한국지반환경공학회 논문집 제7권 제2호 2006년 4월 pp. 55~66

Case Study Of Reducing Specimen Disturbance Using Vertical Fixing Sample Frame (VFSF)

연직고정장치(VFSF)를 활용한 불교란시료의 교란효과 저감사례

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Abstract

The existing Highway LA-1 of US is required to be replaced for covering increasing regional demands of transportation such as Hurricane evacuation and oil industry. This 28 km crosses wetlands and is a sensitive environmental area. Huge amount of soil investigation and laboratory tests were performed with best efforts to overcome inherit errors of sampling, disturbance, and testing procedures for this project. The data scattering was corrected through using central tendency theory.

Keywords : Soil investigation, Laboratory tests, Central tendency theory

요 지

미국의 기존 고속도로인 LA-1 은 허리케인 대피 및 석유산업 등과 같은 수송수단에 대한 증가하는 지역적 요구를 해소하기 위해 교체되어야 할 필요가 있다. LA-1 고속도로 28km 구간은 습지를 가로지르며 예민한 지반으로 이루어져 있다. 본 지역에 대한 시료채취, 시료의 교란 그리고 시험 과정에서 있을 수 있는 고유의 오류들을 줄이기 위해 방대한 양의 지반조사 및 정밀한 실내시험을 수행하였으며 VFSF를 이용한 시료 운반은 지반에 대한 보다 정밀한 데이터를 얻을 수 있다.

주요어 : 지반조사, 실내시험, VFSF

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1. INTRODUCTION

The existing LA-1 is a primary transportation route for foreign oil offloaded from ships in the Gulf of Mexico and also serves as a hurricane evacuation route for the local population of 35,000 plus 6,000 offshore workers (Figure 1). The existing road floods with severe weather. The Louisiana Department of Transportation and Development (LADOTD) desires to replace old LA-1 by constructing a four-lane, fully access controlled, elevated highway on a new location between Golden Meadow and Fourchon, Louisiana. This 17 mile corridor crosses wetlands, bayous, channels, and flood plains and is a sensitive environmental area. It is to consist of low-level and medium-level bridges, two elevated interchanges, and two fixed high-level bridges over navigable waterways: one over Bayou LaFourch and the Boudreaux Canal at Leeville and one over Bollinger Canal. A toll plaza facility and scenic overlook or bird watching area may be included in the final design concept. Phased construction will be used to allow the project to be constructed in segments as funding permits. This paper encompasses only Phase IA beginning from the intersection of existing LA-1 and LA-3090 to the elevated portion of the southern connector bridge near Station 370+00. This paper will address data interpretation, design, and construction related issues.

1.1 Paper Organization

The second section of this paper describes soil parameter interpretation which is a very important step for pile capacity and settlement predictions.

2. PARAMETER EVALUATIONS

Sampling and testing produce inherit errors resulting from stress relaxation which are reflected in the bias of data. Inevitable reading, handling, and testing errors may be reflected in the scatter of test results. These errors may be detected by comparisons to published correlations. Bias may be then adjusted based on geotechnical engineering and data scatter may be corrected using central tendency theory.

For avoiding sample disturbance, all sample tubes were transported within vertical fixing frame during 3 hours driving from site to laboratory as shown in Figure 2. Vane shear tests were performed on the sample before it was excluded from the tubes to reduce error from disturbance. The effects of these efforts were proved from soil parameter evaluation in Section 2.2.

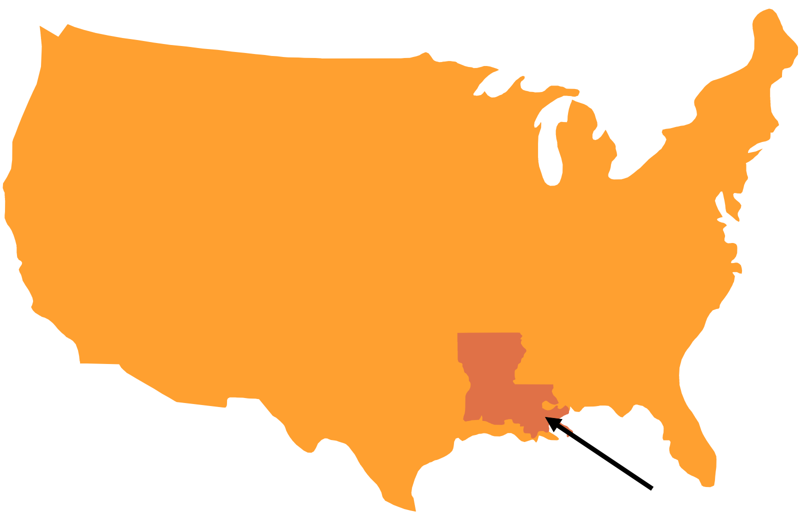

Figure 1(a). Location of New LA-1 Highway

Figure 1(b). Detail Location of New LA-1 Highway

Figure 2. Vertical fixing sample transport frame (VFSF).

2.1 Shear Strength

Soil shear strengths are necessary for calculating the capacities of the piles which support the structures. The shear strength of a cohesive soil depends heavily on its stress history. The relationship of strength and past effective overburden stress is normally expressed by its normalized shear strength parameter (Su/P) where:

Su = Soil shear strength
P = Effective overburden pressure

For the normally consolidated soils within this region, the values of the normalized shear strength parameter (Su/P) range from 0.2 to 0.28, averaging about 0.25. In the case of underconsolidated soils (those which are not yet fully consolidated under the existing overburden pressure), the shear strengths are lower than would be predicted with their (Su/P) ratios. Some underconsolidated soils were noted, the expected normalized shear strength value (Su/P) is smaller than the values indicated above.

The plot in Figure 3 shows the overconsolidation ratios (OCR=maximum past overburden/current overburden) obtained from the consolidation tests from this project versus depth. A typical soil profile is shown next to the plot for easy comparison. As can be seen, the majority of the OCR values are just above 0.5 with elevated OCRs between the depths of 30 and 50 feet and increasing scatter below the 80 foot depth. It is interesting to note the material type's impact on the OCR values. A sand layer approximately 10 feet or more is generally found at the depth of 40 feet throughout the majority of the site. This provides a drainage path for consolidation, so that consolidation near the sand zone proceeds more rapidly. This phenomenon results in the elevated OCR values around the 40 foot depth. Similarly, frequent thin sand and silt layers between the depths of 80 to 110, feet are also reflected in the OCR profile, showing a wide range of distribution. Slightly more consistent

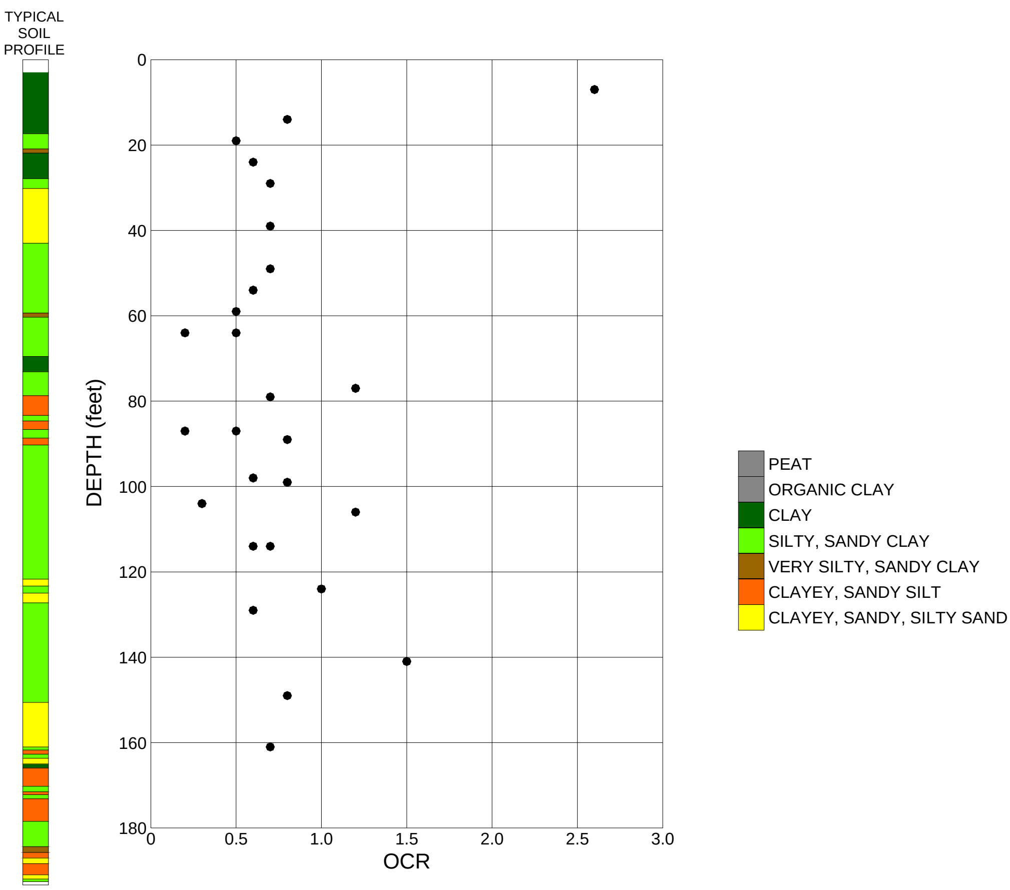

Figure 3. Overconsolidation Ratio as a Function of Depth

but still scattered OCR values were observed below the 110 foot depth for the less frequent silt and sand layers in that area.

Three types of shear strength testing were conducted for this project: laboratory unconsolidated-undrained triaxial tests, miniature vane shear, and in-situ cone penetration tests. The laboratory unconsolidated-undrained triaxial tests were conducted to the 20 percent strain when failed in a yield mode. The sample's in-situ total stress level was used as the confining stress. The miniature vane shear tests were performed on the cohesive samples prior to their extrusion to minimize the effect of stress relaxation. Two sizes of vane were used depending upon the consistency of the soils: 33 mm for very soft materials and 19 mm for the stronger materials. Cone penetrometer soundings were performed to depths ranging from slightly below 100 feet to just over 200 feet. According to McClelland (1956) the soils at this site are under consolidated and have a normalized shear strength parameter (Su/P) of about 0.15. This value is confirmed using Ladd's (1976) *Stress History and Normalized Soil Engineering Properties* (SHANSEP) concept. Figures 4 and 5 presents a scatter plot of shear strengths versus depth near stations 113+00 and 325+00 using all three methods. The vane shear and unconsolidated undrained triaxial tests were plotted as the laboratory tests. The results indicate that both the laboratory unconsolidated undrained traixial tests and the CPT soundings point to a (Su/P) ratio of between 0.15 and 0.17 for clays. A cone factor (Nk) of 15 was used for this evaluation. Laboratory tests showed a much higher scatter. However, the average values are still within this range.

As shown in Figures 4 and 5, soil classification with depth (3rd column from left black=sand, red=silt, green=clay) is a new way to describe soil type estimated from CPT measurements using probability calculation. This method provides co-existing various soil types even in dominant certain soil type distributed of some depth. For example, some sand layer can not be changed interrupt to clay layer. Mother of nature has soil layers which consist of continuous changing characteristics instead of sudden disconnected changing. The previous soil classification method using CPT provided only disconnected classification even if they already recognized of natural changing trend of soil layers. However, probable classified method may show the continuous soil layers using CPT as seen in Figures 4 and 6.

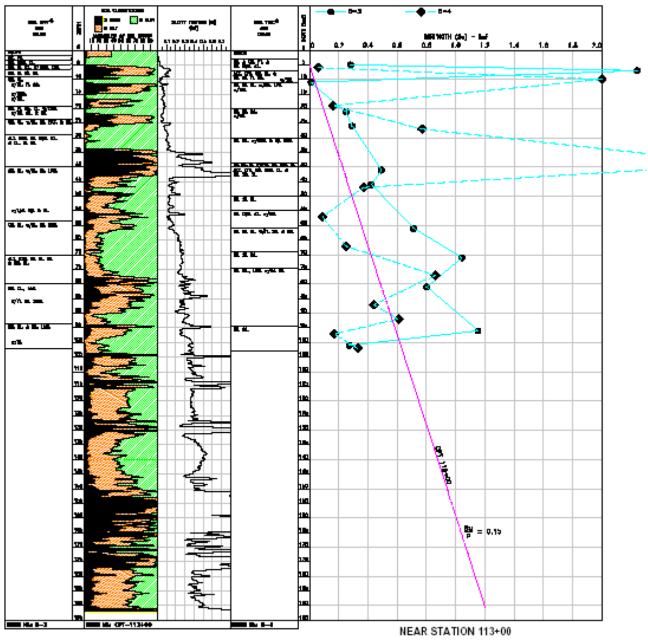

Figure 4. Plot of shear strengths of 113+00

One can verify this normalized shear strength using SHANSEP concept. This concept states that if the shear strength of a normally consolidated clay is known, the shear strengths for various stress histories of the same clay can be predicted by:

$$\left(\frac{S_u}{P}\right)_{OC} = \left(\frac{S_u}{P}\right)_{NC} OCR^m$$

where the average m value is 0.75.

Using an average $(Su/P)_{NC}$ value of 0.25, one can determine the (Su/P) ratio for a clay with an OCR of 0.6 to be 0.17. This value is identical to that determined from CPT soundings and laboratory testing as shown on Figures 4 and 5.

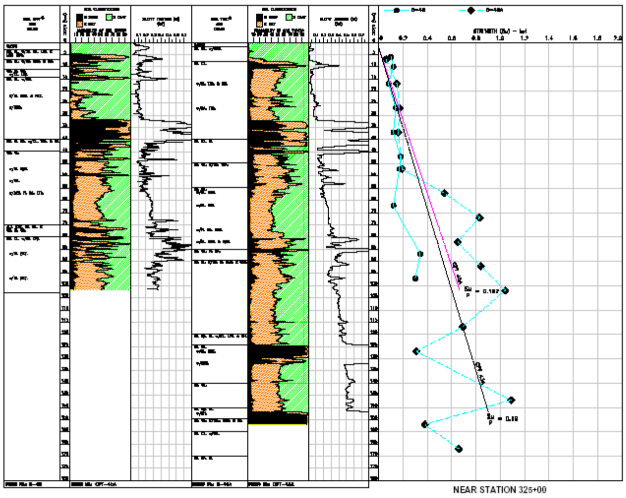

Figure 5. Plot of shear strengths of 325+00

2.2 Compressibility

Settlement is a design consideration for the approach ramps, the areas near them, and the pile foundations. Settlement depends on the applied loadings, the geologic conditions, and the compressibility of the soils. Twenty-eight consolidation tests were performed for both Phase 1A and Phase 1B segments of this project to evaluate the compressibility of the existing soils. These tests were performed on the soil borings at different depths and places along the proposed route of the new LA-1. Table 1 presents a summary of the samples and test results used in the compressibility study and the values of the parameters determined for each sample.

Four parameters are essential for settlement estimates: compression index (C_c), recompression index (C_r), coefficient of consolidation (C_v), and preconsolidation pressure (p_c). The compression index determines the strain of the soils due to stress change for a normally consolidated soil.

The recompression index is similar to the compression index; it is for settlement estimates below the stress level that the soils had been subjected to previously (p_c). For the soils at this site, the recompression index is not a factor; however, a correlation was made to verify the adequacy of the testing. The coefficient of consolidation is used to estimate the rate of consolidation.

In order to establish a reasonable model to evaluate total and time rates of settlements for this project, a parametric study was performed to evaluate the most probable compressibility parameters. This study employed the standard mathematical technique of “Least Square Regression Analyses.” It reduces the trend of the data to an equation. The “goodness of fit” of the resulting equation is measured by the statistical parameter (R^2). For example. If (R^2=0.6), 60% of the variation in the dependent parameter (e.g., recompression index) are explained by variation in the independent parameter (e.g. compression index). The following parameters were studied to determine the correlation, if exists, and to evaluate the reasonableness of the parameters selected.

- Recompression (Cr) and Normal Compression Indices (Cc)
- Moisture Content (W_c)
- Liquid Limit (LL)
- Initial Void Ratio (e_0)

Figure 6 presents the relationship between the recompression index and compression index.

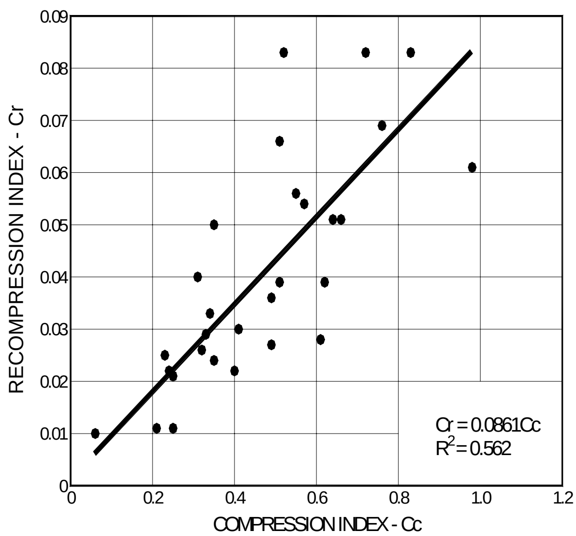

Figure 6. Recompression Index as Function of the Primary Compression Index

For saturated clays, Kulhawy and Mayne(1990) suggested that the recompression index is about 20 percent of the compression index. Nagaraj and Murthy(1985) indicated that the swell index is between 10 to 20 percent of the compression index. The swell index is the same or slightly greater than the recompression index; therefore, the recompression index should also range slightly smaller than 10 to 20 percent of the compression index. The correlation shown on Figure 6 indicates that the recompression index is approximately 9 percent of the compression index, on the lower bound of Nagaraj and Murthy's correlation. STE's 30 year experience wit Louisiana clays confirm that the ratio of the indices should be about 10 percent.

As a check, a relationship between moisture content and initial void ratio relationship was developed to evaluate the effect, if any, of sample disturbance. Figure 7 represents the correlation between these two parameters. For saturated soils, the initial void ratio is the product of the moisture content and specific gravity. For unsaturated soils, saturation times void ratio of a soil would be the same as the product of specific gravity and moisture content of the soil. Figure 7 indicates that the average ratio of specific gravity to saturation is 2.81. The actual measured average specific gravity is 2.7, indicating the initial saturation immediately prior to testing is about 96%. This in turn, indicates only a small disturbance during sampling and test preparation.

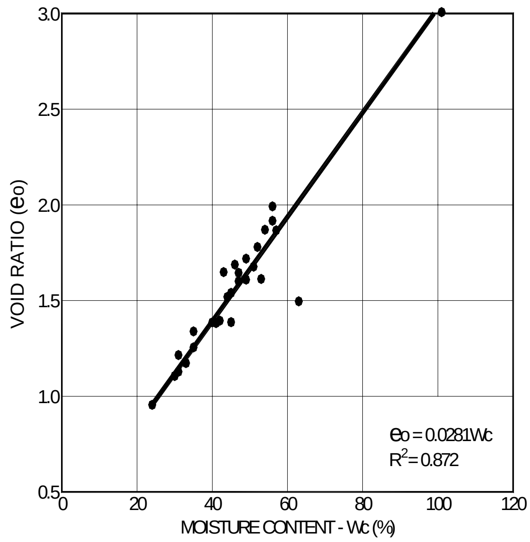

Figure 7. Initial Void Ratio as a Function of the Moisture Content

Figure 8 shows the relationship between liquid limit and compression index developed from the data for this specific project (see Table 1). There are also several published correlations that exist using this form. The most noteable relationship is from Skempton (1944):

$$C_c = 0.009\ (LL - 10)$$

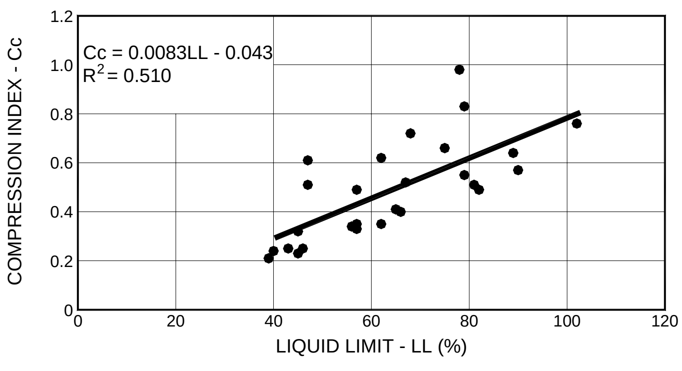

Figure 8. Primary Compression Index as a Function of the Liquid Limit

Table 1. Parameters for the Compressibility Study

Boring No.	Material Description	Depth (feet)	C_c	W_c (%)	LL (%)	e_0	C_r	OCR
T 1	Gray Clay w/ sand & shells	23 25	0.25	35	43	1.005	0.011	0.6
	Gray Clay w/ silt & sand	53 55	0.33	42	57	1.145	0.029	0.6
	Gray Clay	113 115	0.51	41	81	1.132	0.039	0.6
T 2	Gray Clay w/ silty fine sand	48 50	0.35	44	57	1.269	0.024	0.7
	Gray Clay w/ sand	78 80	0.66	56	75	1.667	0.051	0.7
T 3	Gray Organic Clay w/roots	6 8	0.40	49	66	1.359	0.022	2.6
	Gray Clay	28 30	0.62	54	62	1.620	0.039	0.7
	Gray Clay	63 65	0.55	52	79	1.530	0.059	0.5
	Gray Clay	113 115	0.41	35	65	1.089	0.030	0.7
T 4	Gray Clay w/ sand	38 40	0.49	47	57	1.351	0.027	0.7
	Gray Very Silty Clay w/ sand	63 65	0.24	31	40	0.965	0.022	0.2
	Gray Clay w/ sand & shells	128 130	0.49	45	82	1.290	0.036	0.6
T 5	Gray Clay w/ sand & silt	18 20	0.61	56	47	1.742	0.028	0.5
	Gray Silty Clay w/ sand & organics	58 60	0.32	40	45	1.136	0.026	0.5
	Gray Silty Clay w/ sand	98 100	0.21	30	39	0.856	0.011	0.8
T 6	Gray Organic Clay	13 15	0.98	101	78	2.757	0.061	0.8
	Gray Organic Clay	88 90	0.76	57	102	1.617	0.069	0.8
	Gray Organic Clay w/ silt	148 150	0.64	46	89	1.438	0.051	0.8
BR 002	Gray Clay w/silt seams	105 107	0.72	43	68	1.399	0.083	1.2
	Gray Silty Clay w/ silt seams	160 162	0.23	33	45	0.923	0.025	0.7
BR 003	Dark Gray Clay	140 142	0.83	53	79	1.363	0.083	1.5
B 12A	Gray Organic Silty Clay	86 88	0.31	N/A	N/A	1.223	0.04	0.5
B 61	Gray & Brown Clay w/ shells	42 44	0.35	51	62	1.427	0.05	N/A
	Gray Clay w/ silt seams	97 99	0.52	63	67	1.246	0.083	0.6
B 63	Gray Clay w/ silt pockets	103 105	0.34	45	56	1.137	0.033	0.3
B 69	Gray Silty Clay w/ silt seams	123 125	0.25	31	46	0.878	0.021	1.0
B 78	Dark Gray Silty Clay	86 88	0.51	49	47	1.469	0.066	0.2
B 80	Dark Gray Clay	76 78	0.57	47	90	1.395	0.054	1.2

This correlation was established from testing on normally consolidated, unsaturated, uncemented clay. Terzaghi and Peck (1948) modifed it for the clays of low to moderate sensitivity to be:

$$C_c = 0.007\ (LL - 10)$$

From the above figure, the relationship for the soils of this project can be written as:

$$C_c = 0.0083\ (LL\ 5.2)$$

The coefficients of this equation differ slightly from those in both Skempton's and Terzaghi and Peck's equations; however, the results clearly showed the validity of the tests.

Frequently, the compression index is correlated to moisture content in relationships such as:

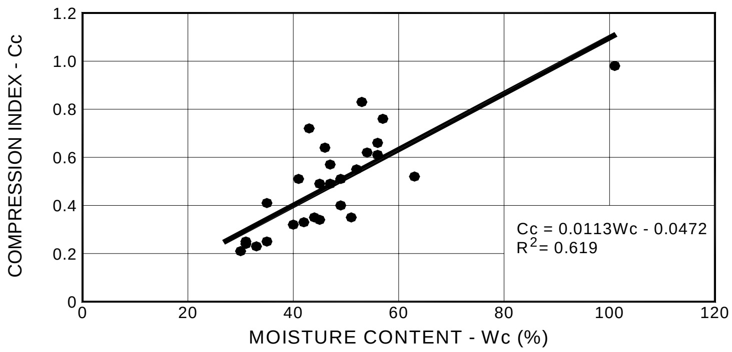

Figure 9. Primary Compression Index as a Function of Moisture Content

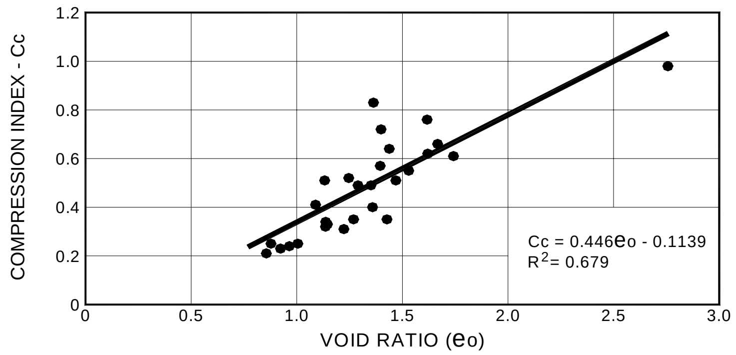

Figure 10. Primary Compression Index as a Function of Initial Void Ratio

C_c = 0.0115 W_c (Moran et al., 1958)
C_c = 0.014 W_c 0.16 (Deubert, 1982)

Deubert's equation is very significant to this project because it was derived from tests on soil samples from Louisiana with a similar geological background.

A correlation using the Moran/Deubert from and data from this project is shown on Figure 9.

This relationship is essentially the same as both Deubert's and Moran's, with an offset (constant term) between those of the two published correlations.

Another correlation of compression index that has been used extensively is initial void ratio. For example, Nishida(1956) established the following for normally consolidated clays with low sensitivity:

C_c = 0.54 (e_0 0.27)

For Brazilian clay, Cozzolino(1961) published the following correlation:

C_c = 0.43 (e_0 0.25)

Figure 10 presents the correlation from this project. The results indicate the following correlation:

C_c = 0.44 (e_0 0.26)

This equation is almost identical to Cozzolino's. It should be noted that this correlation has a much higher coefficient of correlation than the correlations developed for this project using liquid limits or moisture contents; therefore, it provides a better method

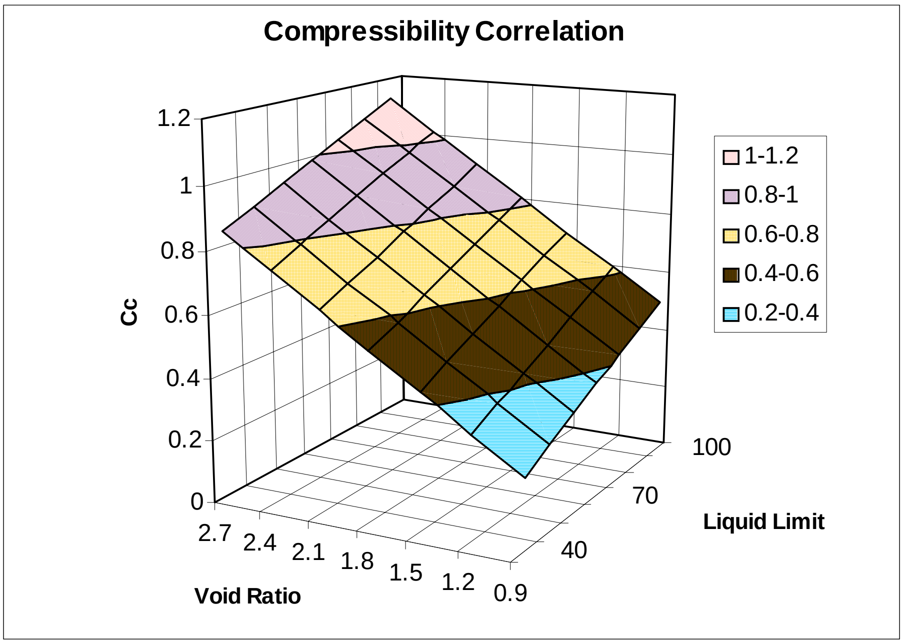

Figure 11. Compression Index vs. Void Ratio and Liquid Limit

for estimating the compression index for the soils at this site.

Since, the factors contributing to the compressibility involve both material type and soil fabric, it is reasonable to relate compressibility to both liquid limit and void ratio. Figure 11 presents such a correlation.

The correlation equation plotted in Figure 11 can be expressed as:

$$C_c = 0.0048\ LL + 0.342\ e_0 - 0.269$$

Where (LL) is expressed as percent. This equation shows that the compression index increases with increasing plasticity as well as with increasing initial void ratio. It is noteworthy to show that this correlation has a Coefficient of Correlation of the above relationship is 0.92 and its standard estimate of error is 0.08; both indicate a very strong correlation. A similar correlation was also performed by Al Khafaji and Andersland(1992) on highly plastic clays. Their correlation is:

$$C_c = 0.00058\ LL + 0.411\ e_0 - 0.156$$

They shows a greater impact from void ratio that the soils from this site and much weaker correlation to material type. Azzouz et al. (1976) showed a much closer correlation to that established from this project:

$$C_c = 0.0011\ LL + 0.37\ e_0 - 0.126$$

All three correlations produce very high coefficents of correlation and very low standard estimate of errors. The correlation developed specifically for this project appears to be reasonable.

2.3 Coefficient of Consolidation

The coefficients of consolidation from the consolidation tests were tabulated previously. Most published correlations relate the coefficient of consolidation to void ratio and in situ effective stress. These correlations are typically quite weak and unreliable, primarily due to the heterogeneity of the soil deposits. For this

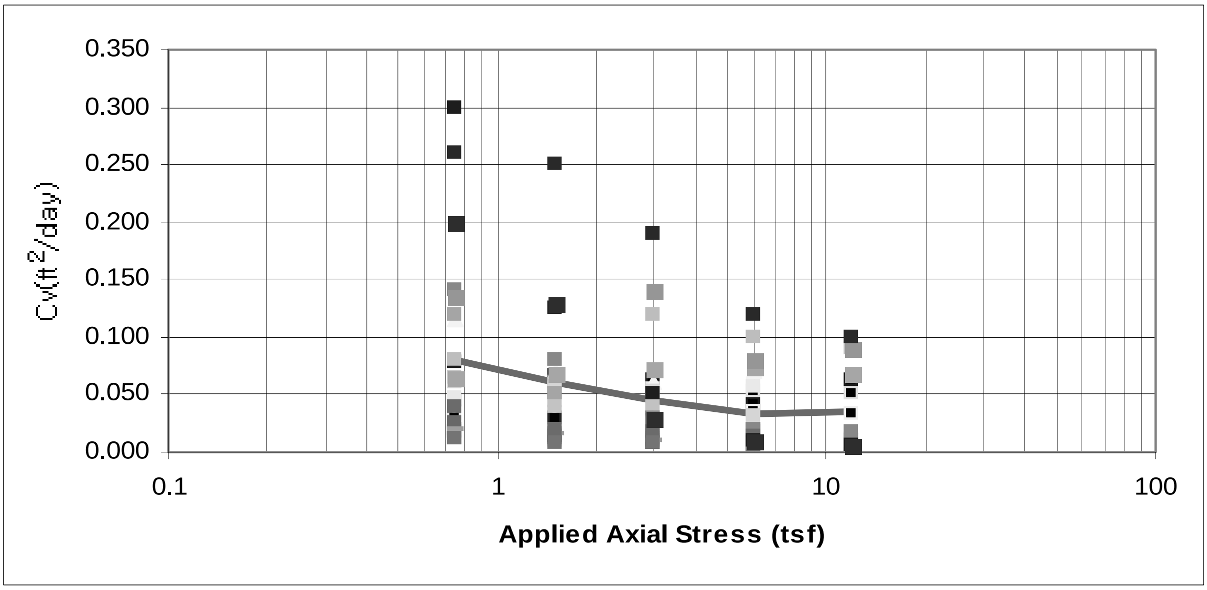

Figure 12. Coefficient of Consolidation as a Function of Applied Axial Stress. The line represents the average values.

project, the coefficients of consolidation are plotted as a function of the applied axial stresses and the results are presented in Figure 12. The average value for the coefficient of consolidation at each applied stress was computed and the points connected by a line, showing, in general terms, a slight decrease in the coefficient of consolidation with increasing of the applied axial stress.

From the results, the average coefficients of consolidation of 0.03ft^2/day and 0.07ft^2/day were obtained from laboratory testing, depending on the stress applied. However, in practice it is well known that the effective coefficient of consolidation can be much higher due to sand lenses and lateral drainage. According to the published literatures, values of the coefficient of consolidation in the field may be as high as 100 times greater than the coefficient of consolidation obtained in the laboratory.

A conservative value of 0.15ft^2/day, representing approximately two to five times the average values obtained from laboratory tests, was assumed for the time rate of settlement analyses.

3. CONCLUSION

Contruction project on sensitive environmental area should be performed based on thorough and detail paln of soil investigation and laboratory tests. The improvement of resolution of CPT can detect thin silt/sand seams during penetration. The wireless CPT need to work together with memory chip to back up sporadic data loss. To obtain reliable soil parameters, the prevention of sample disturbance is critical in soft ground soil investigation. The stress history of soil condition can be verified by relation between OCR and shear stregnth. Static pile estimation provided best predicted pile capacity than any other methods. This static methods should be applied based on thorough and reliable soil parameters. Using Piezocone dissipation tests provide brief estimation of the degree of consolidation.

(접수일자 : 2006년 1월 5일)

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