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Assessment of the compression characteristics of cohesive deposits from construction monitoring data, laboratory data and CPTu testing

Évaluation des caractéristiques de compression des dépôts cohésifs à partir des données de surveillance de la construction, des données de laboratoire et des tests de CPTu

P. Casey
Arup, Belfast, Northern Ireland

P. Kissane
Roughan & O'Donovan, Dublin, Ireland

M. Kileen
Arup, Dublin, Ireland

M. Long
University College Dublin, Dublin, Ireland

D. Ward
In Situ Site Investigation, St Leonards on Sea, UK

R. McWha
Farrans Construction, Belfast, UK

ABSTRACT: The challenge of developing engineering solutions around the consolidation of cohesive deposits is frequently encountered on infrastructure projects, and this can be a particular concern where the depth of the soil deposit reaches 20m – 40m or more. The rate of consolidation can be a significant factor in identifying the most appropriate form of ground improvement and the accurate prediction of settlement rate can result in significant savings over the course of a project. This paper sets out the assessment of the in situ settlement characteristics for cohesive alluvial deposits using settlement monitoring data gathered during the construction of earthworks for a new highway scheme in Northern Ireland. The field coefficient of consolidation calculated using the site monitoring data is evaluated against equivalent data from dissipation tests using standard piezocone testing and laboratory testing.

RÉSUMÉ: Le défi de développer des solutions d'ingénierie autour de la consolidation de dépôts cohérents est souvent rencontré dans les projets d'infrastructure, et cela peut être particulièrement préoccupant lorsque la profondeur du dépôt de sol atteint 20m - 40m ou plus. Le taux de consolidation peut être un facteur important dans l'identification de la forme la plus appropriée d'amélioration du sol et la prévision précise du taux de

tassement peut entraîner des économies significatives au cours d'un projet. Ce document présente l'évaluation des caractéristiques de tassement in situ, en particulier le coefficient de consolidation des gisements alluviaux cohérents, à l'aide des données de surveillance des tassements recueillies lors de la construction de terrassements pour un nouveau projet d'autoroute en Irlande du Nord. Le coefficient de consolidation sur le terrain, calculé à l'aide des données de surveillance du site, est évalué par rapport à des valeurs équivalentes obtenues lors d'essais de dissipation effectués à l'aide d'essais au piézone standard et d'essais en laboratoire.

Keywords: settlement; coefficient of consolidation; CPTu;

1 INTRODUCTION

The A6 Randalstown to Castledawson improvement scheme in Northern Ireland involves the design and construction of 14 km of new dual carriageway and associated primary structures through an area with complex geological conditions. The ground conditions consist of variable alluvial silts, clays and sands, overlying laminated lacustrine clays over glacial till. Approximately 3 km of the new dual carriageway is located in an area between the River Bann and Moyola River, north of Lough Neagh where the ground conditions consist of loose sands over soft compressible silts and clays and older glacial till deposits at depths of approximately 20 to 40m below ground level (m BGL). Ground investigation carried out for the project recorded the presence of a complex sequence of sands, silty sands and clays and silty clays overlying stiff glacial deposits. The ground investigation works included boreholes with piston and thin walled tube (UT) sampling along with cone penetration testing (CPTu).

Arup and Roughan O'Donovan (AROD JV) as designers for the Graham Farrans construction joint venture have developed the detailed design for the scheme, including the geotechnical design for the earthworks and structures in the soft ground areas.

Within the area considered for this paper the new mainline dual carriageway is on a low embankment of 2 to 3m height above the surrounding ground level in order to stay above flood levels. This results in the approach embankments to the overbridges in this area rising to 9m above existing ground levels. Long term performance criteria for the completed road required careful consideration of consolidation settlements and rate of settlement, creep and construction programme to be built into the design. The resulting design includes preloading and vertical drains for the structure approach embankments. This paper presents details around Hillhead junction, located towards the western end of the soft ground area.

2 GROUND MODEL

The ground conditions in the area around Hillhead junction consist of shallow (<1.0m) peat deposits, overlying sands and silty sands to 6m BGL, over silty clay to 18m BGL, over glacial till.

Ground investigation carried out for the scheme included cable percussive boreholes from which piston and thin walled samples were recovered for laboratory testing. Static cone penetration tests with measurement of pore water pressure (CPTu's) were also carried out, with a limited number of dissipation tests. Pore pressure was measured in the shoulder position (u_2).

Groundwater was typically recorded at 1 – 2m BGL and measurements suggest that a hydrostatic profile is present.

The upper alluvial soils (0 – 6m BGL) are thought to originate from flood events associated with the Moyola River and are described as sand and silty sand, over silts and clays down to c. 18 mBGL. The sands from 0 – 6m BGL show a typical natural water content (w_n) of 28%. Below 6m BGL the soils grade as silty clay and show a typical w_n of 33% and plot as clays of low to intermediate plasticity.

The deeper silty clays are believed to have been laid down at the base of the glacial Lough Neagh. These soils have been described as laminated in some explorations with a thin coating of fine sand observed between clay layers and were recorded at depths of 13m BGL and are typically 4 – 5m thick. These soils show a typical w_n of 45% and plot as clays of high plasticity.

Glacial till was encountered below the lacustrine clay.

The distribution of moisture content and plasticity indices with depth is shown on Figure 1.

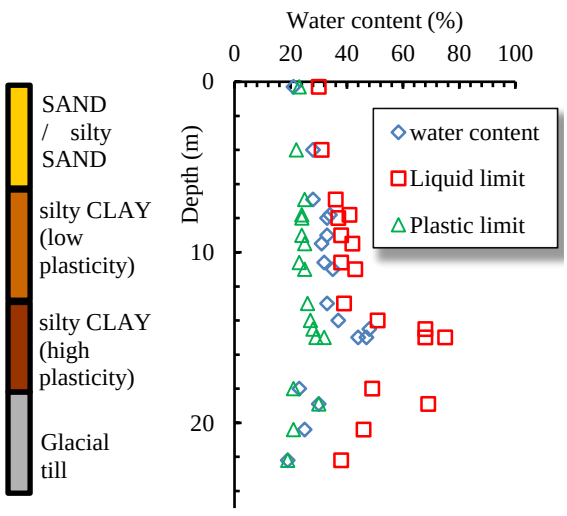

Figure 1. Soil indices and layer boundaries

Figure 2 shows the corrected cone resistance (q_t) and normalised friction ratio (F_r) for two typical CPTu's for the area. Figure 2 also shows the soil behaviour type based on I_c values (Robertson, 2009).

Assessment of the soil behaviour by plotting the pore pressure parameter (B_q) against friction ratio and cone resistance (q_t) was also considered. However, it was noted that some soil behaviour at shallow depths did not correspond with observations from pits, with some clay materials not being identified in the B_q - q_t plot. It is thought that filter desaturation, which can be a problem in intermediate soils such as silty sands and sandy silts, may have occured leading to errors in the pore pressure readings and hence B_q values.

The CPTu data suggest a greater sand content in the upper alluvium (down to c. 6m BGL) than was identified from the boreholes. The CPTu's and boreholes both identify the silty / clay mixture of the lower alluvial soils and the lacustrine clays.

The ground model used for the baseline design calculations consists of 6m of silty sand over 12m of silty clay / clayey silt over glacial till.

The quality of oedometer test results were assessed by determining Δe / e_0 to evaluate sample disturbance, Δe and e_0 being change in void ratio when loading to the vertical stress in situ (σ'_{v0}) and initial void ratio respectively (Lunne et al. 1997). Test results with Δe / $e_0 \leq$ 0.05 were given more weighting in the derivation of characteristic parameters.

Coefficient of consolidation (c_v) is one of the most difficult and, in the case of compression of soft ground, one of the most important parameters to determine. Various methods for deriving c_v were considered: directly from oedometer test results; from CPTu dissipation test results; and also from a combination of the individual component parts.

The c_v values from oedometer tests on good quality samples of silty clay shows results of 30 to 40 m^2 / year recorded at stresses around the in situ stress condition. A limited number of CPTu dissipation tests were successfully carried out within the fine-grained layers at depths of 8 to 16m BGL. These tests show derived c_h values of

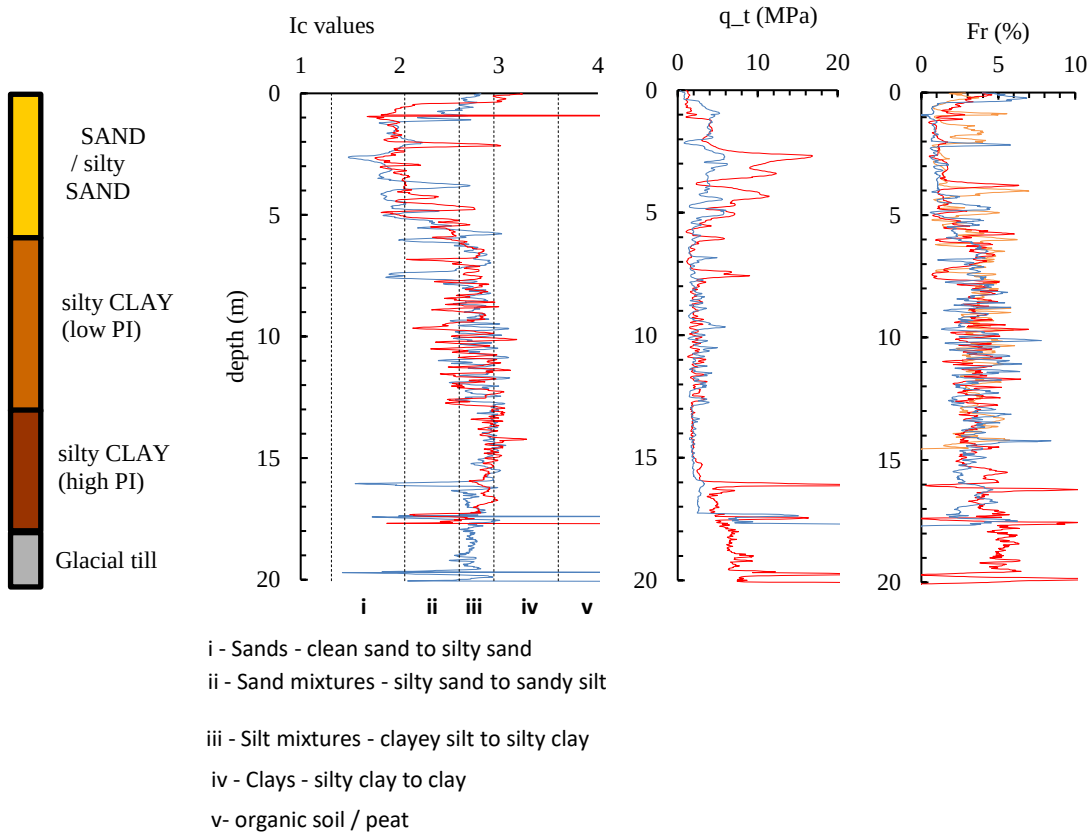

Figure 2. CPTu data and layer boundaries

300 to 1,300 m^2 / year (for rigidity index, I_r = 500) using the Teh and Houlsby (1991) method.

The c_v values derived from the permeability coefficient (k) and the coefficient of volume compressibility (m_v) show values of c. 10 m^2 / year in the high plasticiy silty clay and c. 60 m^2/year in the low plasticity silty clay.

The c_v value is also highly stress dependant and due to the large stress increase that will result from the construction of the embankments a value of 10 m^2 / year was used in the baseline design.

The relationship in Equation 1 was used to calculate the consolidation settlements (McCabe et al, 2014)

$$C_c / 1 + e_0 = 0.1905 (\ln(w_n) - 3.03) \qquad (1)$$

Where C_c is the compression index, e_0 is the initial void ratio, w_n is the natural water content (%). While the relationship in Equation 1 is intended for soils with w_n > 35% direct derivation of C_c from laboratory tests was not used due to the limited number of better quality oedometer samples.

In the absence of a large data set of better quality C_c values a conservative relationship of C_r equal to 15% C_c was used in the baseline calculations.

The compression of the upper sand layer was estimated using the Janbu (1963) procedure. For this calculation a typical w_n of the sand of 35% was used. A constrianed modulus, M (MPa) of 19MPa was derived for stress conditions below

the embankment and at mid-depth of the sand layer.

The baseline calculations referred to in this paper were built up using methods after Janbu and Terzaghi (1996) for immediate and consolidation settlements respectively.

The 1D constrained modulus, (M) was also derived for the cohesive soils directly from the CPTu data using I_c and q_t (Robertson, 2009). This method suggests an M of 20 - 25MPa for the silty clay from 6 – 18m BGL.

3 DESIGN DETAILS

3.1 Baseline Calculations

The road development authority set strict long term differential settlement requirements on the approach to primary structures in the contract documents for the A6RC scheme. The sections of the Hillhead embankments considered by this paper impose a stress increase of 162kN/m^2 on the ground. This results in c. 165mm of surface settlements using the combination of Janbu and Terzaghi methods discussed previously. The 165mm total settlement is made up of c. 100mm compression of the sand from 0 – 6mBGL and 65mm compression of the silty clay from 6 – 18mBGL.

Using the 1D constrained modulus from the CPTu data resulted c. 65mm compression of the silty clay from 6 – 18mBGL. No geological factor has been applied to this figure

The time for completion of 90% primary consolidation was estimated at 1 - 2 years for the baseline design with no ground improvement. The construction programme, consideration of creep and long term performance requirements for the completed road led to the inclusion of ground improvement measures into the design.

In order to achieve 90% consolidation within the construction programme and allow sufficient time for follow on construction works prefabricated vertical drains (PVDs) were used. The resulting accelerated consolidation also allowed the use of surcharging methods to be incorporated into the design to minimise the risks from long term creep. In the Hillhead area, PVDs were installed in a triangular pattern, with a spacing of 1.0 - 1.5m. PVDs were 100mm wide and 3mm thick, consisting of a three dimensional polypropylene core protected from clogging by a filter matting. The final spacing used in the design also took into consideration smear effects and limited experience of this technique in the local soil conditions.

Barron's Equation 1 (Barron, 1948) was used to determine the PVD spacing and resulting time to 90% consolidation of the improved ground. The baseline calculations predicted 90% consolidation would be reached 1 – 2 months after installation of PVDs and placment of the fill.

3.2 Instrumentation and Construction Details

Multipoint extensometers and settlement plates were constructed below and within the embankments to record the magnitude and rate of settlement in the ground in order to verify the design.

Works were undertaken on site over a six week period in August and September 2018. Figure 3 shows the rate of embankment construction and pore water pressure during this period.

3.3 Settlement Observations

Figure 4 and 5 show the development of surface settlements and settlement at depth during construction. Data are shown for surface settlement plates installed at the base of the embankment (HHN01 and HHN02) and for a borehole extensometer drilled below the embankment (EXT04).

Note that the extensometer data referenced in Figure 5 are from a replacement instrument drilled after the original instrument was damaged by the works. The data presented in Figure 5 are the raw data collected on site and

have not been corrected for the settlement which occurred previous to the installation of the replacement insrtument.

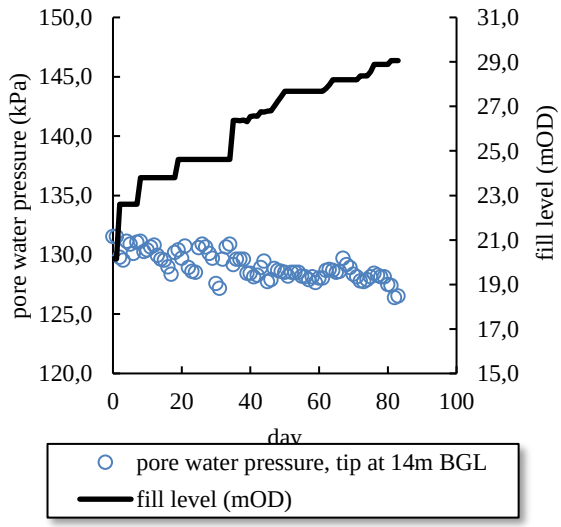

Figure 3. Embankment construction and pore pressure readings (at 14mBGL)

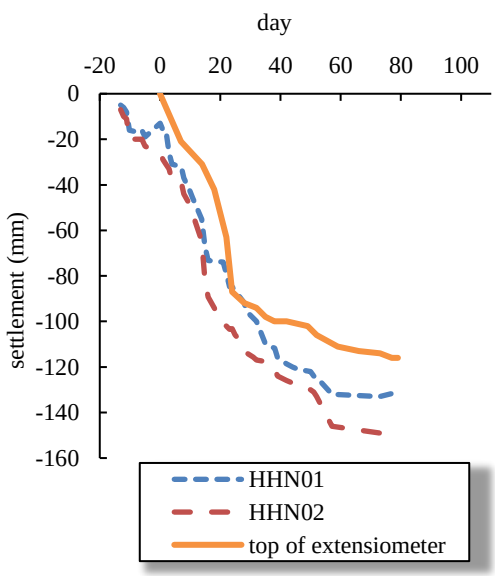

Figure 4. Settlement data

The magnitude, and rate, of settlement of the top plate in this replacement extensometer is consistent with data collected from the adjacent

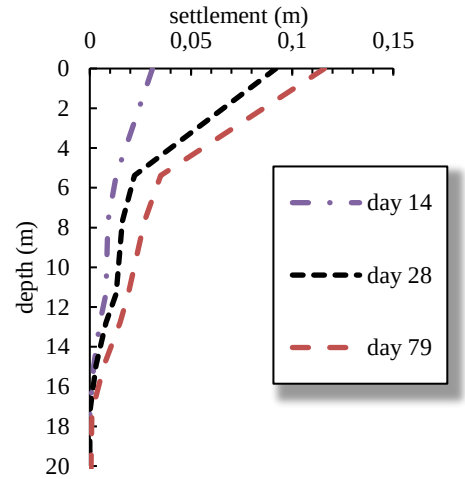

Figure 5. Extensometer data (EXT04)

surface settlement plates and is considered to be reliable.

Comparison to the adjacent surface settlement plates suggests that c. 25mm of surface movement has been omitted from extensometer data presented here.

The raw data from the extensometer shows c. 80mm of compression for the sand from 0 – 6m BGL and c. 35mm of compression for the silty clay from 6 – 18m BGL.

4 DISCUSSION

The baseline calculations predicted 165 cm of surface settlement for the embankments in this area. To date, the monitoring data shows c. 140 - 150 mm of settlement. Some further settlement is expected to be recorded, however, 140 – 150 mm is considered to be a reasonable estimate of the 90% consolidation at this time based on the shape of the settlement data versus time curves and the pore pressure behaviour (Figure 3).

Figure 6a shows the baseline calculated settlement profile over depth and the data from extensometer EXT04. It should be noted that the raw data for EXT04 are presented, and the actual surface movement, and hence also the sub-surface movement, is larger. The settlement profile calculated using the 1D constrained

modulus calculated directly from the CPTu data and the average surface settlement measured from surface settlement plates (blue diamond) are also included in Figure 6a.

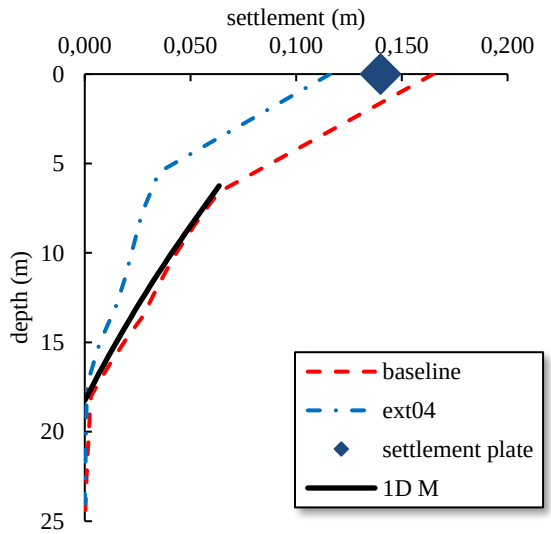

Figure 6a. Settlement over depth

Figure 6b shows the baseline calculation and the extensometer data for day 79 presented as vertical strain.

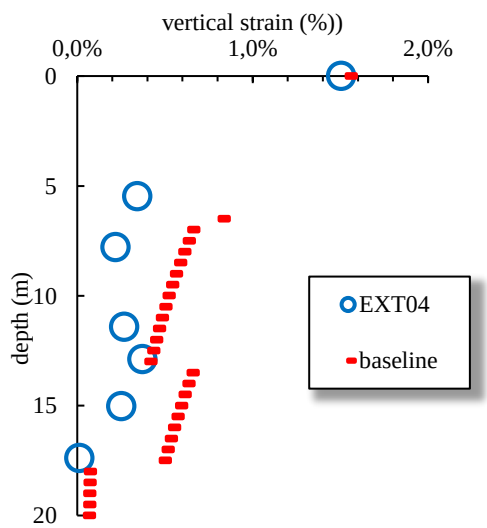

Figure 6b. Strain over depth

Vertical strain was calculated as the amount of settlement between two points divided by the distance between those two points. This plot also presents the baseline calculated settlement for the same chainage presented in a similar way.

While there will be some variation in the actual stratigraphy on the ground compared to the baseline calculation, some observations can be made from Figure 5 and the overall trend in Figures 6a and 6b.

The combination of Janbu and Terzaghi calculation methods appear to give a reasonable prediction for the overall surface settlement in this location, 140mm average measured in the field against 165mm by calculation.

Direct comparison of the sub-surface settlement against the calculations is difficult as the extensometer was installed after filling had commenced and the sub-surface distribution of the 25mm mentioned in Section 3.3 can only be speculatedguessed. However, examination of the settlements and strains at depth show that the field measurements of compression of the silty clay layers down to 18m BGL are c. 50% of what was calculated. The variation reduces to c. 25% below this.

The silty clay remains in the over-consolidated zone after application of the embankment load and thus it is suggested that the actual C_c in the ground is lower than has been derived from the w_n and used in the calculations. The 50% variation between 6 and 18m BGL may be partially explained by the natural water content for this layer being at the extremity for which Equation 1 was developed.

Figure 6a also shows that the settlements calculated for the silty clay below 6m BGL using the 1D constrained modulus closely matches the baseline calculation and thus also over-estimate the actual settlements by 25 – 50%. Literature suggests a geological factor of 0.7 – 1.0 should be applied to the settlements calculated using M (or 1 / m_v) (Tomlinson, 2001). Use of a factor in this range would greatly reduce the variation between the

calculated and measured sub-surface movements in the silty clay.

The data suggest that the compression of the sand layer from 0 – 6m BGL is reasonably well predicted using the Janbu method.

Review of the settlement data and the pore pressures below the embankment suggests that 90% consolidation will be reached in a matter of weeks post completion of fill placement. Back calculation of the rate of consolidation using Barron's Equation 1 suggests a bulk c_h of 30 m^2 / year for the in situ behaviour of the soil under the load from the embankments. The figure of $30m^2$/year is consistent with the c_h values recorded in the higher quality ($\Delta e / e_0 \leq 0.05$) oedometer tests on the silty clay suggesting the assessment of test results using the $\Delta e / e_0$ is appropriate for these soils.

The c_h values predicted from the CPTu dissipation tests using the method by Teh and Houlsby are an order of magnitude greater than the back calculated bulk value. This is consistent with findings by others (Mahmoodzadeh et al, 2014). Based upon this study it is suggested that derivation of c_v using the component parts can be used for practical purposes.

5 CONCLUSIONS

(i) Oedometer data on high quality samples can be used to produce reasonable predictions for both magnitude and rate of settlement in these soils,

(ii) c_v derived from k and m_v can give reasonable values when compared to the observed settlement rates

(iii) CPTu data can be used to give a reasonable prediction for the magnitude of settlement in the silty clays in this area,

(iv) Characterisation by CPTu has proven difficult in the upper silty sand and sandy silt and this may be due to filter desaturation in these intermediate soils

6 ACKNOWLEDGEMENTS

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