - 459.47
		R^2	0.5188	0.4893	0.5359	0.3802	0.396	1

Table for Regression line and Coefficient of Determination R2 between Physical Porperties and Mechanical Properties with Point-wise for Thilawa clays (2/2)

Point Number	Land or River	Formula	Cuu ~ Wp	Cuu ~ Ip	Py ~ Wu	Py ~ Wl	Py ~ Wp	Py ~ Ip
A	In the river	y = ax + b	-3.2848x + 122.7	1.2086x - 4.5756	-6.2062x + 459.32	2.7937x + 20.297	-7.6114x + 391.38	4.8351x + 22.397
		R^2	0.1706	0.2448	0.2179	0.0528	0.0442	0.1195
	On Land	y = ax + b	-0.1247x + 40.876	0.5838x + 16.592	-5.4057x + 415.32	0.1162x + 161.43	-7.9595x + 384.81	2.2545x + 79.23
		R^2	0.0006	0.1053	0.4073	0.0001	0.1254	0.0384
B	In the river	y = ax + b	-1.1581x + 50.441	-0.8248x + 45.088	-0.8774x + 173.88	0.0126x + 127.77	0.0484x + 127.2	0.0136x + 128.07
		R^2	0.1628	0.2622	0.0474	0.000009	0.00002	0.000004
C	On Land	y = ax + b	-	-	-3.8743x + 319.17	-1.8983x + 268.75	-5.5269x + 299.37	-2.3426x + 234.09
		R^2	-	-	0.3853	0.1042	0.1307	0.0732
D	In the river	y = ax + b	0.3821x + 20.816	0.3253x + 19.351	-4.0081x + 343.2	-1.5583x + 243.4	-10.582x + 402.85	-1.1592x + 195.69
		R^2	0.0091	0.0733	0.3544	0.038	0.137	0.0132
	On Land	y = ax + b	0.5689x + 8.7621	0.1304x + 18.37	-7.253x + 506.28	-1.5893x + 252.75	-4.1299x + 260.07	-1.5333x + 211.13
		R^2	0.0724	0.0613	0.3078	0.0363	0.0157	0.0292
E	In the river	y = ax + b	99.522x - 1708.2	13.707x - 287.49	-3.2309x + 327.68	-4.8932x + 423.82	7.8317x + 350.18	-9.4999x + 440.68
		R^2	1	1	0.5149	0.2972	0.1674	0.374

Point Number	Land or River	Formula	Cc ~ Cuu	Cc ~ q_u	r_t ~ Wu	r_t ~ Wl	r_t ~ Wp	r_t ~ Ip
A	In the river	y = ax + b	-0.0079x + 0.7809	-0.0027x + 0.7061	-0.1076x + 22.075	-0.0205x + 18.754	-0.2172x + 23.199	0.0049x + 17.327
		R^2	0.7546	0.3382	0.8686	0.0348	0.4335	0.0015
	On Land	y = ax + b	-0.0056x + 0.6594	-0.0023x + 0.6538	-0.0946x + 21.703	-0.0493x + 20.738	-0.2057x + 22.969	-0.0181x + 18.175
		R^2	0.3774	0.3323	0.8469	0.1742	0.5561	0.015
B	In the river	y = ax + b	0.0005x + 0.6157	0.0006x + 0.5719	-0.11x + 22.41	-0.0612x + 20.281	-0.2173x + 22.562	-0.0717x + 18.962
		R^2	0.0006	0.0049	0.8897	0.248	0.43	0.1484
C	On Land	y = ax + b	-	-	-0.01x + 2.2531	-0.0072x + 2.2531	-0.0212x + 2.384	-0.0087x + 2.1144
		R^2	-	-	0.886	0.5454	0.7067	0.3736
D	In the river	y = ax + b	-0.0043x + 0.6605	-0.0016x + 0.6572	-0.054x + 19.481	-0.0564x + 20.206	-0.2206x + 22.281	0.013x + 16.873
		R^2	0.48	0.0913	0.3115	0.3944	0.6277	0.0171
	On Land	y = ax + b		-0.0011x + 0.6175	-0.0946x + 21.619	-0.0318x + 19.109	-0.144x + 20.728	-0.0289x + 18.211
		R^2		0.0277	0.8606	0.2297	0.2972	0.1493
E	In the river	y = ax + b		-0.0021x + 0.8	-0.0984x + 21.807	-0.1742x + 26.223	-0.3527x + 25.375	-0.29x + 25.469
		R^2		0.2043	0.899	0.621	0.6659	0.4863

APPENDIX “H”

CORRELATION BETWEEN SPECIFIC SURFACE AREA AND PHYSICAL PROPERTY VALUE OF THILAWA CLAY (AT POINT D)

Correlation between specific surface area and physical property value of Thilawa clay (at Point D)

1. Background

With clay having a large specific surface area, it is known that clay having a large liquid limit (w_L) and a small specific surface area has a relatively small inter-particle force as compared with clay having a large specific surface area, so that the liquid limit is small[1]. Conversely, particles with a large specific surface area have large inter-particle forces, excluding physicochemical properties such as ionic load and solution concentration of adsorbed salts, the size of the liquid limit depends on the size of the specific surface area.

Since the specific surface area of Thilawa clay was measured this time, the relation between the magnitude of the specific surface area and the liquid limit was compared with the value shown in the past study.

2. Result of Specific surface area measurement

The measurement result of the specific surface area of Thilawa clay is shown in Table 1. The value of specific surface area is 23.5 m^2/g. From this specific surface area, what is known is considered referring to the following Table 2 and Table 3[1].

Table 4 Specific surface area measurement result by N^2 gas adsorption method

Sample	Specific surface area (m^2/g)		Weight of sample (g)
	Multipoint Method	One point Method	
Clay at Point D In Thilawa	23.5	23.0	1.662

*BET plot value is adapted for multipoint method.
*Value at P/Po=0.2 is adapted for one point method.

Table 2 Liquid Limit of Clay (WARKETINN, BIRRELL, WHITE)[1]

Sample Type	Liqud LImit (%)
Kaolinite —Na	52
—Ca	73
Illite —Na	61
—Ca	90
Montmorillonite —Na	700
—Ca	177
Allophane	
— Raw Soil	231
— Air dry soil	85
Na— Montmorillonite	
— Water	950
—0.01 N NaCl	870
—1.0 N NaCl	350
Ca— Montmorillonite	
— Water	360
—1.0 N $CaCl_2$	310
Kaolinite pH 4	
— Water	54
—0.01 N $CaCl_2$	46
—1.0 N $CaCl_2$	39
Na— Kaolinite pH 10	
— Water	36
—0.01 N NaCl	34
—1.0 N NaCl	40

Table 3 Size of clay particle and Specific surface area (Yong Warkenth)[1]

Clay Mineral	Approximate Thickness (Å)	Maximum Specific Surface Area (m^2/g)	Volume change observed
Montmorillonite	20	800	Big
Illite	200	80	Normal
Kaolinite	1,000	15	Small

The specific surface area 23.5 m^2/g is not so large but is a value typified by illite type clay mineral. According to Table 3, the volume change is neither large nor small, and it is an evaluation that it is ordinary.

On the other hand, a correlation formula w_L (%) = 1.2 S.S. (m^2/g) + 13.9 has been indicated as shown in Figure 1[2].

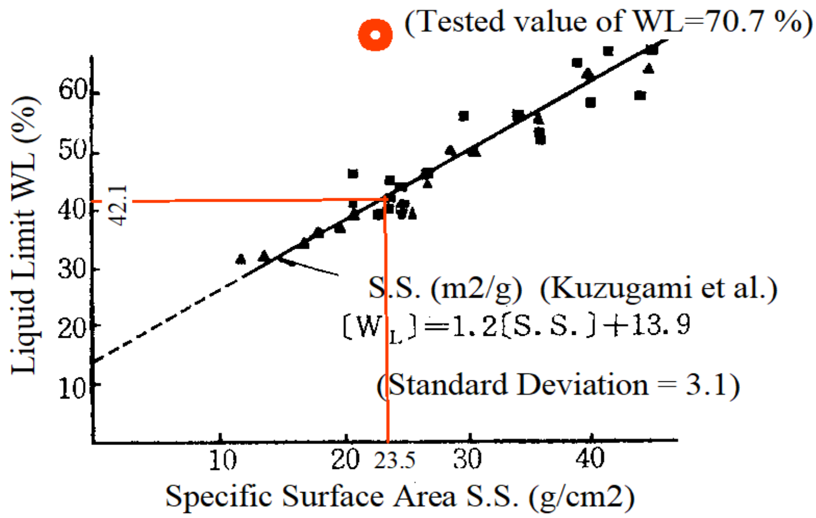


Figure 1 Relationship between Liquid Limit w_L and Specific Surface Area S.S.

According to this result, the test result of this time is S.S. = 23.5 to w_L = 1.2 x 23.5 + 13.9 = 42.1 (%). According to actual liquid limit test results, the liquid limit of this sample was obtained as 70.7%, and the prediction formula gives a small value as shown in Figure 1. From this liquid limitation, it is presumed that the clay is dominated by kaolinite, illite type minerals.

3. Future tasks

It is as described in the previous study[1)] that the liquid limit can be roughly determined by the size of the specific surface area. From this specific surface area, in order to evaluate whether soil structure, compression index and sensitivity ratio are high or low, further study/ research is necessary.

References

1) Maeda, T., Soma, K.: "Factors affecting the Consistency of the Soil" Phisical Property of Soil, Proceedings of the Society of Agricultural Civil Engineering, No. 30, November 1974.
2) Kuzugami, H., Ozaki, E., Nakaya, M.:”Relation between Specific Surface Area and Liquid Limit”, Proceedings of the Society of Agricultural Civil Engineering, No.37, p.61-67, 1971.

APPENDIX “I”

SALINITY CONCENTRATION OF THILAWA CLAY (AT POINT D)

Salinity concentration of Thilawa clay (at Point D)

1. Background

For the purpose of grasping the salt concentration contained in Thilawa clay, the chemical content test was carried out using OLYMPUS 's XRF Analyzer. The results are as follows.

2. Content of salinity concentration in Thilawa clay

As a method of calculating the amount of salt contained from the concentration of chlorine ions contained in Thilawa clay, the following equation is indicated by the Meteorological Agency, Japan "Ocean Observation Guidelines" (1999).

Salt S (‰) = 1.80655 x Chloride ion Cl^- (mg / l) x 10^{-3}

(The salinity of sea water is about 35‰ = 3.5 %)

Samples were taken from three depths (upper, middle and lower) at Point D in Thilawa. The test results are shown in Table 1.

Table 1 Result of Salinity Calculation

GL (m)	Cl (mg/l)	Salinity S (‰)	Remarks
-3.4	4,310	7.79	By XRF Analyzer
-9.4	4,857	8.77	ditto
-18.4	2,864	5.17	ditto
-9.4	8,000	14.45	By X ray Analysis

It is said that, in Japan clay, liquid limit is almost constant when the salt concentration is higher than 5 g /l [1)]. In this case, the salt concentration is about 5 per mil as minimum and they are ranging between 5 and 10 per mil as shown in Figure 1. It is said that the shear strength of clay or shear strength ratio is decreased and also anisotropy of strength becomes remarkable due to leaching of salt and it is larger when salinity concentration is smaller.

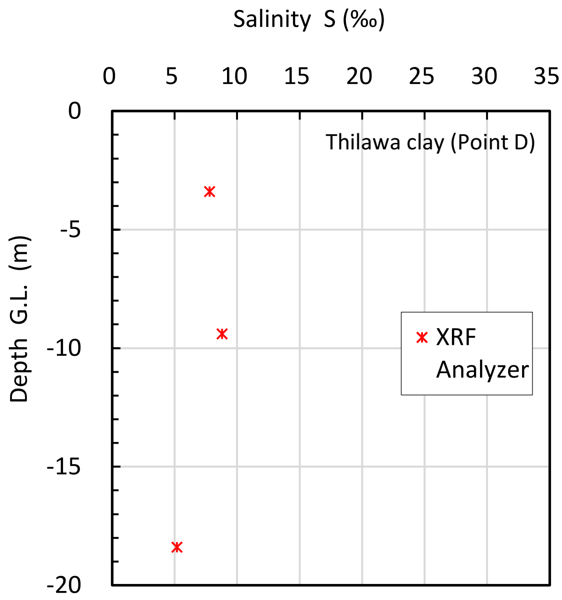


Figure 1 Salinity of Thilawa clay with Depth

3. Future tasks

It is necessary to investigate more on the salt concentration obtained this time compared with Japanese marine clay's one. It is also necessary to study whether leaching has already occurred or not in Thilawa clay.

The chemical analysis result data of Thilawa clay (Point D) are shown on the following pages.

References

1) Otsubo,M.: Small course "Marine Clay", Journal of the Society of Agricultural Civil Engineering, Vol. 67, No. 10, October 1999.

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Sydney NSW 2000
Australia

Client Name	**Sub-project for expansion of yangon port in thilawa floea package 1**
Locality	**Kyauk Tan township yangon region**
LAB No	**BH-5,T-1,Depth 3-3.80m**
Test Method	**XRF Analyzer(Geochem Mode)**
Receive Date	**04.04.2016**
Issue Date	**05.04.2016**
Test Fees	**10000 ks**

***Limit of Detections- Mg to U**

Sr.No	Elements	Amount	±	Unit
1	Mg	7508	2404.42	ppm
2	Al	1.53%	0.0461%	%
3	Si	5.01%	0.0287%	%
4	P	736	40.90	ppm
5	S	1005	20.38	ppm
6	Cl	4310	79.61	ppm
7	K	1.24%	0.0056%	%
8	Ca	2134	20.43	ppm
9	Ti	3079	52.01	ppm
10	V	202	18.95	ppm
11	Cr	114	8.79	ppm
12	Mn	179	8.53	ppm
13	Fe	6.55%	0.0250%	%
14	Co	ND		
15	Ni	166	4.71	ppm
16	Cu	13	3.22	ppm
17	Zn	137	2.55	ppm
18	As	ND		
19	Se	ND		
20	Rb	146	1.23	ppm
21	Sr	82	0.86	ppm
22	Y	30	0.81	ppm
23	Zr	118	1.10	ppm
24	Nb	11	0.85	ppm
25	Mo	ND		
26	Rh	ND		
27	Pd	ND		
28	Ag	ND		
29	Cd	ND		
30	Sn	ND		
31	Sb	ND		
32	Ta	58	3.53	ppm
33	W	ND		
34	Pt	ND		
35	Au	ND		
36	Hg	ND		
37	Pb	60	1.67	ppm
38	Bi	7	2.21	ppm
39	Th	8	2.27	ppm
40	U	ND		
41	LE	83.66%	0.2120%	%

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Address : 855, U Wisara Road, North Dagon
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Client Name	**Sub-project for expansion of yangon port in thilawa floea package 1**
Locality	**Kyauk Tan township yangon region**
LAB No	**BH-05,T-3,Depth 9-9.80**
Test Method	**XRF Analyzer(Geochem Mode)**
Receive Date	**04.04.2016**
Issue Date	**05.04.2016**
Test Fees	**10000 ks**

***Limit of Detections- Mg to U**

Sr.No	Elements	Amount	±	Unit
1	Mg	1.01%	0.2448%	%
2	Al	1.12%	0.0429%	%
3	Si	4.46%	0.0262%	%
4	P	611	39.90	ppm
5	S	6889	42.16	ppm
6	Cl	4857	79.55	ppm
7	K	1.21%	0.0054%	%
8	Ca	3313	23.69	ppm
9	Ti	3124	53.02	ppm
10	V	167	19.19	ppm
11	Cr	117	8.89	ppm
12	Mn	520	11.07	ppm
13	Fe	4.79%	0.0185%	%
14	Co	ND		
15	Ni	130	4.04	ppm
16	Cu	15	2.87	ppm
17	Zn	110	2.17	ppm
18	As	21	1.10	ppm
19	Se	ND		
20	Rb	124	1.07	ppm
21	Sr	81	0.81	ppm
22	Y	28	0.73	ppm
23	Zr	139	1.11	ppm
24	Nb	ND		
25	Mo	ND		
26	Rh	ND		
27	Pd	ND		
28	Ag	ND		
29	Cd	ND		
30	Sn	ND		
31	Sb	ND		
32	Ta	48	3.08	ppm
33	W	ND		
34	Pt	ND		
35	Au	ND		
36	Hg	ND		
37	Pb	45	1.39	ppm
38	Bi	9	2.05	ppm
39	Th	6	2.11	ppm
40	U	ND		
41	LE	85.38%	0.2186%	%

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Address : 855, U Wisara Road, North Dagon
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Client Name	**Sub-project for expansion of yangon port in thilawa floea package 1**
Locality	**Kyauk Tan township yangon region**
LAB No	**BH-05,T-6,Depth 18-18.80**
Test Method	**XRF Analyzer(Geochem Mode)**
Receive Date	**04.04.2016**
Issue Date	**05.04.2016**
Test Fees	**10000 ks**

***Limit of Detections- Mg to U**

Sr.No	Elements	Amount	±	Unit
1	Mg	8205	2468.71	ppm
2	Al	1.46%	0.0462%	%
3	Si	5.05%	0.0289%	%
4	P	448	40.08	ppm
5	S	7316	44.70	ppm
6	Cl	2864	81.56	ppm
7	K	1.27%	0.0057%	%
8	Ca	2681	22.55	ppm
9	Ti	3309	55.23	ppm
10	V	219	20.25	ppm
11	Cr	124	9.28	ppm
12	Mn	594	11.85	ppm
13	Fe	4.73%	0.0185%	%
14	Co	ND		
15	Ni	132	4.13	ppm
16	Cu	15	2.95	ppm
17	Zn	117	2.27	ppm
18	As	23	1.21	ppm
19	Se	ND		
20	Rb	140	1.15	ppm
21	Sr	73	0.78	ppm
22	Y	32	0.78	ppm
23	Zr	131	1.10	ppm
24	Nb	14	0.82	ppm
25	Mo	ND		
26	Rh	ND		
27	Pd	ND		
28	Ag	ND		
29	Cd	ND		
30	Sn	ND		
31	Sb	40	9.65	ppm
32	Ta	51	3.18	ppm
33	W	ND		
34	Pt	ND		
35	Au	ND		
36	Hg	ND		
37	Pb	61	1.56	ppm
38	Bi	ND		
39	Th	12	2.17	ppm
40	U	ND		
41	LE	84.83%	0.2195%	%

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APPENDIX “J”

COMPOSITIONAL ANALYSIS RESULTS OF THILAWA CLAY (POINT D) BY FLUORESCENT X-RAY ANALYSIS

Compositional analysis results of Tillawa clay (point D) by fluorescent X-ray analysis

1. Introduction

The X - ray diffraction pattern of clay collected at Thilawa (Point D) was measured. In addition, fluorescent X-ray analysis was also conducted in order to make reference to the crystalline component identification based on the X-ray diffraction pattern.

2. Sample and method

The sample is clay taken at Thilawa Area (Point D) in Myanmar.

XRD-7000 manufactured by Shimazu Corporation was used as the X-ray diffractometer, and EDX-7000 manufactured by Shimazu Corporation was used as the X-ray fluorescence analyzer. In the X-ray diffraction analysis, in addition to the indefinite azimuth sample, a fixed direction sample was prepared and ethylene glycol treatment was also performed. The measurement conditions are shown in Table 1 and Table 2.

Table 1 Measurement conditions of X-ray diffraction analysis

	Indefinite azimuth sample	Oriented sample
X-ray tube	Cu	Cu
Tube voltage	40kV	40kV
Tube current	30mA	30mA
Slit type	1°- 0.3mm -1°	1°- 0.3mm -1°
Scan speed	0.6°/min	1.2°/min
Scanning range	3°~ 70°	3°~ 35°

Table 2 Measurement conditions of X-ray fluorescence analysis

Target element: $_{11}Na \sim _{92}U$)

	Heavy Element	Light Element
X-ray tube	Rh	Rh
Tube voltage	50kV	15kV
Tube current	(Auto) μA	(Auto) μA
Filter	Nil, #1, 3, 4	Nil, #2

After air-drying the sample, it was pulverized in an agate mortar to prepare an indefinite azimuth powder sample, which was filled in a glass sample holder and the X-ray diffraction pattern was measured.

In addition, a part of the sample in an indeterminate orientation powder was dried at 105 ° C., packed in salt burning, pressure-molded, subjected to fluorescent X-ray analysis (target element: $_{11}$Na to $_{92}$U), and analysis of the obtained fluorescent X- After determination of the existing elements, estimated quantitative (semi-quantitative) calculation by oxide method equivalent FP method (fundamental parameter method) was carried out. In the FP method, only the detected element is normalized to 100%.

Furthermore, clay content was collected from irregular azimuth powder sample by elutriation method to prepare a fixed orientation sample, and X-ray diffraction pattern was measured. After measurement, in order to confirm the smectite, ethylene glycol was sprayed on the oriented sample and the X-ray diffraction pattern was measured again.

After confirming the existence of smectite by measuring the oriented sample treated with ethylene glycol, the diffraction pattern of the indefinite azimuth sample was searched and the crystal component was identified. For the search, elemental information by fluorescent X-ray analysis was used.

3. Results of Measurement

X-ray diffraction search results are shown in Table 3 and estimated quantitative analysis results by X-ray fluorescence analysis are shown in Table 4.

In addition to report the following data;

① X-ray diffraction pattern, peak list and search results

② X-ray diffraction patterns and peak lists of the oriented sample and the ethylene glycol treated sample

③ Overlapping pattern of fixed direction sample and ethylene glycol treated sample

④ Fluorescent X-ray spectrum, peak list and estimated quantitative (semi-quantitative) analysis result of sample in indefinite azimuth

Table 3 X-ray diffraction search result

Sample	Identified Ingredients	Less abundance or less reliable part
Thilawa Clay at Point D	SiO_2 ［石英/Quartz］ $(K,Na)(Al,Mg,Fe)_2(Si_{3.1}Al_{0.9})O_{10}(OH)_2$ ［白雲母/Muscovite］ $(Mg,Fe,Al)_6(Si,Al)_4O_{10}(OH)_8$ ［クリノクロア/Clinochlore］	$Al_2Si_2O_5(OH)_4$ ［カオリナイト/Kaolinite］ $(Ca,Na)(Si,Al)_4O_8$ ［灰長石/Anorthite］ $Na_{0.3}(Al,Mg)_2Si_4O_{10}(OH)_2 \cdot xH_2O$ ［モンモリロナイト/Montmorillonite］ FeS_2 ［黄鉄鉱/Pyrite］ $(Ca,Na)_{2.26}(Mg,Fe,Al)_{5.15}(Si,Al)_8O_{22}(OH)_2$ ［苦土普通角閃石/Magnesiohornblende］

*There are weak diffraction lines which cannot be analyzed other than the above.

Table 4 Fluorescent X-ray analysis estimated quantitative analysis result

Compound Name	Thilawa Clay at Point D
Na_2O	1.2
MgO	2.3
Al_2O_3	21.7
SiO_2	60.7
SO_3	3.0
Cl	0.80
K_2O	2.8
CaO	0.39
TiO_2	0.76
Cr_2O_3	0.034
MnO	0.076
Fe_2O_3	6.2
NiO	0.014
CuO	0.014
ZnO	0.012
Ga_2O_3	0.003
As_2O_3	0.004
Br	0.003
Rb_2O	0.017
SrO	0.010
Y_2O_3	0.006
ZrO_2	0.021
NbO	0.002
BaO	0.017
PbO	0.003

*The peak of Rh on the qualitative chart is generated from the X-ray tube as the excitation source.

The composition analysis results by fluorescent X-ray analysis of Thilawa clay (Point D) on the following pages are shown.

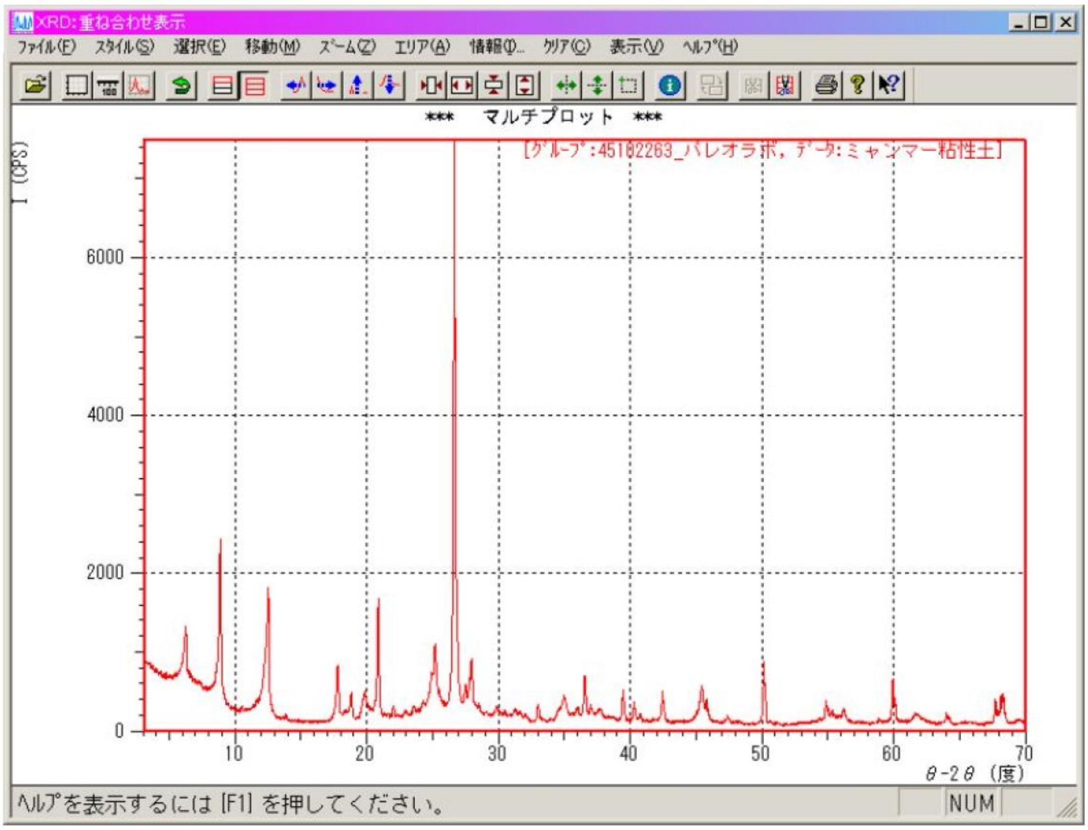


Figure 1 XRD Pattern Chart for Indefinite Azimuth Sample

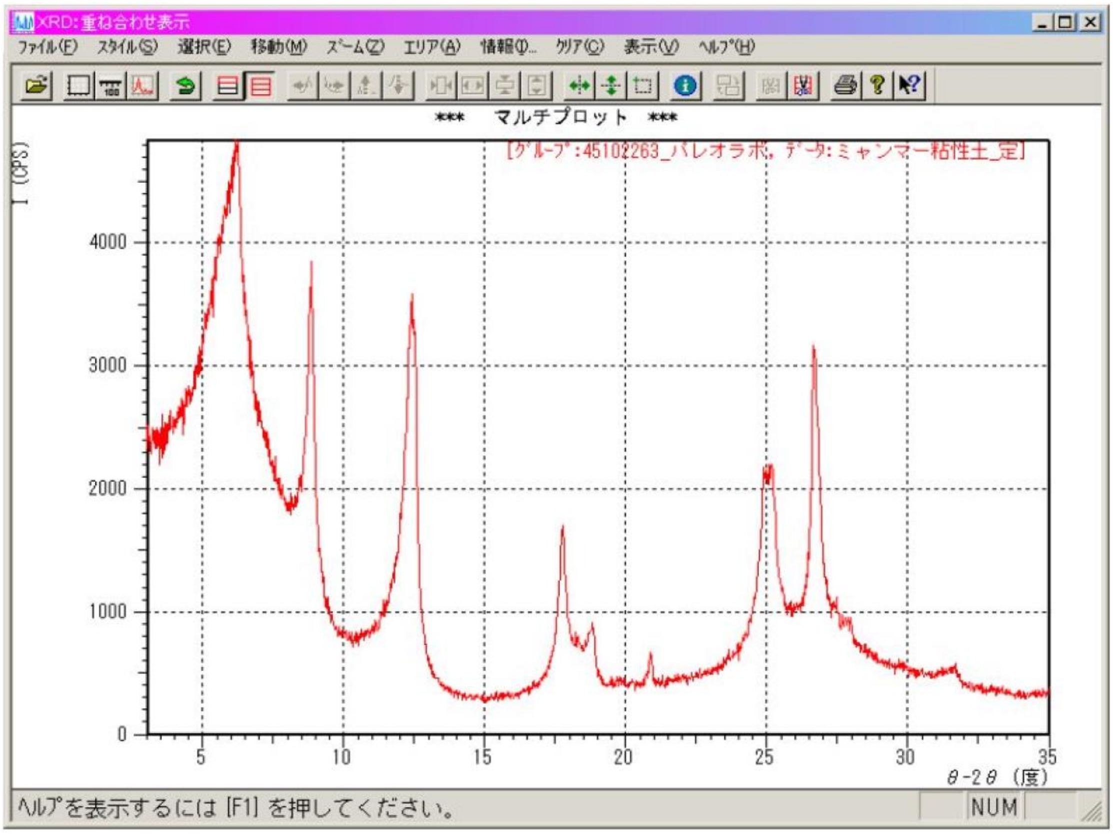


Figure 2 XRD Pattern Chart for Oriented Sample

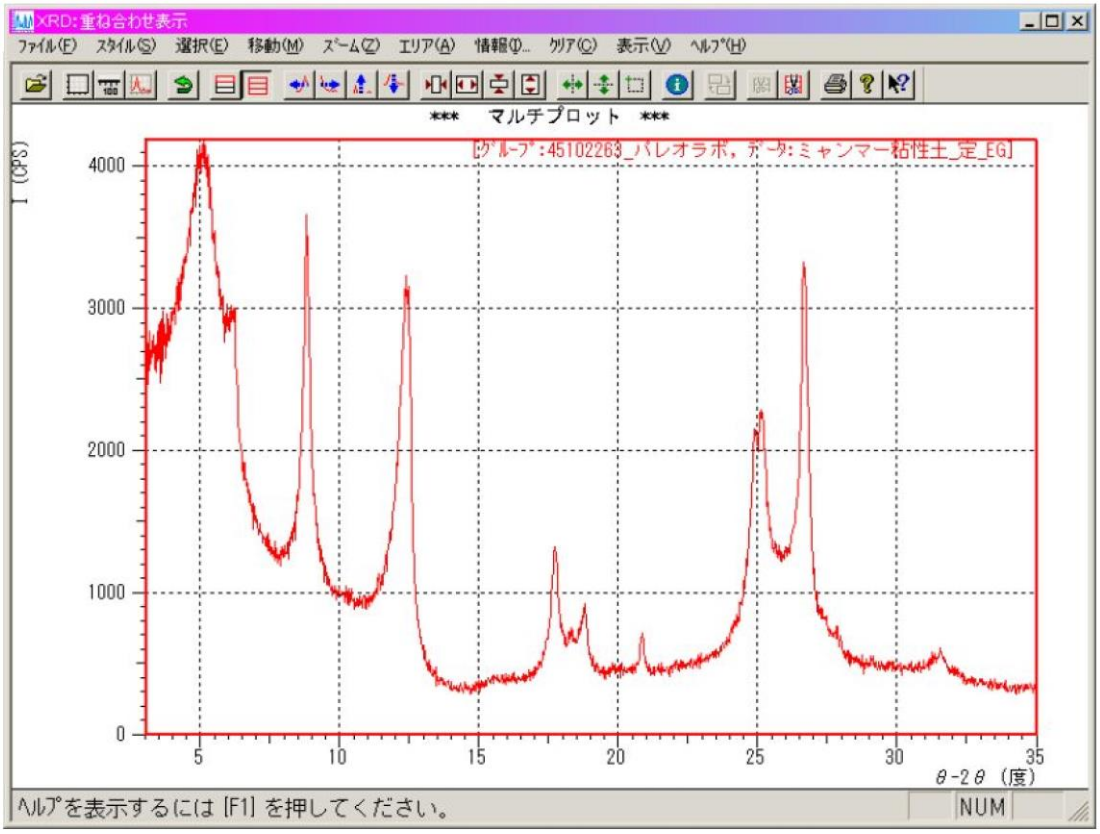


Figure 3 XRD Pattern Chart for EG Treated Sample

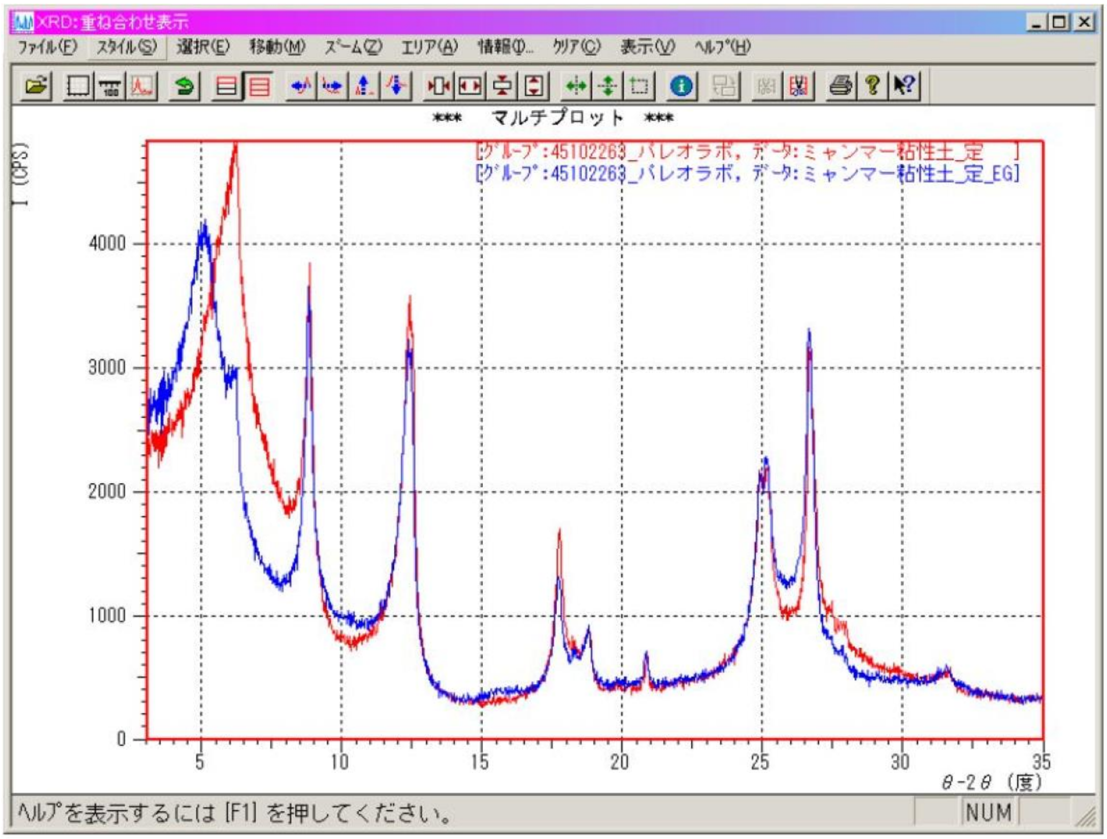


Figure 4 XRD Pattern Chart for Two Curves Over Written

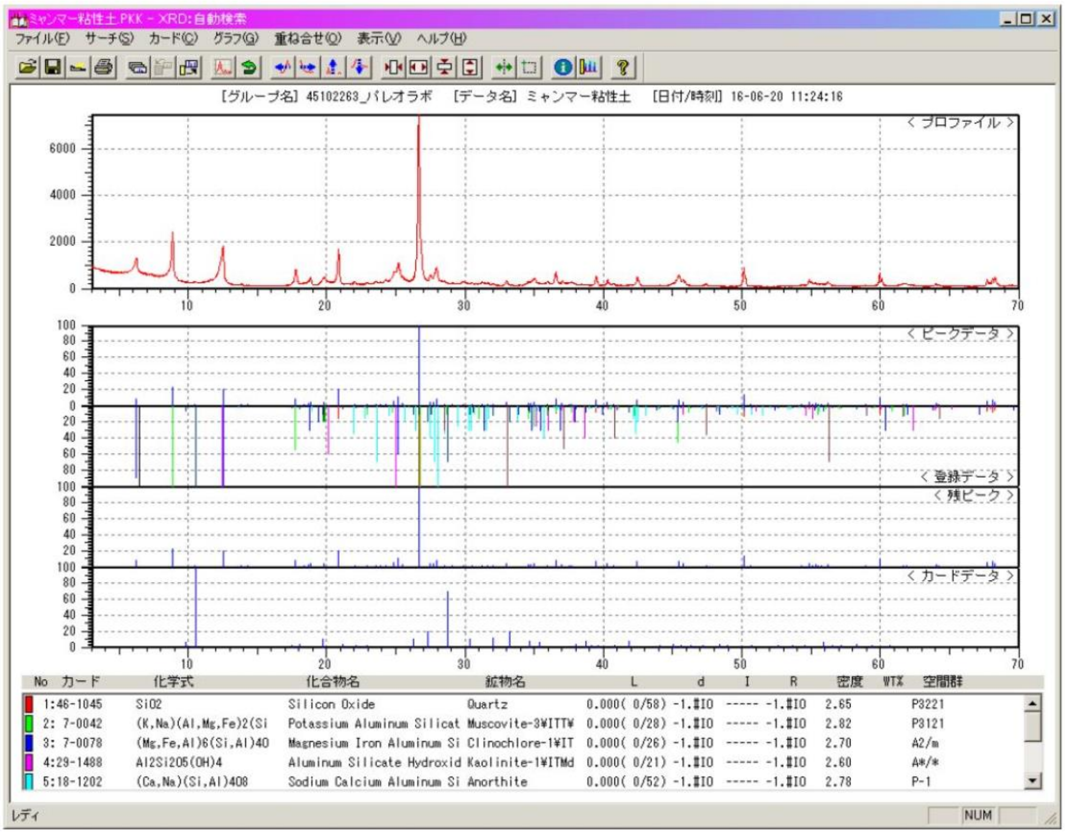


Figure 5 Qualitative Analysis Result 1

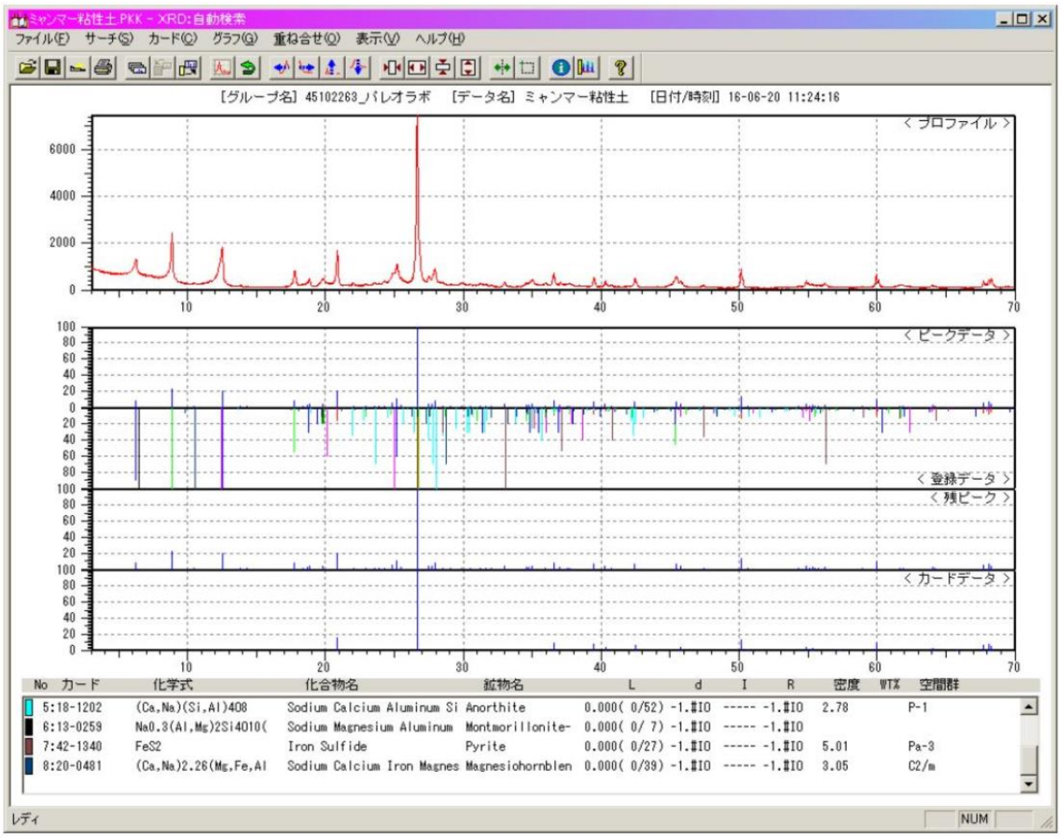
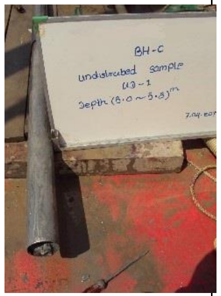


Figure 6 Qualitative Analysis Result 2

APPENDIX “K”

PHOTOS OF THILAWA CLAYS

Thilawa

Layer Names	Point A		Point B		Point C	Point D		Point E
	Land	River	Land	River	Land	Land	River	River
CLAY-I		-		-		-	-	-
CLAY-II								
CLAY-III								

APPENDIX “L”

CORRELATION GRAPHS AND DETERMINATION FACTORS BETWEEN PHYSICAL AND MECHANICAL PROPERTIES OF JAPAN CLAYS (PORT AND HARBOR AREA)

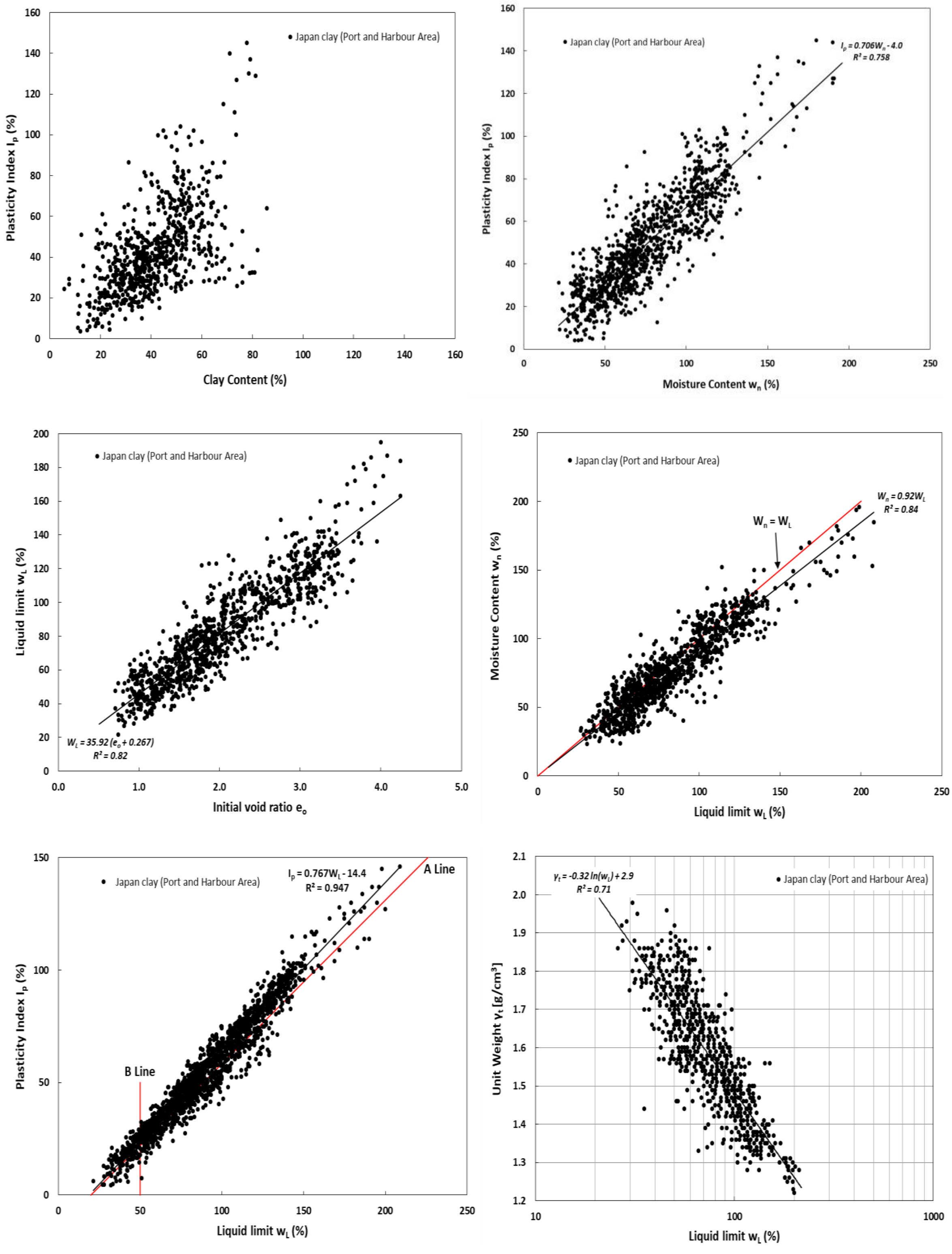


Figure 1 Correlation Graphs and Determination Factors between Physical and Mechanical Properties of Japan Clays (Port and Harbor Area)

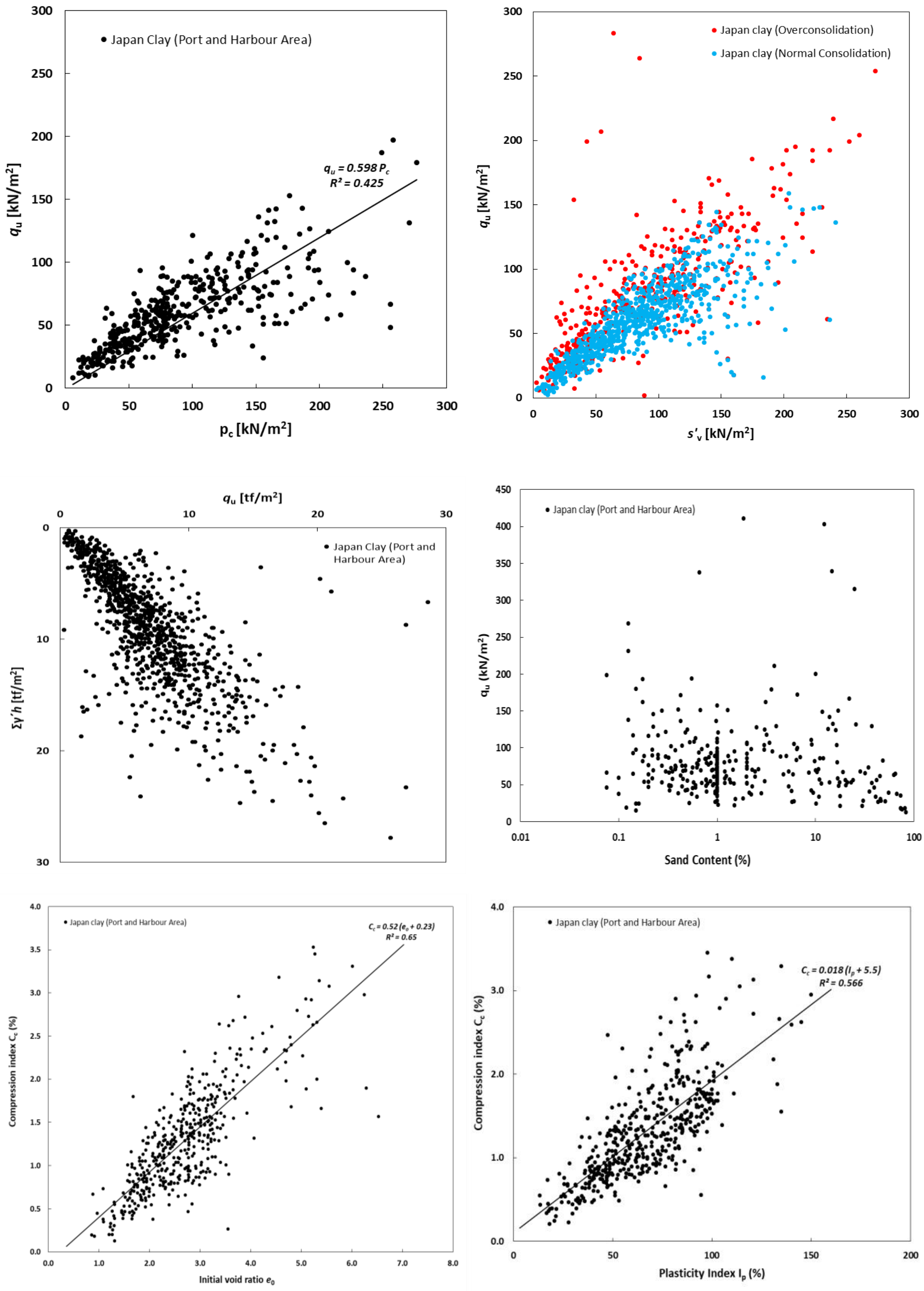


Figure 1 Correlation Graphs and Determination Factors between Physical and Mechanical Properties of Japan Clays (Port and Harbor Area)

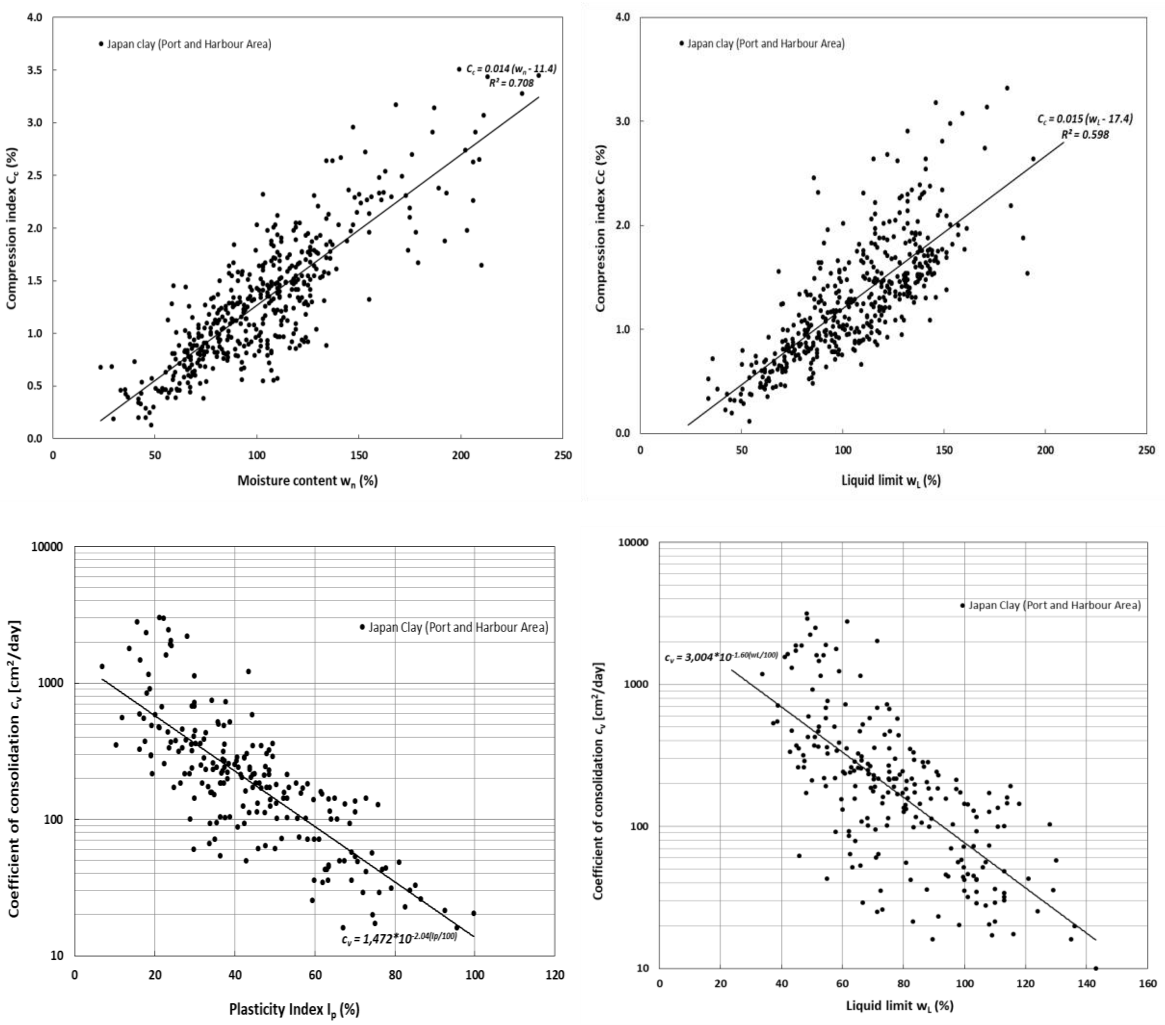


Figure 1 Correlation Graphs and Determination Factors between Physical and Mechanical Properties of Japan Clays (Port and Harbor Area)

APPENDIX “M”

DISTRIBUTION GRAPHS WITH DEPTH FOR PHYSICAL AND MECHANICAL PROPERTIES OF HOCHIMINH CLAYS

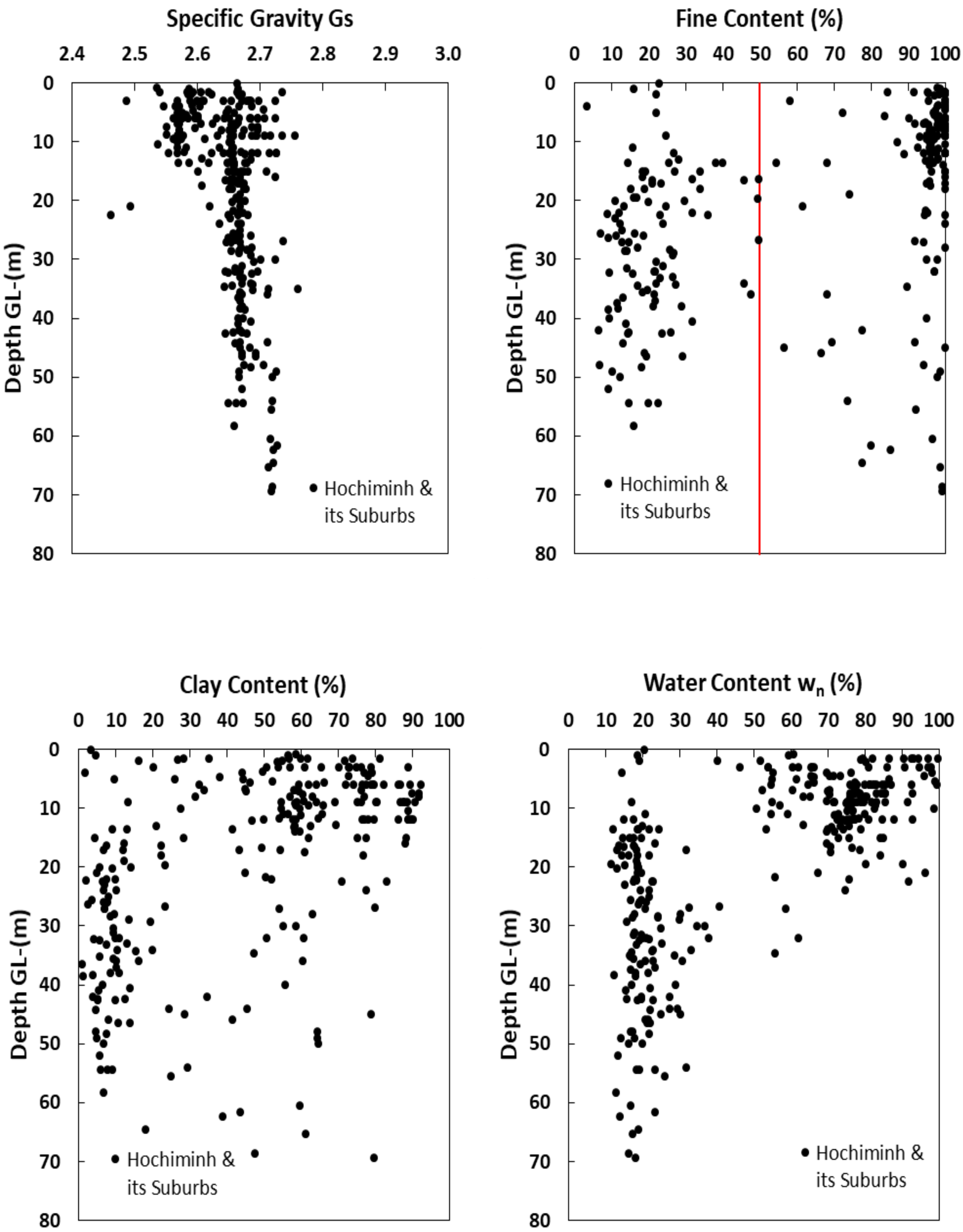


Figure 1 Physical Properties of Hochiminh and its suburbs areas Graphs

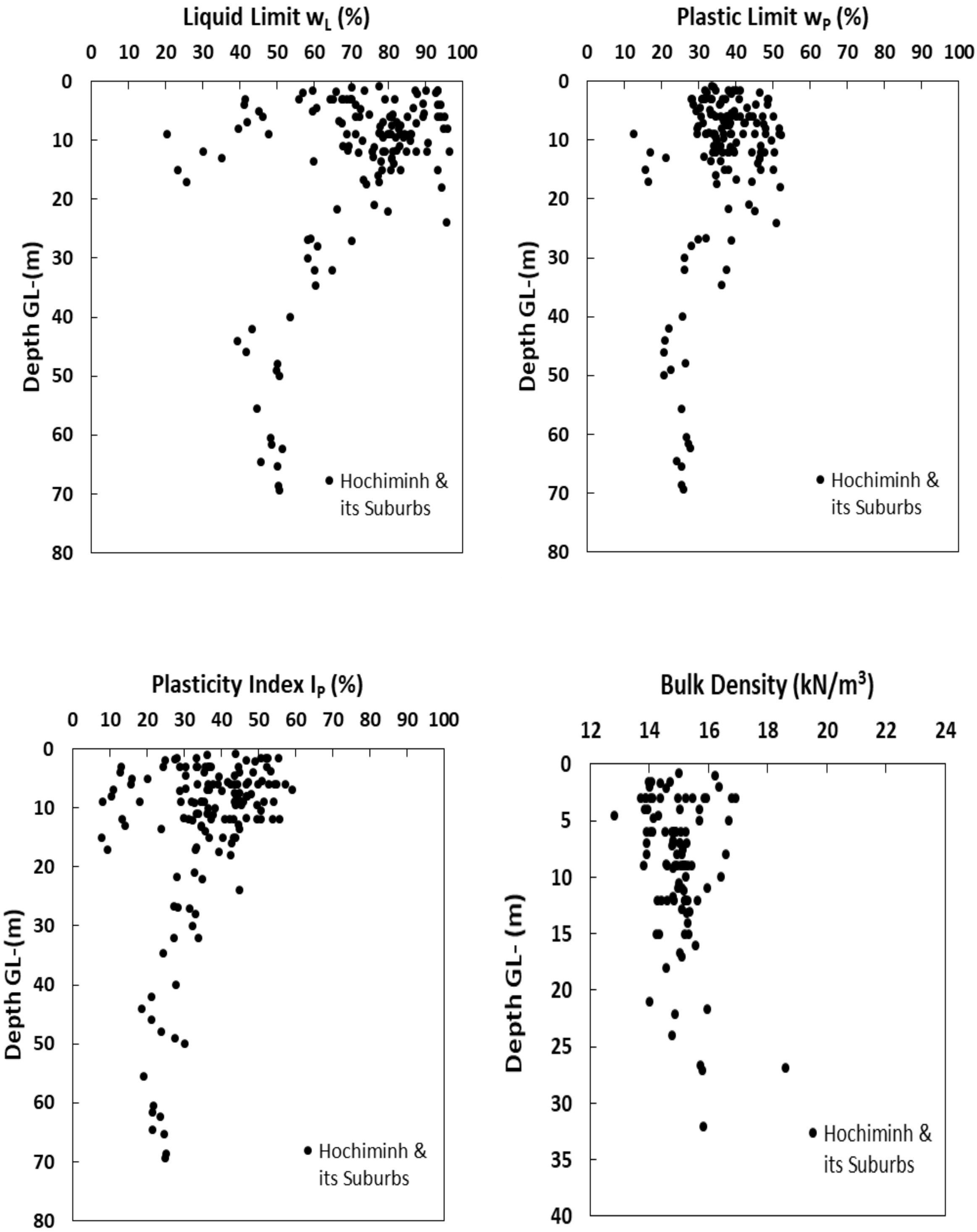


Figure 1 Physical Properties of Hochiminh and its suburbs areas Graphs

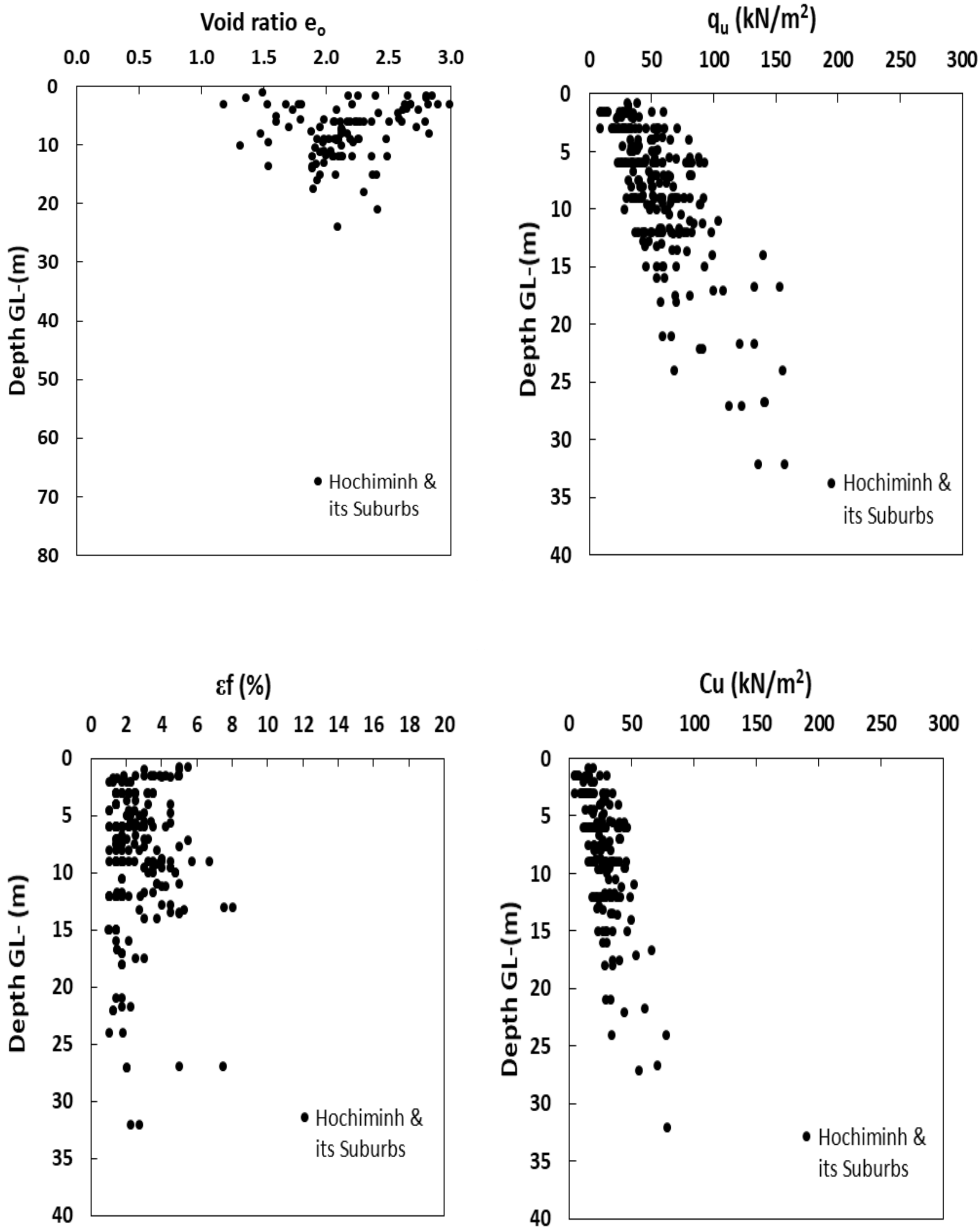


Figure 2 Mechanical Properties of Hochiminh and its suburbs areas Graphs

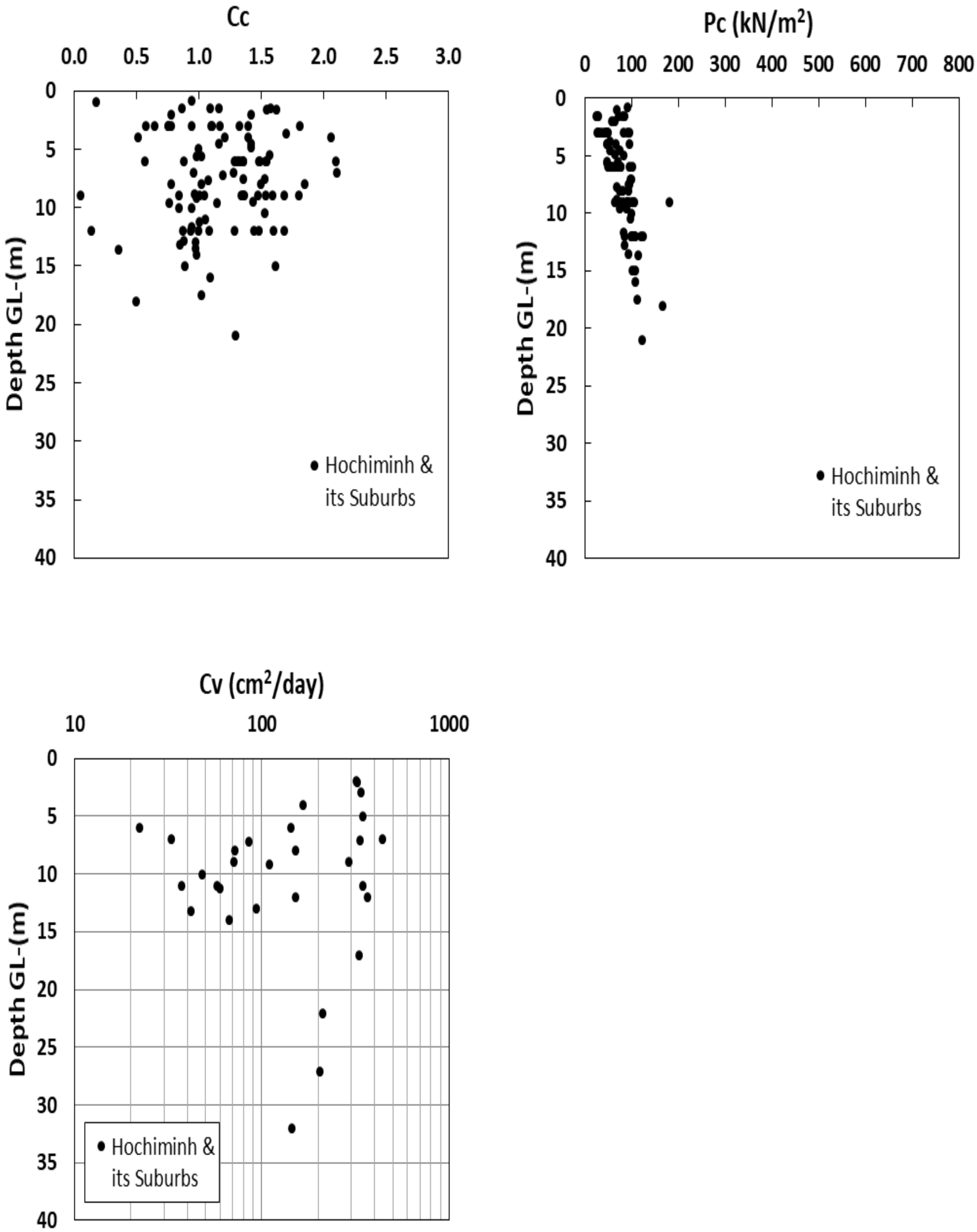
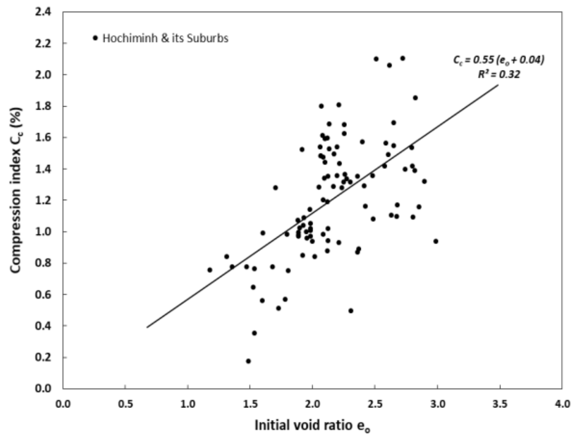
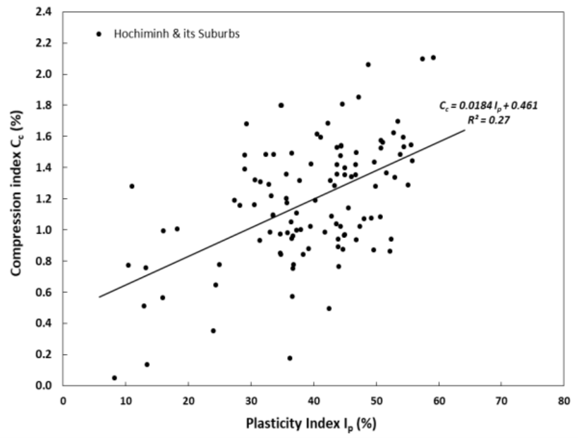


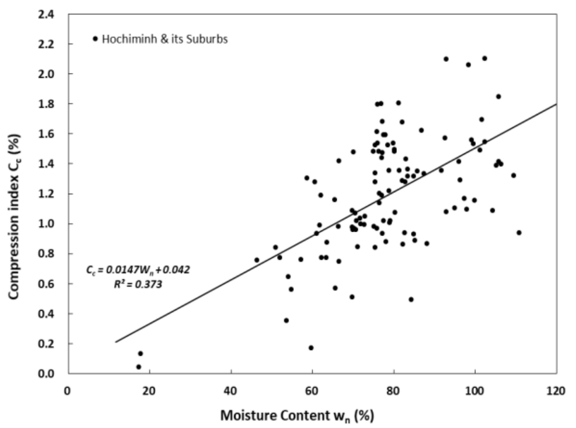
Figure 2 Mechanical Properties of Hochiminh and its suburbs areas Graphs

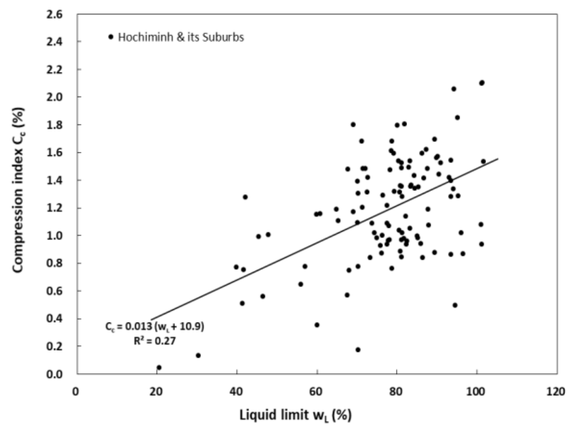
APPENDIX "N"

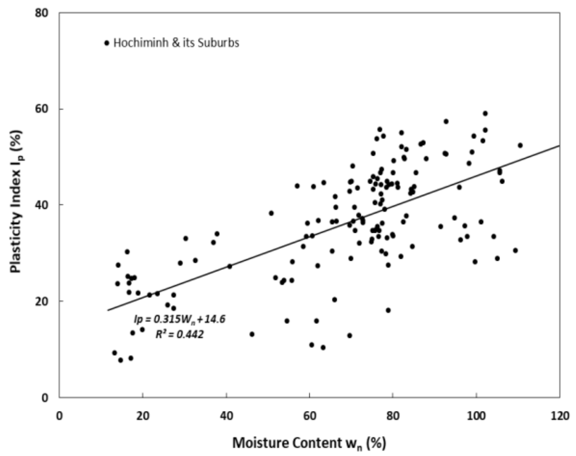
CORRELATION GRAPHS AND DETERMINATION FACTORS BETWEEN PHYSICAL AND MECHANICAL PROPERTIES OF HOCHIMINH CLAYS

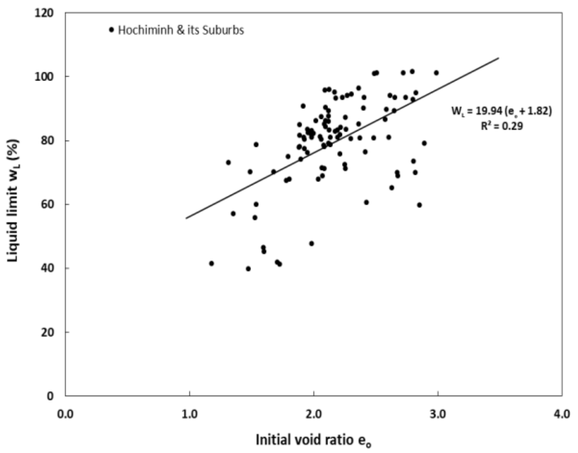

Hochiminh & its Suburbs
$C_c = 0.55\,(e_o + 0.04)$
$R^2 = 0.32$
Compression index C_c (%)
Initial void ratio e_o

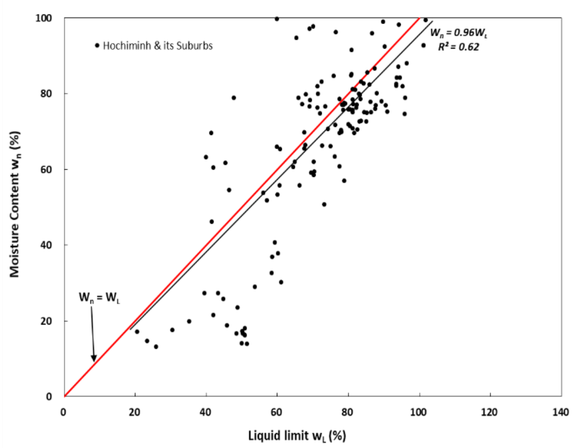

Hochiminh & its Suburbs
$C_c = 0.0184\,I_p + 0.461$
$R^2 = 0.27$
Compression index C_c (%)
Plasticity Index I_p (%)

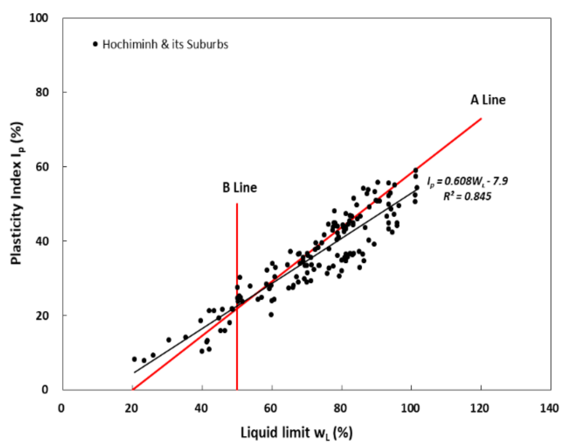

Hochiminh & its Suburbs
$C_c = 0.0147W_n + 0.042$
$R^2 = 0.373$
Compression index C_c (%)
Moisture Content w_n (%)


Hochiminh & its Suburbs
$C_c = 0.013\,(w_L + 10.9)$
$R^2 = 0.27$
Compression index C_c (%)
Liquid limit w_L (%)


Hochiminh & its Suburbs
$Ip = 0.315W_n + 14.6$
$R^2 = 0.442$
Plasticity Index I_p (%)
Moisture Content w_n (%)


Hochiminh & its Suburbs
$W_L = 19.94\,(e_o + 1.82)$
$R^2 = 0.29$
Liquid limit w_L (%)
Initial void ratio e_o


Hochiminh & its Suburbs
$W_n = 0.96W_L$
$R^2 = 0.62$
$W_n = W_L$
Moisture Content w_n (%)
Liquid limit w_L (%)


Hochiminh & its Suburbs
A Line
B Line
$I_p = 0.608W_L - 7.9$
$R^2 = 0.845$
Plasticity Index I_p (%)
Liquid limit w_L (%)

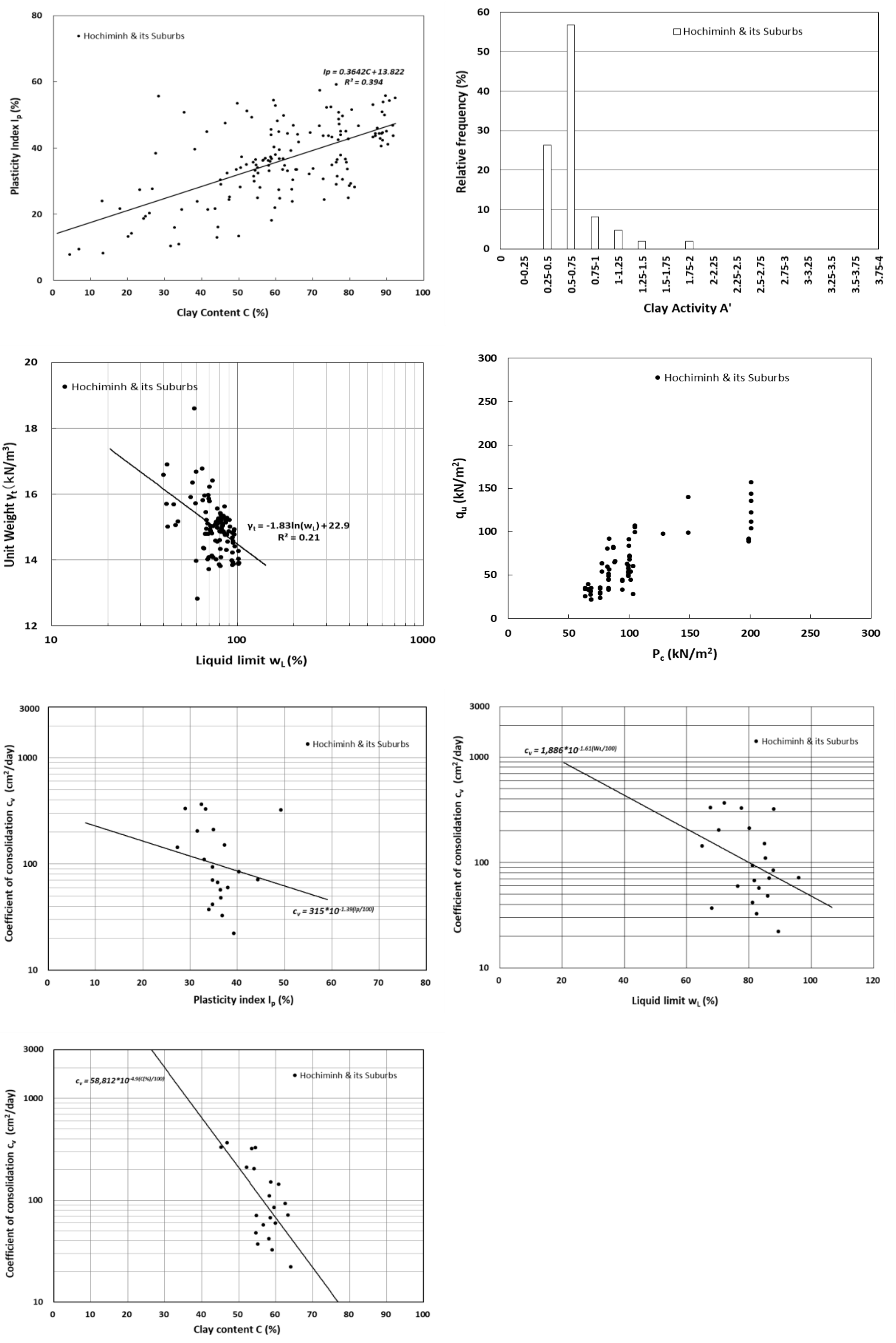


Figure 1: Correlation Graphs and Determination Factors between Physical and Mechanical Properties of Hochiminh Clays