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The observational method *in* geotechnical engineering

tions. Such is the chaotic background of natural deposits that local variation is often unpredictable. For constructions in which elements feature repeatedly such as ground anchorages, piles, grout injections, etc., knowledge of ground properties has to be at most on a scale of 1 to 2 m to be confident of the outcome of predictions. The economics of site investigation do not usually allow this luxury.

Consequently there is a need to base designs on predictions and to be prepared to change the construction as work proceeds. With appropriate foresight this need not be disruptive, as some of the proper options will have been planned.

There is clearly a need for monitoring site operations to check initial design assumptions and to observe and define variations from them, and also to control action in face of continuous variations.

The initial statement that the observational method is inherent in geotechnical work follows from this situation because of the nature of the ground—the working material for geotechnics.

Observations are therefore a vital part of geotechnical site practice. They can take several forms. Almost anything can be used which throws light on the ground conditions or behaviour, even if only by calibration against a norm. Records of intermittent events or findings should be meticulous. Often their value is unknown until it is necessary to vary actions. Otherwise indirect observations via plant performance are useful. The reaction of machines to their work in the ground—power demand, rotation speed, advance rates, thrust or pulling forces, fluid flow rates and pressures—can all give indirect information.

Direct observations of the effects of geotechnical work can arise from deliberately instrumented ground. Tilting can be measured by spirit or electro-levels; deformation by surface levels or extensometers in the body of the soil; direct stress measurements might be taken or pore fluid pressures from piezometers and stand-pipes etc.

The important thing is that all such records and observations should be subject to continuous surveillance and display in order to detect change in real-time and so to allow a developing situation to be acted upon if necessary.

It should be clear that instruments used for observations must be appropriate for their task. Sensitivity to the range of change of the observation should be considered. Also, the precise location of an instrument to give signals appropriate to the intent is important. The numbers of instruments used and their reliability need to be thought about, as well as their type and the range at which they must be capable of detecting change. The influence of extraneous 'noise' on the observed results should be considered while at the same time being aware of electronic smoothing of signals which may eliminate important peak observations.

These practical considerations are very important if geotechnical engineering works are to be managed responsively in their application.

It is noted also that in most instances current commercial arrangements between clients, engineers, contractors and subcontractors do not fit comfortably with the concept of changing details as work progresses. The clients are inclined to be suspicious of the techniques of the engineering profession and of the integrity of contractors, and are disinclined to allow varying prices. Yet this is what the observational method demands in the interests of good practice to the benefit of the client and the community at large. More appropriate commercial arrangements are required if geotechnical practice is to serve the community well.

C. A. Raison, *Chief Engineer, Keller Foundations*

It appears from the symposium in print that the observational method is a technique to be used only for large projects under certain conditions. However, the approach has long been applied to piling and ground improvement works, although it has never been formally recognized as the observational method. This view can be substantiated by an example where the method was applied to ground improvement and piling works carried out at the British Gas North Morecambe Terminal site near Barrow-in-Furness.

The North Morecambe site comprised an area of derelict land containing old lagoons of very soft wet pulverized fuel ash. Typical ground conditions beneath the pulverized fuel ash lagoons are loose alluvial sands over glacial sands to 20 m or more below ground level.

On account of the critical nature of the site and its sensitive location, the plant has been designed to cater for the small but significant risk of earthquake. Feasibility studies suggested that under seismic loading the very loose granular soils would liquify to depths up to 20 m below ground level resulting in major problems for the foundation design.

The foundation solution put forward by Keller was to overcome the tendency for liquefaction using deep vibro densification techniques to increase the in-situ relative density of the sands. Short driven cast in place piles were then required to transfer structural loads through the soft pulverized fuel ash to the treated soils (see Fig. 7).

Many of the Papers contained in the symposium illustrate the observational method in the form of a flow diagram. These usually detail the form of the initial design, the type and scope of the observations necessary during the construc-

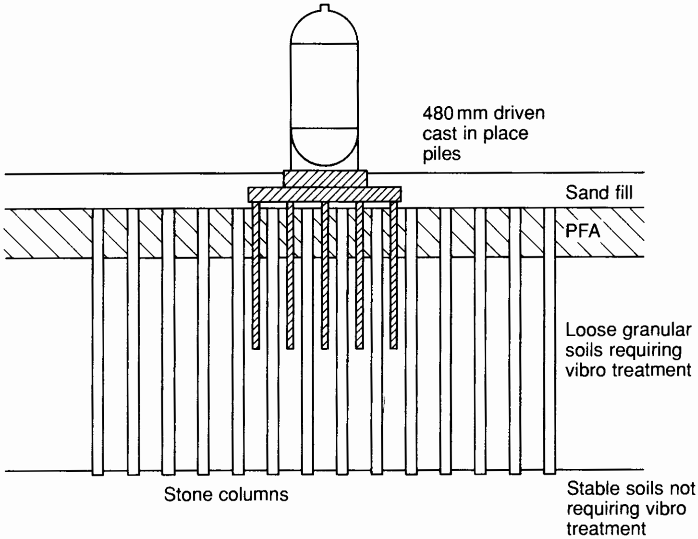

Fig. 7. Proposed foundation solution

tion period and trigger values relating to these observations that initiate any contingency measures. Table 2 gives a simplified flow chart as applied to the ground improvement works.

In the observational method the initial or preliminary design stage is based on probable conditions (assessed from site investigation). The choice of whether the most probable, more probable or moderately conservative condition should be assumed is open to discussion. This is governed by the quality of the site investigation, the degree of risk and commercial considerations. Worst case conditions are also considered to determine suitable contingency measures.

The vibro densification works were initially designed based on available site investigation data, mainly borehole Standard Penetration test (SPT) data, supplemented by cone penetration tests (CPT). Minimum CPT cone resistance (q_c) profiles were developed for the range of susceptible soil gradings as shown in Fig. 8. These values were to form the trigger values for control and monitoring of the works.

Before the main works, the preliminary design was to be verified by an extensive test programme to investigate variations in stone column layout and procedures. These would form the main observations, to be supplemented during the works by ongoing post treatment CPTs. Fig. 9 details the stone column layout of the main test area, and Fig. 10 shows the variable ground conditions typical for the site. Columns were arranged in various configurations with the achieved soil densification being measured at the mid points using CPTs. Vibro treatment was carried out up to 18 m below ground level.

Completion of the vibro trials enabled adjustments to be made to the proposed vibro layout within the seismic protection area. Site control was maintained using CPT measurements on a 25 m grid specific to the proposed structures. Fig. 11 shows a typical comparison between pre- and

Table 2. Observational method flow chart

Preliminary design	Based on site investigation Moderately conservative conditions Worst case behaviour	
Observations	Initial trials Ongoing measurements	
Trigger values	CPT q_c Drive blows Minimum length	Vibro Piling
Contingency measures	Closer centres Longer piles More piles	Vibro Piling

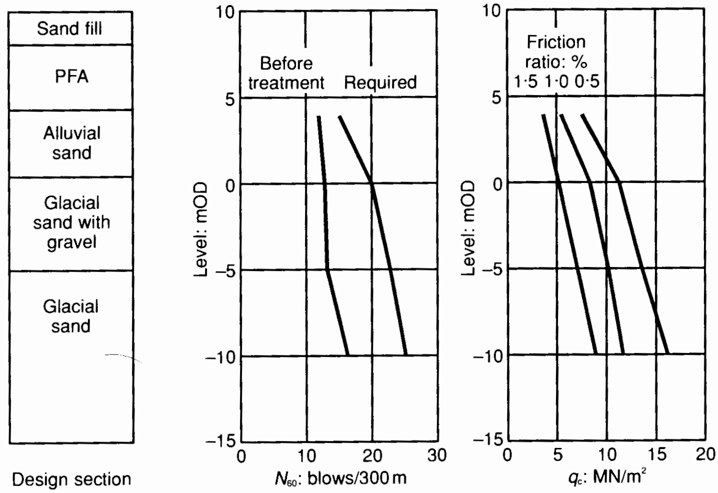

Fig. 8. Minimum SPT and CPT requirements

post-treatment CPT profiles, demonstrating the success of the densification works. CPTs formed the major observations on which the progress of the works was based, with achieved penetration of the vibrator acting as a secondary measure of local variations in the ground. Pile drive blows and achieved penetration provided additional data.

The last item shown on the observational method flow chart was the contingency measure should the trigger value be reached. Where the minimum CPT q_c was not achieved it was proposed to use additional stone columns installed at closer centres. However, in practice, only two limited areas needed further treatment, representing less than 0·5% of the treated soil.

The subsequent piling works carried out for the Barrow Project were also dealt with using a form of the observational method. After completion of the vibro trials, non-working trial piles were driven into the treated soils and into the adjacent non-treated area as shown on Fig. 9. Piles were driven to about 9 m in the treated zone and slightly deeper in the untreated area.

As with the vibro design, the preliminary pile design was based on the site investigation data.

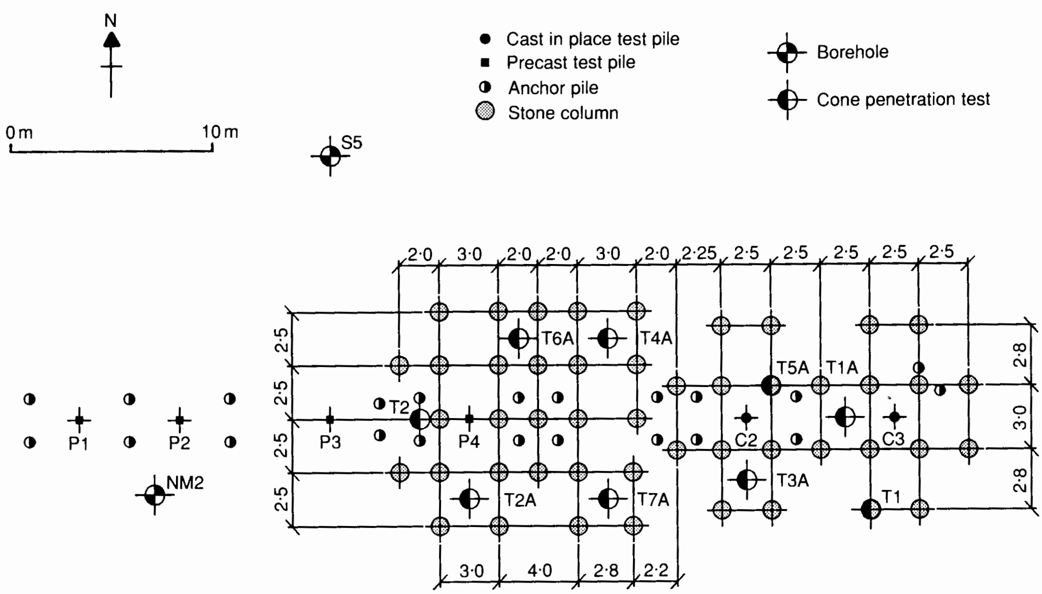

Fig. 9. Main test area showing stone column and pile layout

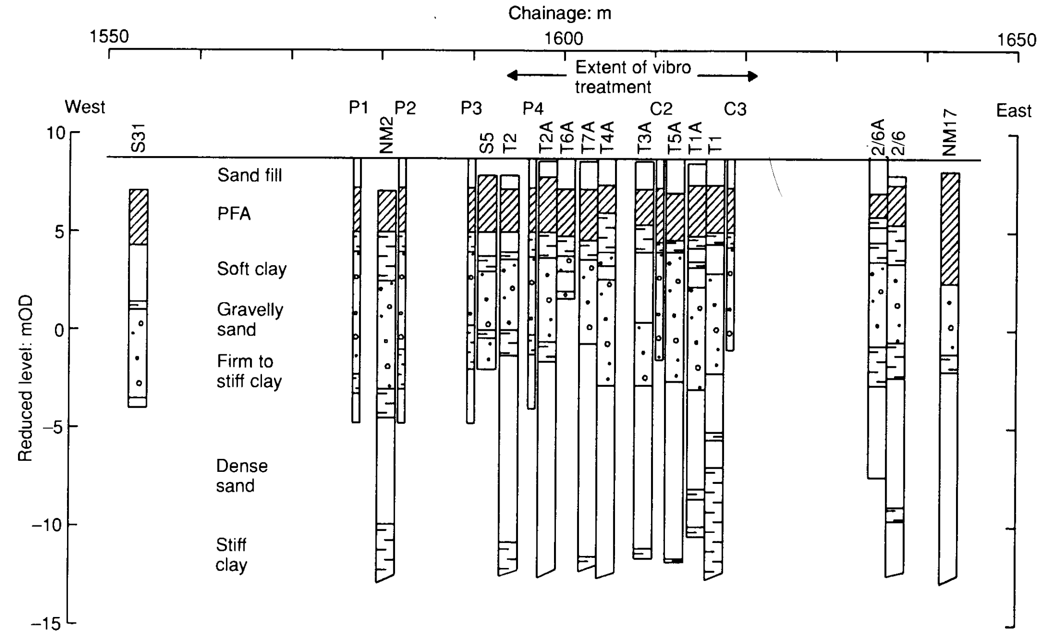

Fig. 10. Section through main test area showing ground conditions

The advance testing allowed drive trials to establish drive criteria, CASE and CAPWAC testing and static compression, tension and lateral load testing. Trigger levels were defined in terms of minimum pile lengths and drive blows. Piles were to be monitored during installation and a proportion load tested for verification. Contingency measures were also planned. These were to continue driving to achieve longer piles, or where necessary, to install additional piles. Deeper

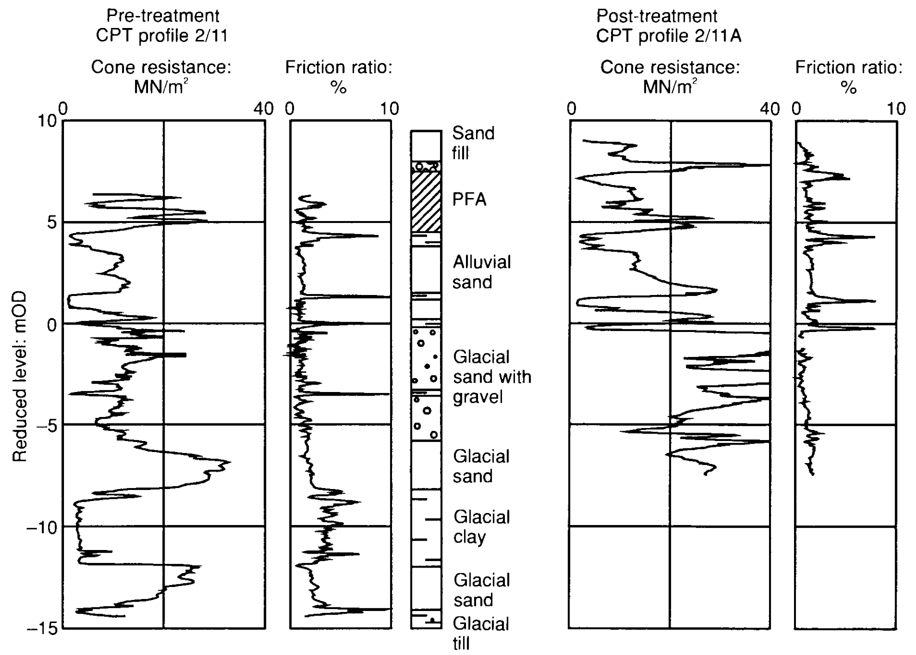

Fig. 11. Comparison between pre- and post-treatment CPTs

driving was required in some very localized areas but no additional piles were necessary.

This short note has demonstrated one example of how the observational method can be applied to other areas of geotechnical engineering. Continual feedback, particularly the depth of penetration of piling or the ground improvement vibrator, can provide intensive observations of the local variability of the ground. The use of computer monitoring and the generation of hard-copy printout enhances the ability to obtain these measurements. Correctly applied, these data can be used to control and modify the works to achieve the required solution.

D. Russell, *Geotechnical Engineer, Mott MacDonald*

The Whitewall Creek embankment forms part of the Medway Immersed Tube Tunnel's western approach structures. The embankment crosses a tidal creek over a thin layer of very soft alluvial clay. An observational approach was adopted for the construction stages and numerical analyses were undertaken to assist in establishing trigger levels for allowable deformations. Predictions from the numerical analyses are compared with field data and the general application of numerical analysis within the context of the observational method is discussed.

The Medway Immersed Tube Tunnel forms the third crossing of the Medway River near Chatham in North Kent. The tunnel was constructed as part of Kent County Council's Medway Towns Northern Relief Road. A design and construct contract for the tunnel and 1200 m of approach structures was awarded to a joint venture between Tamac and HBM. Mott MacDonald undertook the design for the joint venture and provided engineering support during construction. One challenging aspect of the approach structures was the economic construction of 200 m of embankment over Whitewall Creek.

Whitewall Creek is within the tidal zone of the Medway river and is covered by water twice a day. The clay at the base of the creek is very soft and has a maximum depth of approximately 6 m. The preliminary design calculations showed that the stability of the embankment which has a maximum height of around 7 m, including a 1 m surcharge, would be relatively low during the construction stages, therefore an observational approach was proposed and appropriate trigger levels, instrumentation and contingency measures established.

A critical measure of stability for such structures is the lateral movements at the toe. However, due to the limited depth of soft clay the typically used empirical relationships for the prediction of lateral movements were not considered appropriate. Therefore a series of numerical analyses was carried out to predict lateral movements and aid in the setting of trigger levels for acceptable construction deformations.

Geology of the Whitewall Creek area

Figure 12(a) shows the general geology of the Whitewall Creek area which comprises made ground over alluvial clay, an intermittent band of gravel and then chalk. The thickness of the alluvial clay increases towards the river and within the creek itself the made ground is absent. On account of the very soft nature of the clay it was only practical to carry out Macintosh probing to determine the clay depth supplemented by hand vane tests. The uncorrected peak and residual vane strength values are plotted in Figure 12(b) together with the proposed design strength profile. The small increase in strength over the normally consolidated line is thought to have been produced by previous ground water fluctuations and desiccation.

Preliminary design

Preliminary design parameters were assessed as follows: bulk density from laboratory test results; compression index from the averaged oedometer results and the empirical correlation with liquid limit $C_C = 0{\cdot}009$ (liquid limit 10%) (Terzaghi & Peck, 1948); the coefficient of consolidation from laboratory test result and correlated for similar clays with data given by Rowe (1972); the clay was assumed to be normally consolidated and the undrained shear strength ratio estimated from field data and empirical correlations (Muir-Wood, 1990). The parameters used in the preliminary design are shown in Table 3.

The preliminary design was based on a two-stage construction programme (Fig. 13). The bund and first lift stability were assessed using plasticity theory (Hird & Jewell, 1990). The increase in strength was calculated for the consolidation stages assuming one-dimensional compression and that the undrained shear strength ratio remained constant. The second lift was analysed using limit equilibrium methods.

Construction philosophy

The preliminary design was based on the most probable parameters and allowed for a factor of safety of 1·3. However, given the limited amount of site investigation within the creek it was decided at an early stage in the design to use an observational approach for each of the two lift stages and that the key measurement would be the lateral movement at the toe. Typically, the amount of lateral movement would be assessed