USE OF RI-CONE PENETROMETER IN FOUNDATION ENGINEERING

UTILISATION DE RI-PENETROMETRE A CONE POUR LA FONDATION

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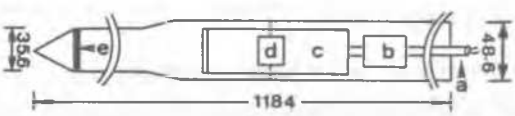
SYNOPSIS : As the suitable sites for the construction is getting scarer, more and more construction projects are moving off-shore, where it is not only difficult to obtain good samples but at times they are prohibitively expensive. The authors have designed and developed RI-Cone Penetrometers and have used them successfully at different sites. These sites are mainly characterized by marine clay deposits or reclaimed sites along the Osaka Bay. Results from one of the sites (Kinkai Bay) have been introduced. It has been noted that the presence of the halide contents can significantly underestimate the moisture content of the soil. FE analysis was carried out to simulate the actual construction sequence. The void ratio (e) obtained using the water content profile obtained using NM-Cone is in good accordance with those obtained through FE analysis.

INTRODUCTION

For many geotechnical analysis, it is important to know the density (ρ_t) of the soil, the water content (w_n) as well the void-ratio (e), among other soil parameters. Though the boring provides many of the useful information such as N-value, density, water content, soil stratigraphy etc. Yet removing the soil from the depth of the earth put some doubts on the reliability of the obtained data due to stress relief and other disturbances. γ-rays have been used in the geotechnical engineering for a number of years to measure the density of the soil, and neutron have been used to measure the water content of the material. In the beginning it was difficult to use these instruments due to the bulkiness of the instrumentation as well as the associated electronics. With the advances in the electronics, it is now possible to miniaturize various electronic components, which enables us to take the personal computer in the field to have data logged. Taking advantages of these advances in the associated technologies, the authors have designed and developed two different kinds of Radio-Isotope (RI) Cone Penetrometers (Neutron-moisture (NM-) and Nuclear-density (ND-) cone penetrometers). In what follows, we have very briefly described the principles behind the working and the construction of these cone penetrometers. These cone penetrometers have been used successfully at various sites. Test results from Kinkai Bay have been introduced. Numerical analysis in terms of elasto-viscoplastic finite element method have been carried out to estimate the amount of settlement and the void ratio of the marine clay deposit. Numerically calculated results have been compared with the results obtained through the RI-cones. It has been shown that both results are in good agreement. It has also been shown that the results obtained through these RI cone penetrometers can be used effectively in assessing the settlement of the clay deposit.



a: cable leading to data collection system; b: pre-amplifier; c: He^3-filled proportional tube; d: Cf^{252} fast neutron source; e: porous ceramic filter (all dimensions in mm)

Fig. 1. Schematic diagramme of NM-Cone.

DESCRIPTION OF RI-CONE PENETROMETERS

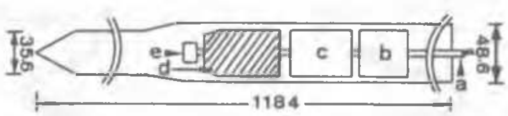
Fig. 1 and Fig. 2 show the schematic diagramme of the NM- and ND-Cones, for detailed description see Shibata et. al., (1991, 1992). Basic features of these two cone penetrometers can be summarized as follows:

NM-Cone Penetrometer: It makes use of the fact that the neutron are slowed down in the presence of the hydrogen and this very fact can be used to measure the water content of the soil if all the hydrogen present in the soil is in the form of water. NM-cone uses Cf^{252} as the fission source of the neutron having half life of 2.65years. The detector used in the cone penetrometer is the He^3-filled proportional tube.

ND-Cone Penetrometer: Principle of using gamma-ray to measure the density of the soil is well established. Gamma-ray interacts with the soil predominantly in three ways. All the interaction is dependent on the energy level of the gamma-ray. They are at low energy(< 600keV) photoelectric absorption, between 600keV to 1.2MeV it is Compton scattering and above 1.2MeV it is electron-positron pair production. If the detector is so designed that it measures only the gamma-photonsbetween 600keV and 1.2MeV then the incoming gamma-photons are only function of the density of the material and it can be given by the following equation:

$$I = I_0 \exp(-\mu_m \rho_t x) \tag{1}$$

where, I_0: incident radiation intensity; I: transmitted radiation intensity; μ_m: total mass absorption coefficient; ρ_t: density of the absorbing material; and x: thickness of the absorber. The gamma source used for the ND-Cone is Cs^{137} and the detector is photomultiplier tube.



a: cable leading to the data collection system; b: pre-amplifier; c: photomultiplier tube; d: lead (Pb) shield; e: Cs^{137} gamma-ray source (all dimensions in mm)

Fig. 2. Schematic diagramme of ND-Cone.

RI-CONE PENETRATION TEST AT KINAKI BAY SITE

Kinkai Bay is located in the south west of Honshu island of Japan facing inland Seto sea. Fig. 3 shows the plan view of the Kinkai bay. This area has traditionally been used in making salt from the sea water. Here the deposit is rather uniform, basically made up of about 29m thick bed of clay, which is underlain by sandy deposit. Embankment was constructed in 1950's. The authors have carried out penetration testing using the above mentioned RI-cone penetrometers at the point in shown

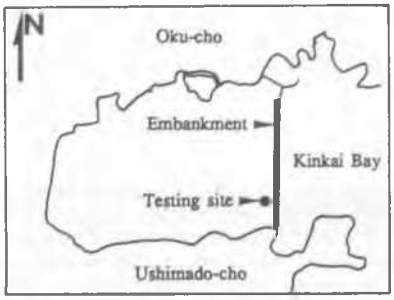


Fig. 3. Plan view of Kinkai Bay (the testing site).

in Fig. 3. Fig. 4a to Fig. 4d show the q_t, u, w_n and ρ_t, profiles at one of the testing sites at the Kinkai Bay area. It is not very difficult to observe the stratigraphy of the area. Corrected penetration resistance (q_t) increases monotonically with the depth upto a depth of 27m and then increases rapidly. The corresponding decrease in the pore pressure can

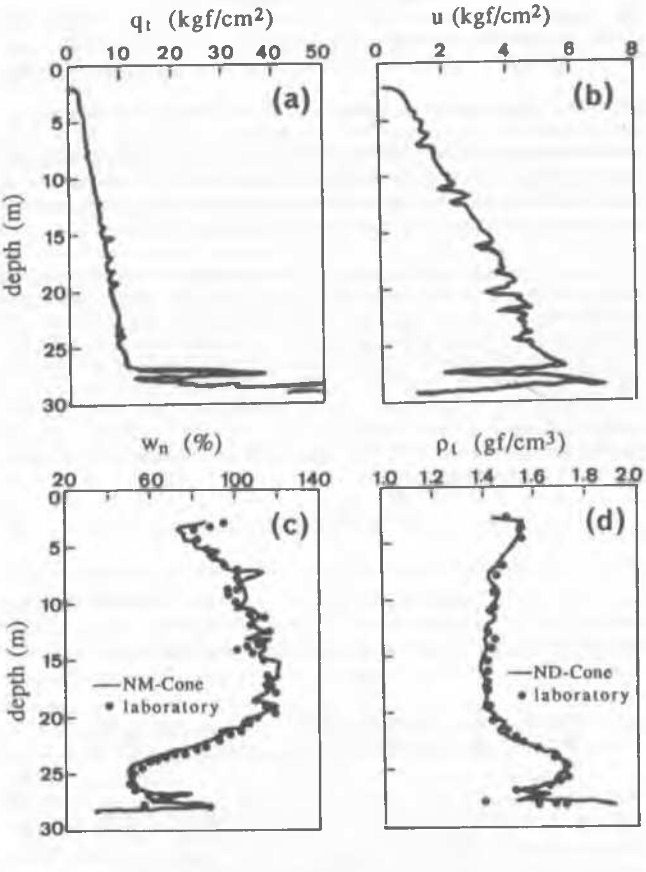


Fig. 4. Typical results obtained through NM-Cone and ND-Cone at Kinkai Bay (1kgf/cm^2 = 0.1MPa).

be visualized through the u profile (Fig. 4b). Also plotted are the natural water content (w_n) (Fig. 4c) profile measured using NM-Cone and the density (ρ_t) profile (Fig. 4d) measured using ND-Cone. Fig. 4c and Fig. 4d also show the water content (w_n) and density (ρ_t) profile as obtained from the laboratory testing on the obtained samples. The plot of sleeve friction (fs) and the fs/q_c ratio has been omitted from the text as the values were very close to zero. Shibata et. al. (1992) noted that the presence of the halides in the soil affects the moisture measurement, also as mentioned above that this area has traditionally been used in making salt, it was decided to collect the samples to carry out the halide analysis. Fig. 5 shows the distribution of the halide content with depth. Fig. 6 shows the reduction in count ratio with the increase in the halide content as carried out in the laboratory. It may be seen that with the 2% concentration of halide ion reduces the count ratio by almost 15%. The water content (w_n) profile shown in the Fig. 4c is the corrected profile, i.e., the effect of the chloride presence in the soil has been accounted for. Though some scatter can be seen which may due to the presence of bound water in the lattice space of the clay, nonetheless, a very good agreement can be seen between measured and those obtained in the laboratory.

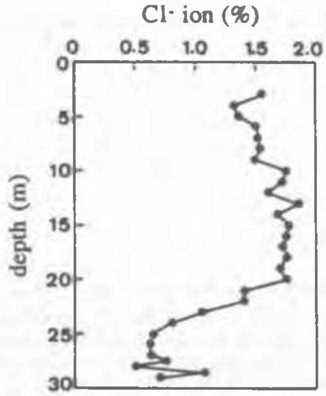


Fig. 5. Distribution of halide content with depth at Kinkai Bay site.

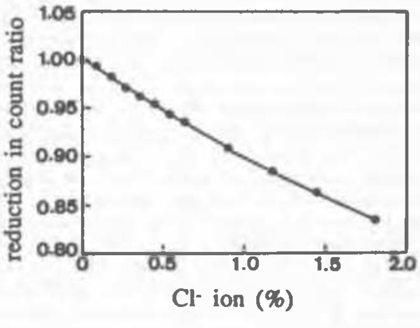


Fig. 6. Reduction in count rate ratio of NM-Cone as determined in the laboratory.

ASSESSMENT OF SETTLEMENT DUE TO EMBANKMENT CONSTRUCTION

Elasto-Viscoplastic Finite Element Analysis

Sekiguchi (1977) proposed elasto-viscoplastic model and alongwith his coworkers he modified it to plain-strain form (Sekiguchi et al., 1982). The viscoplastic flow rule for the plain-strain model can be expressed as

$$\dot{\varepsilon}^{p}_{ij} = \Lambda \frac{\partial F}{\partial \sigma_{ij}} \qquad (2)$$

where, F is the viscoplastic potential and Λ is the proportional constant. The viscoplastic potential F is defined as:

$$F = \alpha \, . \ln\left[1 + \frac{\dot{v}_0 . t}{\alpha} \exp(\frac{f}{\alpha})\right] = v^p \qquad (3)$$

where, α is the secondary compression index; $\dot{v}_0$ is the reference volume strain rate; f is the function in terms of the effective stress and the v^p is the viscoplastic volumetric strain. Mimura and Sekiguchi (1986) have discussed the plain strain model the finite element procedures in used here in details.

The soil profile for the FE analysis is determined on the basis of the soil boring log as well as the soil parameters measured by the RI-cone penetrometers described above.The initial conditions for the FE analysis is determined on the basis of the results obtained from the RI-cone penetration testing. The initial void ratio (e_0) is determined from the NM-Cone, through water content profile and the initial overburden pressure (σ'_{v0}) is determined through ND-Cone from the density (ρ_t) profile. Fig. 7 shows mesh prepared for the FE analysis together with the location of the RI-cone penetrometer testing as well as the region considered for comparison of results. The region selected for the FE analysis is 200 m long and 31 m deep into the clay layer. The foundation assumed for the FE analysis is divided into 8 layers. Below the embankment 12m deep sand drain piles were installed in triangular formation (Okumura et al., 1979). Finite elements corresponding to actual construction process are generated, which resulted into 362 nodes and 316 elements for the partial cross-section of the Kinkai Bay. Soil parameters were determined by the procedure outlined by Mimura et al. (1990).

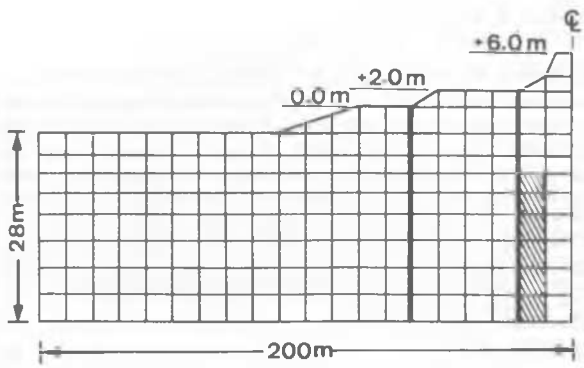


Fig. 7. FE analysis mesh for Kinkai Bay. The bold lines are the testing sites

RI-Cone Based Assessment and Discussion

The corrected cone resistance (q_t) profile of both the plain site and the embankment shoulder site is shown in Fig. 8. As may be seen the embankment loading does not increase the value of the q_t significantly. Here the increase in the q_t is investigated using the vertical stress profile obtained by FE analysis. The relation between the corrected cone penetration resisance (q_t) and the undrained shear strength (c_u) can be expressed as follows:

$$c_u = \frac{q_t - \sigma_v}{N_{kt}} \qquad (4)$$

where, σ_v is the total vertical stress and the N_{kt} is the cone factor. The cone factor (N_{kt}) is consistent for the clay and depends on the plasticity index (I_p). For the present discussion related to Kinkai clay two assumptions are made, i.e., the cone factor (N_{kt}) is equal to 15 (Robertson and Campanella, 1986), and $c_u/p = 0.3$.

According to the FE analysis, vertical stress increase increment ($\Delta\sigma_v$) varies between 2.2 and 2.66 tf/m^2 at the points where the RI-Cone

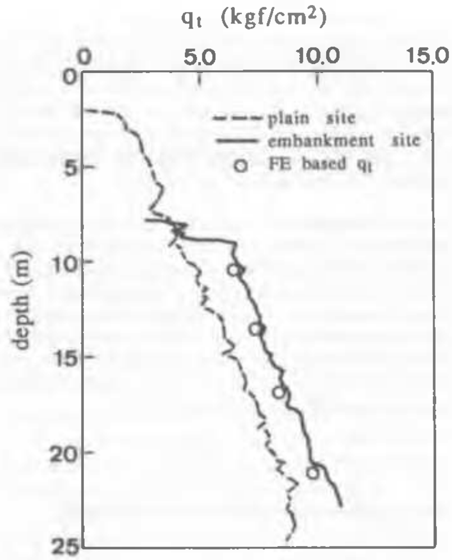


Fig. 8. Comparison of q_t at the plain site and the embankment site. Also shown are the plots of FE based q_t.

penetration tests were performed. The corrected cone penetration resistance at the embankment (q_{temb}) can be evaluated by solving the equation (4), by substituting the stress conditions obtained from FE analysis. FE based q_t points are also plotted in Fig. 8, and a close agreement can be seen between the measured and the FE based q_t. As seen here, the corrected cone resistance is not very sensitive to the embakment loading, that is to say, to the settlement of the clay. As discussed above, the RI-Cone is capable of measuring various soil parameters, the authors have tried to use those parameters to analyse the settlement of the clay foundation subjected to embankment loading.

Fig. 9 shows change in values of void ratio (e) with depth as derived from the natural water content of the soil as measured by using NM-Cone at the embankment shoulder as well as at the plain site. At the embankment shoulder upto a depth of 10m, a casing was installed as the original clay was completely replaced by pebbles and gravels during the

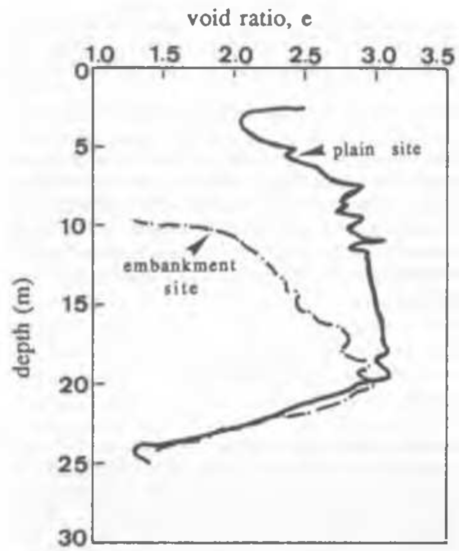


Fig. 9. Change in void ratio (e) as determined through NM-Cone at the plain site and at the embankment shoulder.

construction of the embankment. The void ratio (e) as shown in Fig. 8 was determined through the water content profile measured through NM-Cone (Fig. 4c), assuming the saturation (Sr) of the soil to be 100% and the specific gravity (Gs) profile along the depth of the penetration depth. Also the change in void ratio (e) profile at the embankment shoulder shows a reduction in the void ratio (e) due to the consolidation of the clay deposit brought in by the embankment construction. It is also observed that the void ratio (e) for both the sites remain unaffected by the construction activity beyond the depth of 20m.

Fig. 10 shows the comparison of the void ratio (e) as calculated from the FE analysis as well as the void ratio (e) derived from the water content measurement at the plain site. This plain site is about 70m inland or away from the embankment. A very good agreement is seen between the calculated and measured void ratio (e) at this site, which implies that construction of the embankment has no effect on the void ratio at this site. As no effect of the embankment construction is noticed at this site, the authors have taken this value of void ratio (e) as the initial void ratio (e_0) in the formulation of the FE analysis.

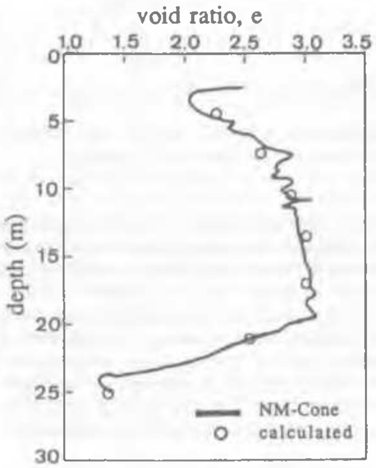


Fig. 10. Comparison of void ratio (e) between calculated and determined through NM-Cone at the plain site.

Fig. 11 shows the comparison of void ratio (e) obtained from the FE analysis for the clay layer, the hatched area in Fig. 7 and the void ratio (e) obtained from the water content as obtained through the NM-Cone for the embankment shoulder site. A very good agreement is observed between the calculated profile of the void ratio (e) and the void ratio (e) obtained from the water content measured value through the NM-Cone.

As noted earlier, the clay deposit upto a depth of 10m were removed during the construction of the embankment and were replaced with pebbles and gravels. Therefore, the total amount of settlement the underlain deposit may undergo is required for the safe operation of the embankment. The total amount of compression or the settlement of the clay deposit can be estimated by measuring the change in the void ratio (e), through the change in water content as measured through the NM-Cone. The total compression by NM-Cone, s_{cone}, can be expressed by the following equation:

$$s_{cone} = \int_0^z \left(\frac{e_0 - e_f}{1 + e_0} \right) dz \tag{5}$$

where, z is the total depth under consideration, e_0 is the initial void ratio and e_f is the final void ratio measured through the NM-Cone. In this particular case the e_0 corresponds to the values obtained at the plain site and the e_f is the value obtained at the embankment shoulder. Both these values are continuously measured as can be seen from the Fig. 4c. s_{cone} was found to be 1.26m and the total settlement of the corresponding point from the FE analysis, s_{FEM} was calculated to be 1.18m. Both the results are in good accordance. It can be said that the data obtained from

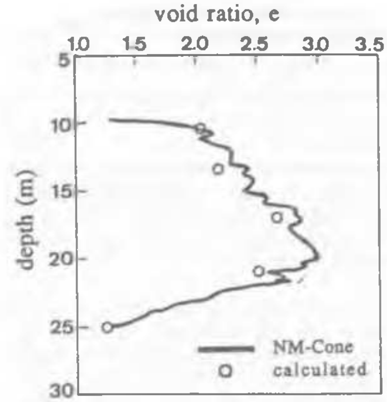


Fig. 11. Comparison of void ratio (e) between calculated and determined through NM-Cone at the embankment shoulder.

the RI-Cone testing can be used effectively in assessing the settlement of the clay deposits. One of the major advantages RI-Cone offers over conventional cone penetrometers is that, the e-profile, through the water content profile, can be monitored continuously along with other profiles without sampling and laboratory testing.

CONCLUSION

The authors have briefly described the construction of the RI-cone penetrometer. It has been shown that these RI-cone penetrometers are versatile instruments and can measure continuously various soil parameters such as ρ_t or w_n with depth. The data offer the initial condition of the foundation, σ'_{v0} and e_0 for the FE analysis as well as other cone parameters. Furthermore, using these RI-Cones at the construction sites at various stages of construction, settlement of the foundation can be assessed by monitoring the change in the void ratio (e) with the advancement of construction. The validity of RI-Cone based assessment is ascertained through the comparison with the correspondingly calculated performance of the elasto-viscoplastic FE analysis. It is suggested that as more experience is gained through these RI-cone penetrometer, these should become an indispensable tool for the subsurface investigation.

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