GEOTECHNICAL CHARACTERIZATION OF SOFT CLAY ALONG A HIGHWAY IN THE RED RIVER DELTA

P. H. Giao [1] and D. H. Hien [2]

ABSTRACT: The Red River Delta (RRD) consists mostly of Pleistocene to Holocene deposits, including soft clays. Besides environmental hazards such as flood, storm, coastal erosion, saltwater instruction and contamination of ground water, the wide distribution of soft clay has caused obvious geotechnical difficulties for infrastructure development projects. It is observed that the economic growth of this booming region of Vietnam goes at a faster rate than that of the infrastructure development and the latter has not yet been supported by a modern geotechnical investigation practice. Data analysis and soil characterization become even more difficult for a long linear infrastructure like a road or highway, whose route runs over different soil types. This study deals with a comprehensive geotechnical characterization of soft soils underlying the national highway No. 18 (NH18) that has often had problems of differential settlements or other construction damages. Besides the common approach of lumping testing data in the averaged graphs and tables, visualizations were made to assist in characterization of the soil layers. A number of empirical correlative relationships were deduced for various geotechnical parameters, especially the undrained shear strength and the cone tip resistance.

Keywords: Soft clay, Red River delta, data visualization, geotechnical characterization, site investigation
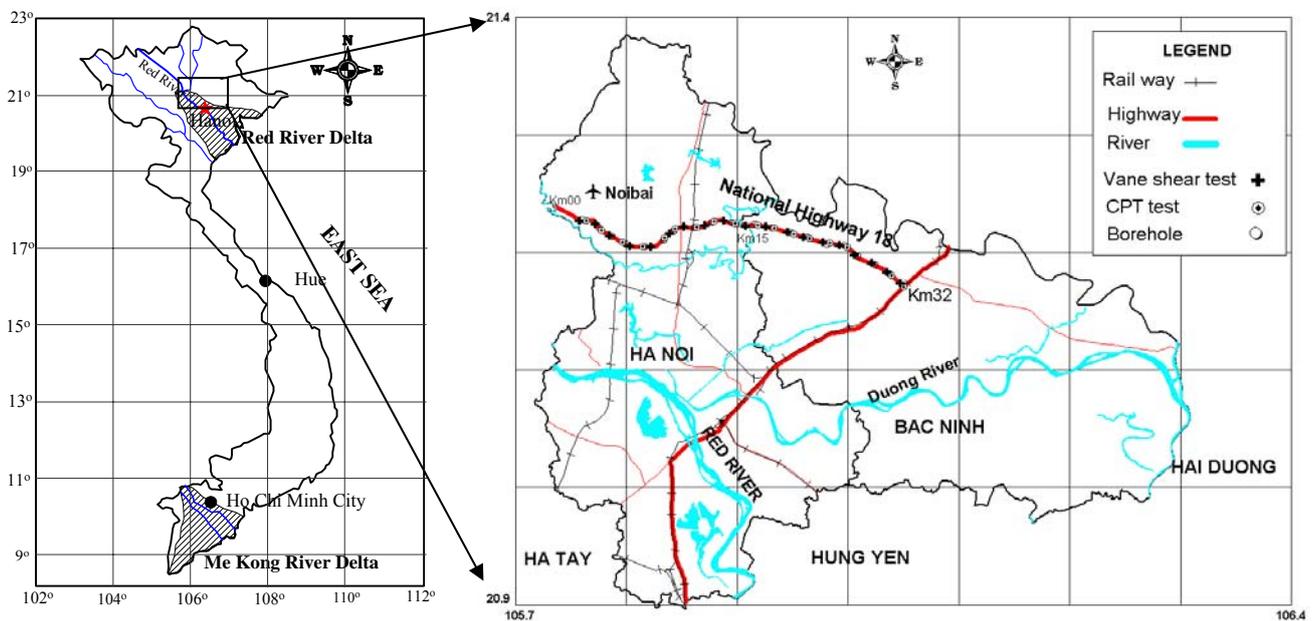
INTRODUCTION

Fig. 1 Location of the national highway 18 (NH18) in the Red River Delta

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The Red River Delta (RRD) is the second largest delta in Vietnam with an area of 150,000 km^2 and considered one of the fastest economically growing regions of the country. Its location is shown in Fig 1. Annually, amount of goods, agricultural and industrial products have been produced more and more that need to be distributed to the other regions inside and outside the delta. However, the quality of many existing highways is being deteriorated due to simple construction method and an ever-increasing traffic loading. Consequently, the highways improvement and widening are necessary. A large-scale project funded by World Bank, named as Road Network Improvement Program (World Bank, 2005) has such a purpose to improve the quality of national highways in the RRD. The highway upgrading includes site investigation and construction works for widening and heightening of the road embankment. Consequently, many geotechnical investigations have been carried out along a number of national highways (NH21, NH38B, NH10, NH39 and NH18), producing large amounts of data that need to be analyzed. This paper deals with the detailed geotechnical characterization of the RRD soft clay underlying the national highway 18 with the following scopes: i) collection and reanalysis of geotechnical data using the approach proposed by Leroueil (1989) in which the properties of the clays are presented in three states, i.e., remolded, intact and in the passage from the intact to the destructred-remolded conditions; ii) application of visualization techniques for displaying and characterizing the investigation data along a long and linear infrastructure; and iii) depicting the useful role of fast in-situ testing techniques such as vane shear test and CPT for a long structure like a highway; iv) correlation between data to provide a set of reliable geotechnical parameters that can be used in settlement calculation as needed for highway upgrading works.

SAMPLING, TESTING AND INITIAL DATA ANALYSIS

The clays along the NH18 belong to Vinhphuc formation ($Q_{IV}^3 vp$) of late Pleistocene. During this period, as the maximum marine transgression reached, the deltaic, lacustrine and swamp sediments were deposited (Nghi et al., 1991). The route of the NH18 from Noibai airport to Bac Ninh province is almost in the W-E direction as shown in Fig 1. In the Road Network Improvement Program project (World Bank, 2005), a total of 204 boreholes with maximum depth of 20m were drilled along this highway. The boreholes are distributed along the highway with a distance between two boreholes from 0.2 to 0.5 km. The soil samples are taken at every 2 m by a 0.8-m long Shelby tube. Distributions of the boreholes and samples are plotted in Fig. 2. The samples were brought to the laboratory for the following geotechnical tests, i.e., grain size, water content, unit weight, Atterberg's limits, conventional consolidation, undrained unconsolidated triaxial and unconfined compression tests. Except the grain size analysis tests that were done on all the samples, the other laboratory tests were performed only on a certain number of them. In addition, two types of field testing were done, and namely, vane shear test (VST) and cone penetration test (CPT).
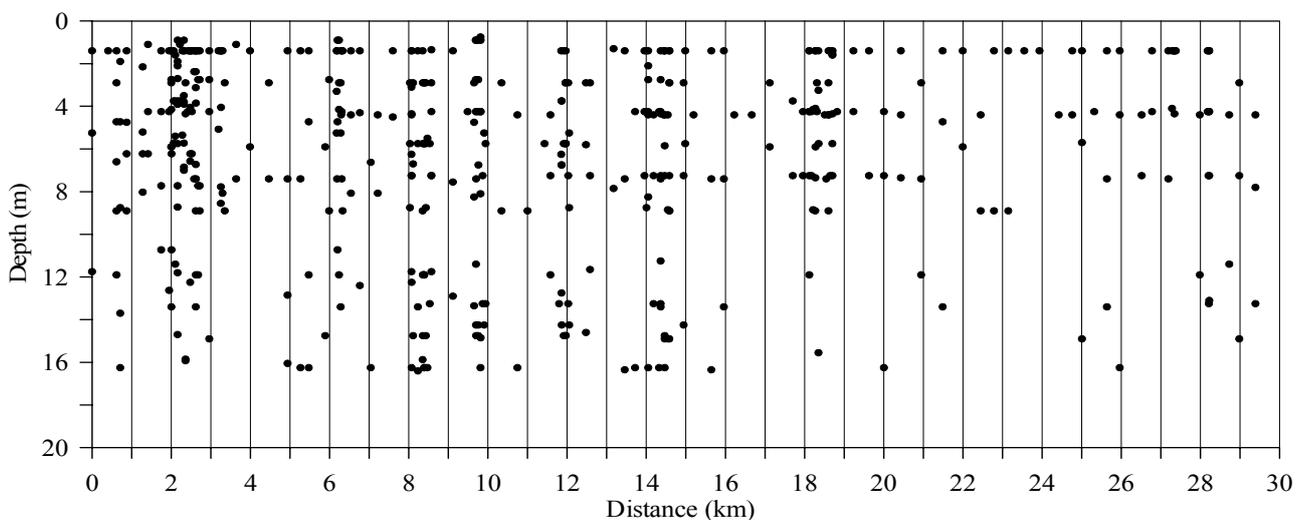

Fig. 2 Distributions of the boreholes and collected samples along the NH 18

The initial analysis was done using a common approach, any geotechnical engineer used to do, which is to compile all the tested data and plot them versus depth. Such a geotechnical profile was synthesized for the NH18 as shown in Fig. 3a and 3b for two sets of data, corresponding two segments of the highway, Km.0 - Km. 15 and Km. 15- Km. 32, respectively. From Fig. 3a &b

one can see that until the depth of 20 m the subsoil underlying the NH18 can be divided into four sublayers, i.e., a fill material layer from the ground surface to 1.0 m depth; the soft to medium clay from 1.0 to 5.0 m; the sand layer from 5.0 up to 17.0m, and below this sand layer is a medium to stiff clay layer, which was not fully penetrated by the boreholes.
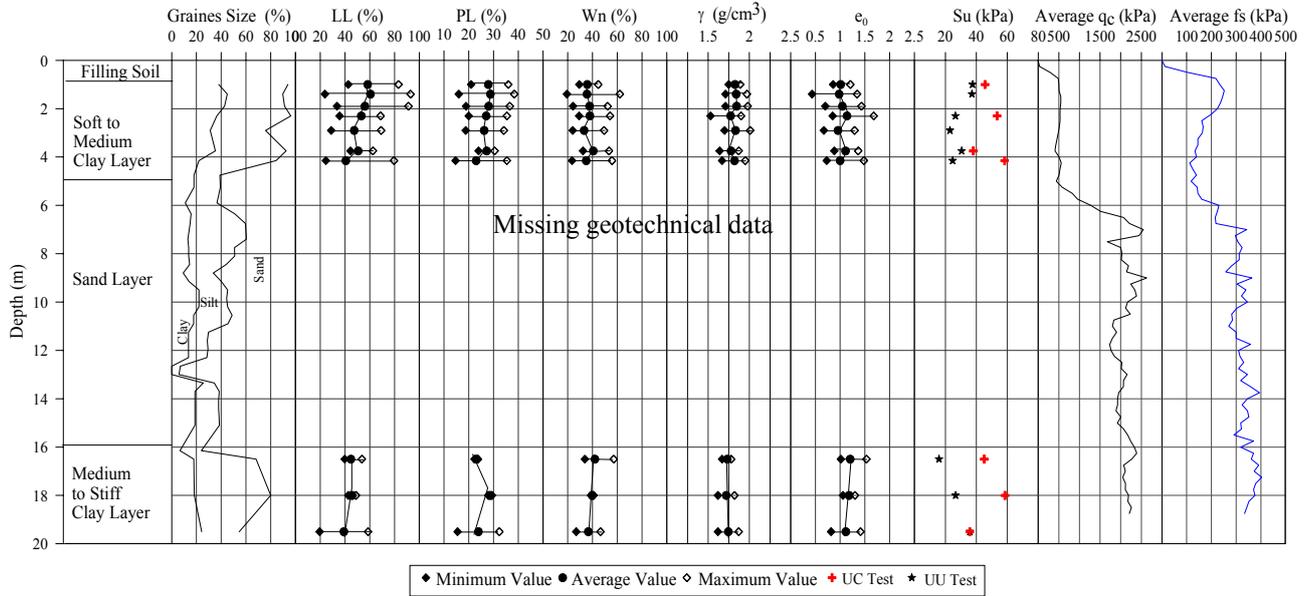
(a) Km. 00 – Km. 15
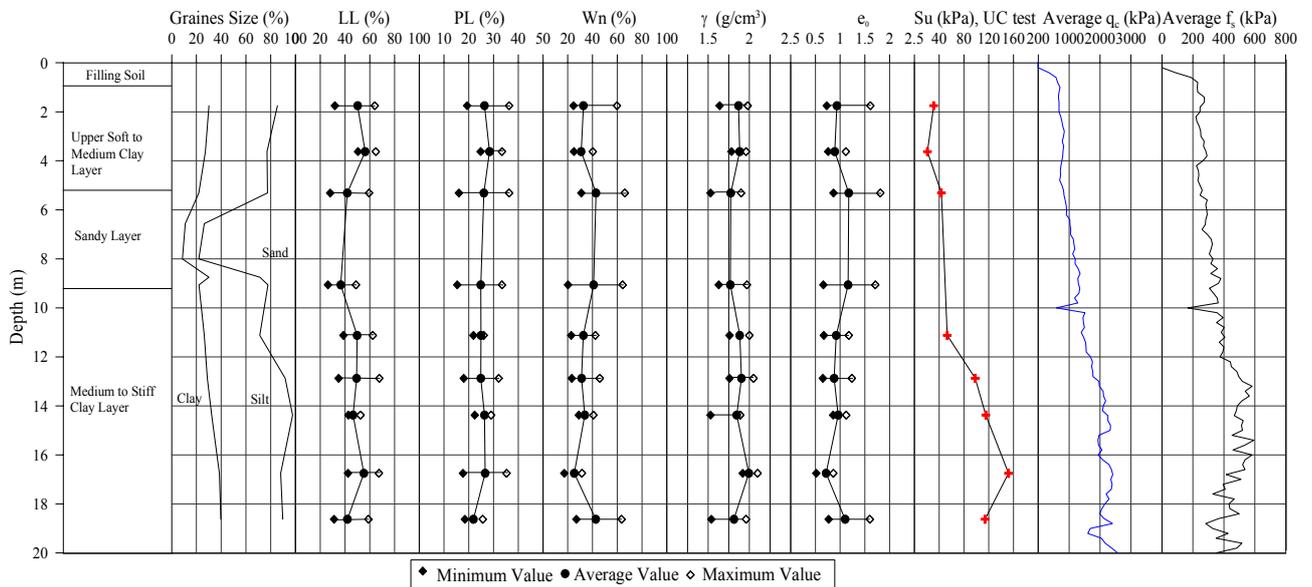
(b) Km. 15 – Km. 32

Fig. 3 A summarized geotechnical profile for the NH18

The following difficulties were found when characterizing the subsoil underneath the NH18: (i) Data are missing for the sand layer intercalated between the

upper soft to medium clay and the lower medium to stiff clay due to unavailability of samples and hence the testing results; (ii) Although a large amount of

samples were taken and many tests were done, i.e., among 860 collected samples, 170 consolidation tests, 150 undrained unconsolidation (UU) triaxial tests and 210 unconfined compressions (UC) tests were performed, testing results still seem to be incomplete because of the very long route; (iii) It is difficult to find out the typical range of geotechnical parameters for each layer because of their wide variations due to both geological and testing quality reasons.

VISUALIZATION OF THE SUBSOIL

Characterization of the subsoil along a long highway passing through different geological units is certainly difficult. Visualization techniques were proposed to help solving this task. Because the NH18 is not exactly linear, its route was divided into 7 segments as shown in Fig 4, which are connected together at the end of analysis to make a complete profile. Visualization was done using the software ROCKWORKS 2006 of Rockware (http://www.rockware.com).
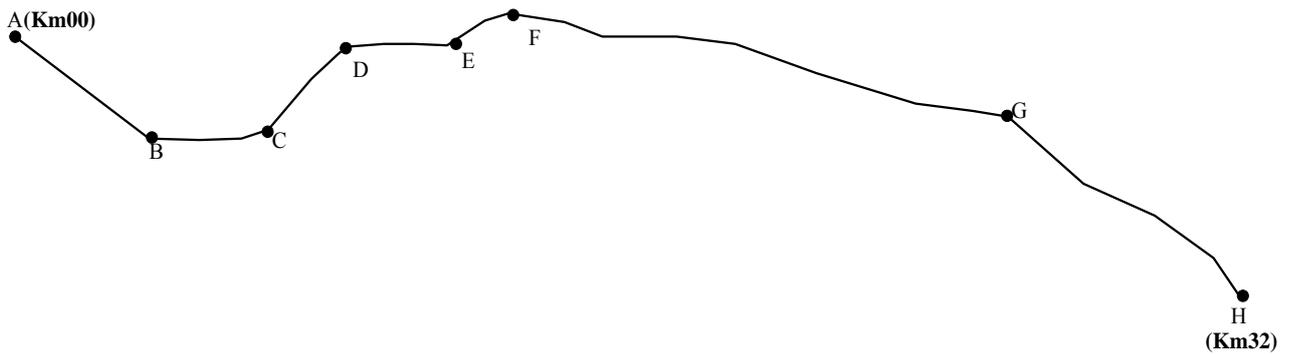

Fig. 4 Separation of the NH18 route in seven segments for visualization
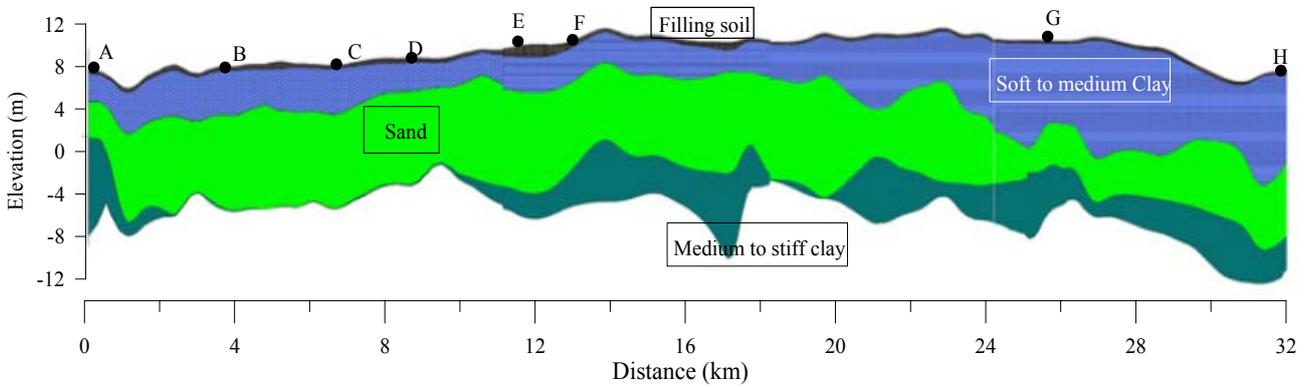

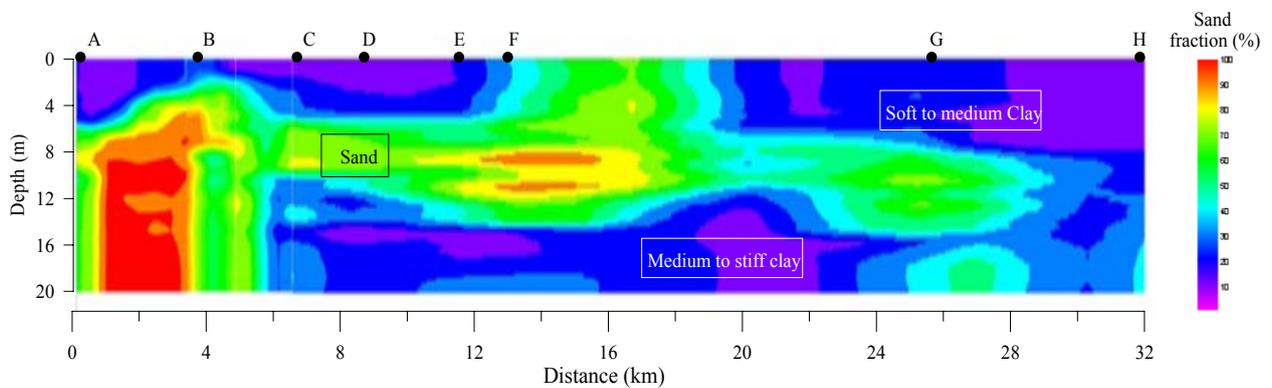Fig. 5 Visualization of the NH18 subsoil based on the field bore logs

Fig. 6 Visualization of the NH18 subsoil based on the sand fraction (0.075-2mm) determined from sieve analyses

Fig. 5 shows visualization of the soil layers based on the field bore logs. The subsoil appears to be consisting clearly of four units, and namely, (i) a thin filling material layer on top; (ii) the upper soft to medium clay layer, which is about 4 meter thick from Km. 00 to Km. 20, and then is thickening towards the end of the highway at Km. 32, reaching 8 m thick in this later part; (iii) the sand layer is underlying the upper soft to medium clay, it has a thickness of about 8 m until Km. 20 and gets thinner after that; (iv) the lower medium to stiff clay layer is underlying the sand layer. Until Km. 8 this layer does not show up. The field bore log descriptions seems to indicate a thin clay layer of variable thickness of around 4 m or less, which may occur in form of some lenses.

As the grain size analysis test results are available for all the depth levels, visualization of sand fraction change with depth is done for the entire route of the NH18 as shown in Fig. 6, which confirms the existences of three main layers of soft to medium clay, intercalated sand and the lower medium to stiff clay.

GEOTECHNICAL CHARACTERISTICS OF THE CLAY LAYERS

Geotechnical characteristics of the two identified clay layers were analyzed and presented, following Leroueil's approach (Leroueil, 1989) that consists of classifying and presenting the geotechnical properties of the investigated clay into three groups, i.e., (i) properties of the remolded clay (Atterberg's limits, activity); (ii) properties of the intact clay (OCR, undrained shear strength, net cone tip resistance); (iii) properties associated with passage from the intact to destructured-remolded conditions (sensitivity, compression index).

Water Contents and Atterberg Limits

The averaged water contents and Atterberg limits of the upper and lower clays are quite similar, water content varies mostly from 30 to 40%, LL from 40 to 60%, and PL from 20 to 30%. The plasticity chart is plotted in Fig. 7. The points of these clays follow more or less the A-line. Those points of the upper soft to medium clay indicate that it is predominantly of the CH type with higher plasticity, while most of the lower medium to stiff clay is of the CL type with low plasticity.

Activity

Fig. 8 shows the activity chart for the clays under the NH18. The activity values of the upper and lower clays
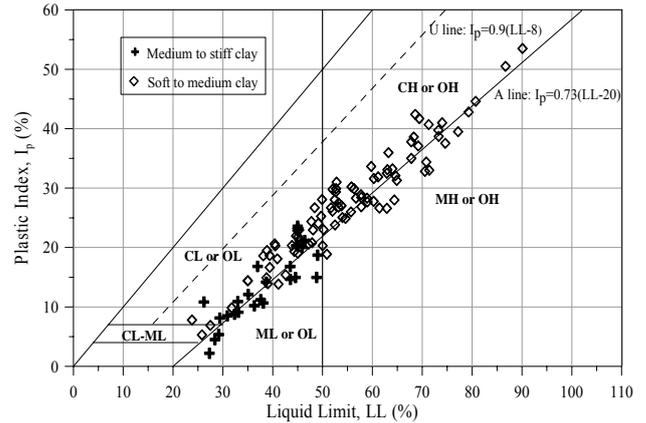
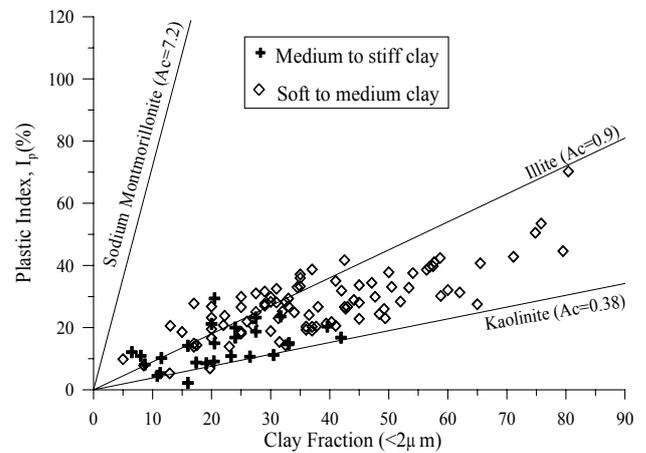Fig. 7 Plasticity chart of the upper and lower clays along the NH18

Fig. 8 Activity chart of the RRD clay along the NH18

along the highway mainly varies from 0.38 to 1.38. For the high clay fraction the activity is between 0.38 and 0.9, indicating the predominant clay minerals are illite and kaolinite.

Preconsolidation Pressure and OCR

The preconsolidation pressure (σ_{oc}') was determined based on the conventional oedometer tests (the load increment ratio is 1 and the duration of each increment loading is 2 hours) performed on samples taken with 76.2 mm diameter and 20mm thickness. The preconsolidation pressures of both upper and lower clays are shown in Fig. 9, decreasing from 140 kPa to 50 kPa with the increasing depth. The OCR values calculated based on the oedometer tests are shown in Fig. 10. They vary from 0.35 to 4 and the OCRs of the samples taken below 12 m the OCRs are found less than 1. Such a trend of decreasing preconsolidation pressure and OCR with depth is not

normal and clearly indicated of either soil disturbance or improper procedure of testing. Several causes to the low OCRs are suspected as follows, i.e., (i) the 2-hour increment loading rate as adopted by the local procedure is quick, therefore the void ratio obtained may be less comparing to that by ASTM standard (ASTM Designation D 2435-96), in which the increment loading rate is 24 hours; (ii) Soil disturbance, induced by the sampling techniques, can also cause low σ_p' and low OCRs. Man (2003) compared the sampling techniques by piston and

Shelby tube samplers for Hai Phong clay and found that the Shelby tube sampler caused more disturbance than the piston sampler; (iii) Testing methods can also influence the evaluated values of OCR. Leroueil (1996) and Tanaka & Tanaka (1999) had mentioned that preconsolidation pressures determined from a conventional oedometer test are definitely lower than those obtained from constant rate of strain (CRS) tests due to strain rate effects.
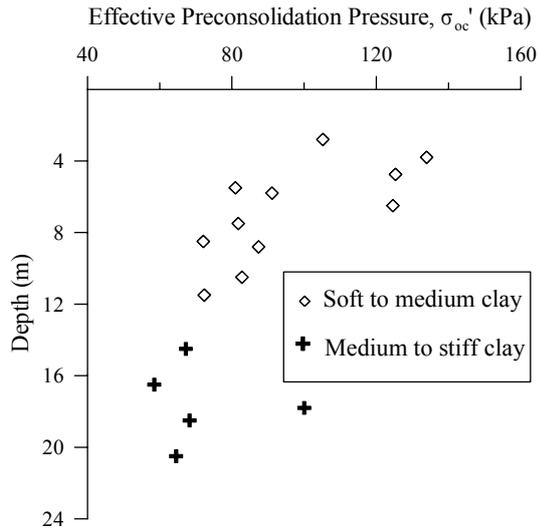
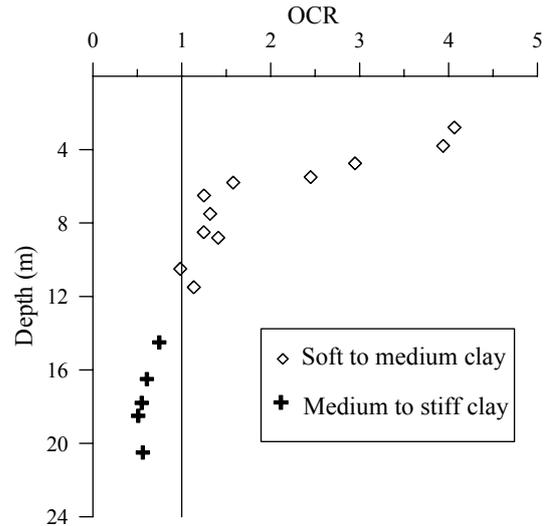Fig. 9 Preconsolidation Pressure of clays underneath the NH18

Fig. 10   The OCRs of clays underneath the NH18 as determined from Consolidation Test

Compression Index

The compression index, Cc, of the RRD clay along NH 18, was correlated with a few geotechnical parameters such as water content, void ratio, sensitivity and swelling index. Correlations in the form of y = a.x were made separately for two main segments, Km00-Km15 and Km15-Km32, as well as for the entire NH18 route from Km00 to Km32. As they yielded similar relationships with quite high correlation coefficients we presented only the relationships that were obtained for the entire NH18 route as shown in Fig. 11a-d.

The correlation between compression index (Cc) and water content (Wn) is shown in Fig. 11a and rewritten below:

$$Cc = 0.005*Wn \tag{1}$$

As compression index is much influenced by the structure of natural clays, it can be well related to void ratio and sensitivity, which reflect the structure (Leroueil

et al., 1983). The compression index, Cc, is plotted against the natural void ratio, e_0, in Fig. 11b. The correlation between the compression index and initial void ratio was found as follows:

$$Cc = 0.181*e_0 \tag{2}$$

Figure 11c depicts the correlation between compression index, Cc, and swelling index, Cs:

$$Cs = 0.227*Cc \tag{3}$$

Fig. 11d shows that the average sensitivity values calculated for both upper and lower clays along the NH18 follow almost exactly the 1-4 line of sensitivity.

The swelling index is useful when one would like to calculate the primary consolidation settlement of both upper and lower clay layers under a surcharge of ΔP using a bilinear compression model as given below:
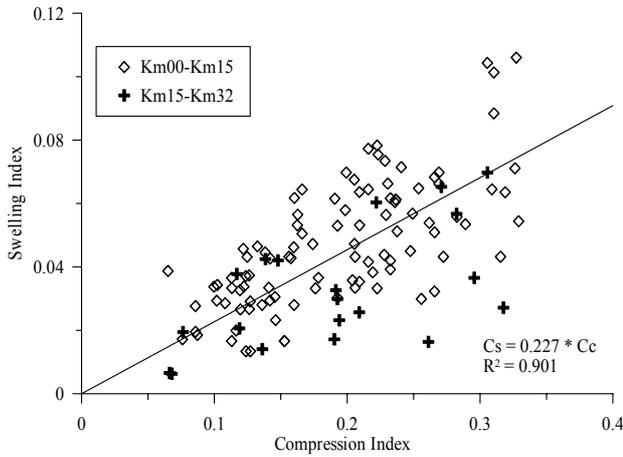
$$S = \frac{H.C_S}{1+e_0} \log\left(\frac{\sigma_{oc}{}'}{\sigma_o{}'}\right) + \frac{H.C_C}{1+e_0} \log\left(\frac{\sigma_o{}'+\Delta P}{\sigma_{oc}{}'}\right) \tag{4}$$
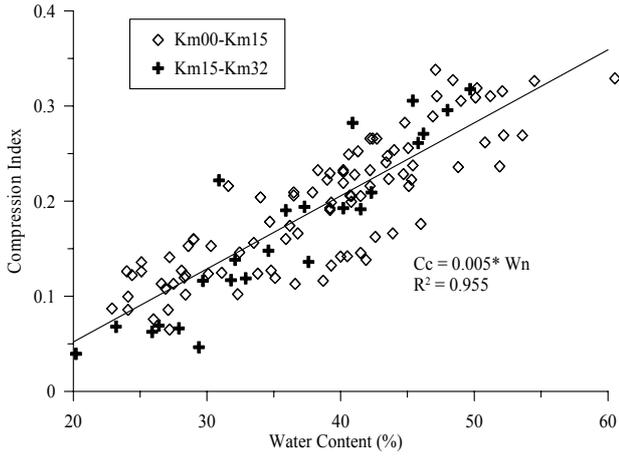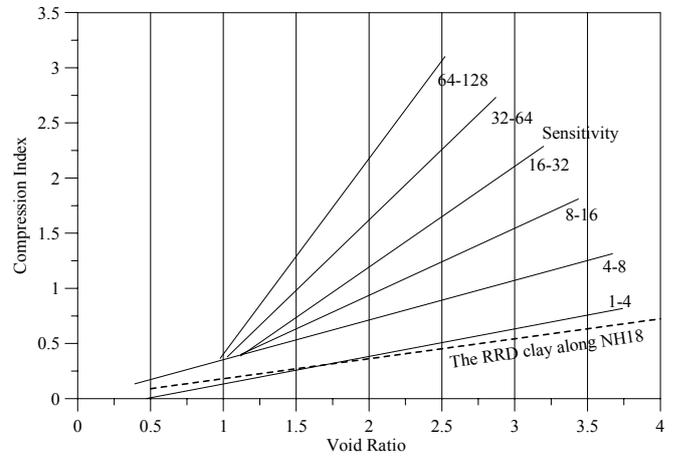
a) Correlation between Cc and Wn from Km00-Km32
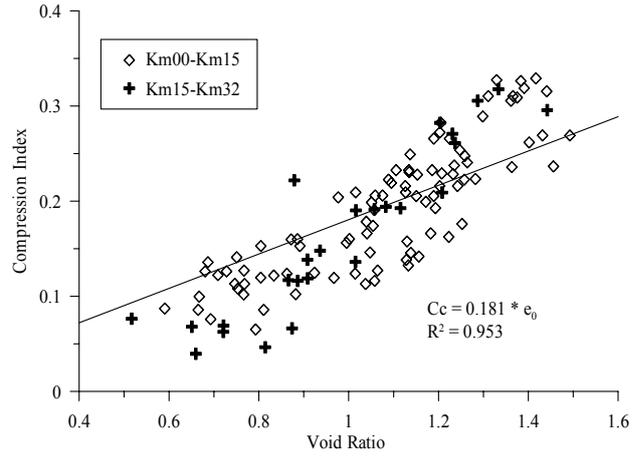
b) Correlation between Cc and e_o from Km00-Km32

c) Correlation between Cc and e_o from Km00-Km32

d) A general Cc-e_0 relationship (Leroueil et al., 1983) and sensitivity values of NH18 clays

Fig. 11 Correlation between compression index Cc and some other geotechnical parameters

where S is primary consolidation settlement; σ_o', σ_{oc}', H and e_0 are effective overburden pressure, preconsolidation pressure, thickness of clay layer and initial void ratio, respectively.

Undrained Shear Strength from Field Vane Shear Test (VST)

The vane shear tests were carried out by using equipment GEOTECH-NILCON N[0]163 (Sweden). The vane has a 65-mm diameter and 130-mm height. Fig. 12a shows the average undrained shear strength profile by vane shear test with its standard deviation at different depth levels. The vane test could be done only in the upper clay layer and top part of the sand layer, up to a depth of about 12 m. As seen in Fig. 12a the average undrained shear strength of the RRD clay along NH18 varies from 35 to 52 kPa, indicating a soft to medium clay.

Mayne and Mitchell (1988) derived the following empirical relationship to estimate the overconsolidation ratio (OCR) of a natural clay deposit based on the undrained shear strength:

$$OCR = \beta \frac{S_{u(Field)}}{\sigma_v'} \tag{5}$$

where $S_{u(Field)}$ is the undrained shear strength from vane shear test, σ_v' is the effective overburden pressure and β is the function of plasticity index: $\beta = 22*(Ip)^{-0.48}$. To determine the OCR based on Eq. 5 one could use the values of the plasticity index value, Ip, and effective overburden pressure, σ_v', of the samples taken from a borehole next to the the vane shear test. Eq (5) was employed in this study and the calculated field OCR values are shown in Fig. 12b.
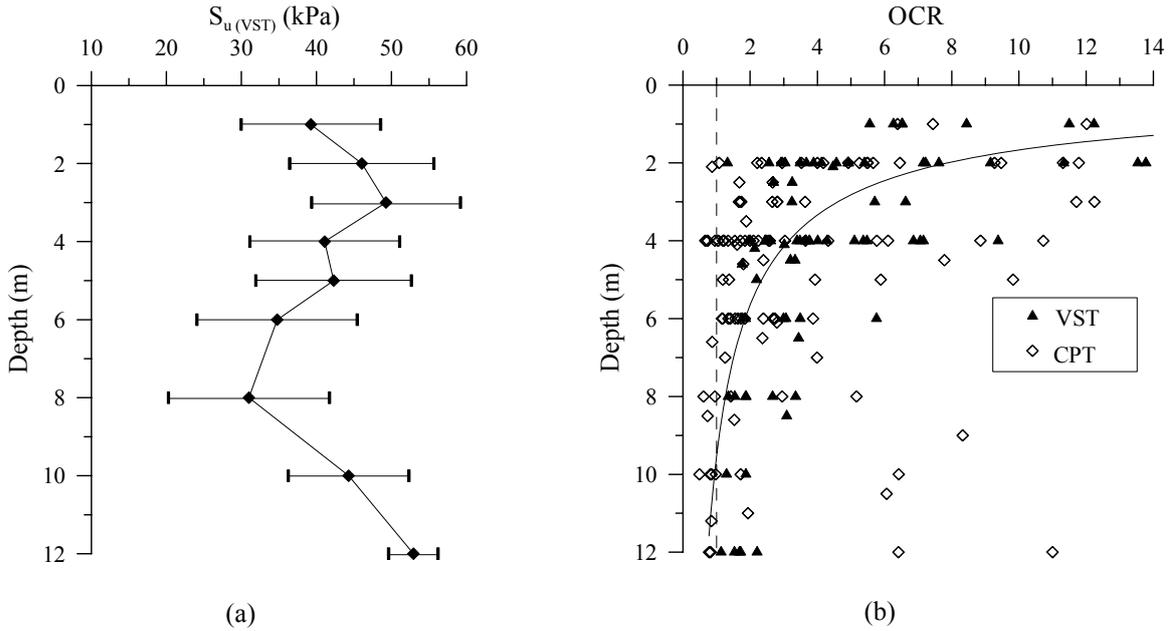
Fig. 12 (a) Undrained Shear Strength of the upper clay along NH18 by VST;
(b) The field OCRs determined from VST and CPT

Cone Penetration Test (CPT)

Equipment CPT GOUDA was used. It has a cone with sleeve friction with a diameter of 35.6mm, tip angle of 60°, and area of $10cm^2$. The measurements of cone resistance (q_c) and skin friction (fs), were taken at every 20cm.

OCRs can also be calculated from the CPTU test, using the following relationship (Lunne et al., 1997):

$$OCR = k(\frac{q_t - \sigma_v}{\sigma_v'}) \qquad (6)$$

Where q_t is the corrected cone tip resistance; σ_v and σ_v' are total and effective vertical stresses, respectively; k is a factor, varying from 0.2 to 0.5, depending on clay deposit. For clay along NH18, the k parameter is taken as 0.35 based on Man (2003).

The field OCR values based on CPT vary from less than 1.0 to 12.25 as seen in Fig. 12b.

The net cone resistance (q_t-σ_v') is correlated with the undrained shear strength from UC and UU triaxial tests as shown in Fig. 13a and 13b, and with the undrained shear strength from vane shear tests as shown in Fig. 14. From these correlations, one could obtain the cone factor, N_{kt}, which is 11.73 for the upper soft to medium clay for UC and UU test and 12.34 for vane shear test, respectively. For the lower medium to stiff clay layer N_{kt}, was found equal to 7.8 for UC and UU test.
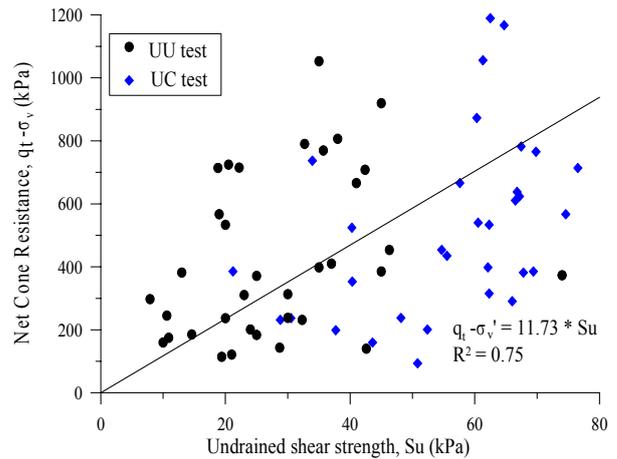

Fig. 13a Correlation between the Net Cone Resistance, q_t-σ_c, and Undrained Shear Strength from UU and UC Tests for the Upper Soft to Medium Clay

Summary of Geotechnical Characteristics of the Clays along the NH18 Highway

By reanalysis of a large amount of geotechnical data from both laboratory and field testing, the geotechnical characteristics of the upper and lower units of clays beneath the NH18 are obtained and summarized in Table 1.
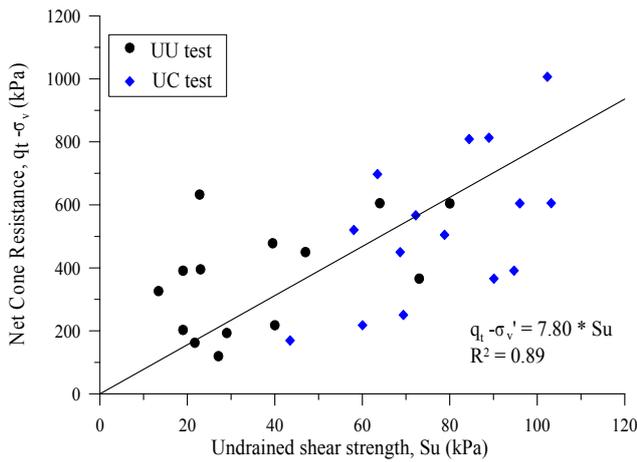
Fig. 13b Correlation between the Net Cone Resistance, q_t-σ_c, and Undrained Shear Strength from UU and UC Tests for the Lower Medium to Stiff Clay
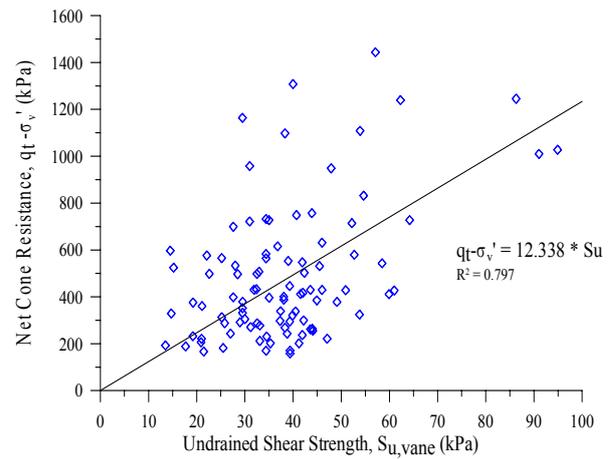

Fig. 14 Correlation between the Net Cone Resistance, q_t-σ_c, and Undrained Shear Strength from VST for the Upper Soft to Medium Clay

Table 1   Average Geotechnical Characteristics of the Clays along the NH18 Highway

Soil type	Depth (m)	Wn (%)	LL (%)	PL (%)	γ (g/cm^3)	Cv (mm^2/s)	Cc	Cs	e_0
Upper Soft to Medium Clay	1-7	39.32	54.9	27.60	1.79	0.17	0.221	0.042	1.051
Lower Medium to Stiff Clay	14-20	33.10	38.00	25.14	1.85	0.12	0.142	0.038	1.012

CONCLUSIONS

1) To improve soil characterization along such a long and linear geotechnical structure like the NH18 visualization was done for seven linear segments, which were then connected together to give a fully visualized section of the 32-km long highway. The subsoil underlying the NH18 can be characterized by four sublayers, i.e., on top is a fill material up to 1.0 m deep, which is followed by a soft to medium clay from 1.0 to 5.0 m, a sand layer from 5.0 up to 17.0m, and below this sand layer is a medium to stiff clay layer, which was not fully penetrated by the investigated boreholes.

2) The upper soft to medium clay is more of CH type, while the lower medium to stiff clay is more of CL type. The predominant clay minerals of clays under the NH21 are Illite and Kaolinite.

3) The OCR values determined from the oedometer tests show an abnormal decreasing trend with depths and is getting lower than 1.0 below the depth of 12m. Although this low OCR happens for the other clays as well due to the soil disturbance caused by sampling technique, the main cause is suspected in this study is the inadequate testing procedure of the oedometer test as performed by the local engineers with a short duration of loading increment (just 2 hours for all loading step instead of 24 hours).

4) Analyses of field test data (vane shear and cone penetration) were done. In the case of the NH18, eventhough a big number of boreholes were made and many samples were taken for laboratory testing there are still some depth intervals without data as seen in Fig. 3a&b. Field tests like vane shear and CPT tests could compensate this, giving a more complete characterization of the whole soil profile. More than that, the field test data could be used to infer a number of important parameters, including OCR, the cone factor and the undrained shear strength.

5) The correlations between compression index and the other parameters for both clays were found as follows:

Cc= 0.005*Wn
Cc=0.181*e_0
Cs= 0.227*Cc

6) Relationship between net cone resistance and undrained shear strength were found as follows:

q_t-σ_v'=11.77*Su *for UU and UC tests and the upper clay*

q_t-σ_v'=12.39*Su *for vane shear test and the upper clay*

q_t-σ_v'=7.80*Su *for UU and UC tests and the lower clay*

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