Integrating finite element and load-transfer analyses in modelling the effects of dewatering on pile settlement behaviour

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Abstract: A method of analysis is developed by integrating finite element (FE) and load-transfer analyses to predict the negative shaft resistance and settlement of piles due to ground water lowering. A program is written in MATLAB for linking the FE results of ground movements with the input interface system of a new pile load–settlement analysis program (named "PILESET") developed by the author. PILESET is specially designed to allow automatic input of electronic site investigation data, although manual input of laboratory soil test data is also possible. Using PILESET, custom-defined load-transfer relationships can be either input manually or calculated internally by PILESET based on the input data from in situ or laboratory soil tests. To demonstrate the validity of the suggested analysis procedure, a case record is analyzed where sump pumping was to be carried out underneath a deep basement situated close to an existing building supported on 15 m long piles. Based on assumed steady-state flow conditions, ground settlements are calculated using FE analysis and used with site investigation data to predict the negative shaft resistance and settlement induced in the piles. The results are found to agree well with field measurements.

Key words: ground water lowering, pile settlement, load-transfer analysis, finite element analysis, software development.

Résumé : Une méthode d'analyse est développée par l'intégration des analyses par éléments finis (EF) et du transfert de charge dans le but de prédire la résistance négative de l'arbre et le tassement des pieux dus à l'abaissement de la nappe phréatique. Un programme est écrit avec MATLAB pour relier les résultats d'EF du mouvement du sol au système d'entrée de données d'un nouveau programme d'analyse de charge-tassement de pieux, développé par l'auteur (appelé PILESET). PILESET est conçu spécialement pour permettre l'entrée automatique de données électroniques d'investigation de sites, et l'entrée manuelle des données d'essais de laboratoire est aussi possible. Grâce à PILESET, des relations de charge-transfert définies pour chaque cas peuvent être entrées manuellement ou calculées par PILESET à partir des données d'essais de sols in situ ou de laboratoire. Afin de démontrer la validité de la procédure d'analyse suggérée, une étude de cas est analysée dans laquelle du pompage devait être utilisé sous un sous-sol profond, situé près d'un bâtiment existant supporté par des pieux de 15 m de long. Les conditions d'écoulement étant assumées en régime permanent, les tassements du sol sont calculés à l'aide de l'analyse par EF et sont utilisés en conjonction avec les données d'investigation de site pour prédire la résistance négative de l'arbre et les tassements induits dans les pieux. Les résultats correspondent bien avec les mesures de terrain.

Mots-clés : abaissement de la nappe phréatique, tassement des pieux, analyse de transfert de charge, analyse par éléments finis, développement de logiciel.

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Introduction

Geotechnical engineers are often faced with problems of substructure instability and foundation settlement caused by ground water level changes, which may be either intentional or unintentional. A simple and commonly used method for estimating settlements caused by ground water lowering involves calculation of the drawdown curve using a simple analytical – semi-analytical method, such as that suggested by Preene et al. (2000). This method may be applied to a long dewatering system such as the one illustrated in Fig. 1, whereby lines of closely spaced wells are installed on both sides of an excavation of assumed infinite length, so that only plane flow to the sides is considered while radial flow to the ends is ignored. To calculate and plot the drawdown curve, it is first necessary to estimate the influence distance, L_o (see Fig. 1), defining the distance (from well line) beyond which the ground water level is no longer affected by the well pumping.

Preene et al. (2000) stated that the drawdown due to a pumped well system installed directly into a low-permeability soil is assumed to extend laterally from a line of wells at a rate dependent on the depressurisation behaviour of the soil. On this basis, consolidation theory may be applied to the problem as, owing to capillary effects, an unconfined aquifer comprising a low permeability soil may remain saturated, implying that any drainage of pore water results from a change in pore volume as the soil consolidates. This is a reasonable

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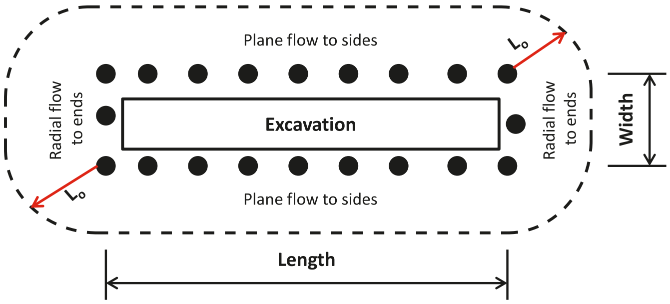
Fig. 1. Influence distance in plane flow to long excavation (radial flow neglected).



assumption provided that the drawdown of the water table is not too large. Thus, the progress of the drawdown curve with time may be estimated based on representation by successive nondimensional curves (known as isochrones) relating drawdown s to distance from the line of wells at time, t, since the commencement of pumping. Assuming that the drainage flow takes place in the horizontal direction only, Preene et al. (2000) suggested that the drawdown, s, at any horizontal distance x from the well line may be represented by a single parabola in terms of the following four parameters: (*i*) x, (*ii*) L_o, (*iii*) λ, and (*iv*) s_o, where λ is a curve-fitting constant and s_o is the drawdown at the well line position (i.e., at $x = 0$). Figure 2 shows the relationship suggested by Preene et al. (2000), which can be expressed as follows:

$$[1] \qquad \frac{s}{s_o} = \lambda\left(1 - \frac{x}{L_o}\right)^2$$

By digitizing the curve in Fig. 2 and performing parabolic regression, the author has calculated the constant λ to be 0.9876. Preene et al. (2000) suggested that the influence distance can be estimated using eq. [2], with SI units used throughout

$$[2] \qquad L_o = \sqrt{12c_{hv}t}$$

where c_{hv} is the coefficient of consolidation for vertical compression of soil under horizontal drainage $(= k_h E'_o/\gamma_w)$; k_h is the coefficient of permeability, along the horizontal direction, of the stratum through which the drainage flow occurs; E'_o is the stiffness of the stratum in one-dimensional compression; γ_w is the unit weight of water, and t is the elapsed time since commencement of pumping. Equations [1] and [2] may then be used to predict the decrease in the water table at any location defined by distance x from the line of pumped wells. This allows calculation of the effective stress change, $\Delta\sigma'_v$, in the soil at any depth and distance x from the line of pumped wells. For a given stratum (say the ith stratum) of thickness D_i, stiffness E'_{oi}, and average vertical stress change $\Delta\sigma'_{oi}$, the average compression ΔD_i due to ground water lowering is given by the following equation:

$$[3] \qquad \Delta D_i = D_i\left(\frac{\Delta\sigma'_{vi}}{E_{oi}}\right)$$

The limitations of the Preene et al. (2000) method outlined above are as follows:

1. It is not applicable when the ground water flows through multiple strata with different properties (especially k_h, E'_o, and c_{hv}).
2. It assumes that the drawdown curve is a simple parabolic profile, but this is not justifiable given the complex conditions in which drainage flow occurs.
3. Flow is assumed to be purely horizontal, yet it is obvious that vertical flow is also possible.
4. No account is taken of possible sources of vertical and horizontal recharge in the strata located within the distance influenced by well pumping.

In this paper, an alternative method that removes drawbacks (1) to (3) above is implemented for analysis of the effects of ground water lowering on the shaft resistance and settlement of nearby pile foundations. This is achieved by integrating a specially developed Windows program (PILESET) for pile analysis with a finite element analysis (FEA) of well pumping. Pore-water pressures as well as soil displacements under long-term conditions are calculated in the FEA. PILESET is designed to capture electronic site investigation data from in situ tests, which may be supplemented by manually input laboratory soil test data. In this method, results of ground settlement from FEA are automatically transferred into a pile analysis program, which incorporates a module for load-transfer analysis, commonly referred to as the "t–z" analysis (Coyle and Reese 1966). In this paper, the analysis is formulated in such a way as to allow customization of shaft and base transfer models and generation of the required soil–pile parameters directly from site investigation records. The program is an extension of a previous version reported by Omer et al. (2006).

Development of the proposed method of analysis

From eq. [3], the settlement δ_n at the middle of the nth soil layer in contact with a pile shaft (see Fig. 3) can be determined as follows:

$$[4] \qquad \delta_n = \sum_{i=1}^{i=n} \Delta D_i$$

In this paper, an interactive FEA program, PLAXIS (Plaxis bv 2006), was used to perform a steady-state ground water flow analysis of well pumping. The computed results of ground settlement at various depths along a pile shaft are transferred into a prepared pile analysis program as input text files. After inputting the soil profile and pile dimensions into the program, values of δ_n are generated automatically from eq. [4], which forms part of the algorithm in the author's program. The program allows the user to input a set of parameters to define the t–z and the q–z curves (where q is the unit base resistance) representing unit shaft and base resistance variations with displacement, respectively. As explained previously, the displacements will have already been read into the program from the results file of the FEA. The local unit shaft resistance, f_{sn}, for a given soil layer is related to the relative pile–soil displacement, δ_n, at the mid-depth of the layer (see Fig. 3) using the following form of the hyperbolic function, with all parameters in SI units, suggested by Kim et al. (1999):

$$[5] \qquad f_{sn} = \frac{f_{usn}\delta_n}{(\sqrt{B}/C\alpha_1) + (\delta_n/\alpha_1)}$$

Fig. 2. Dimensionless drawdown curve for horizontal plane flow to a line of wells (low-permeability soil).

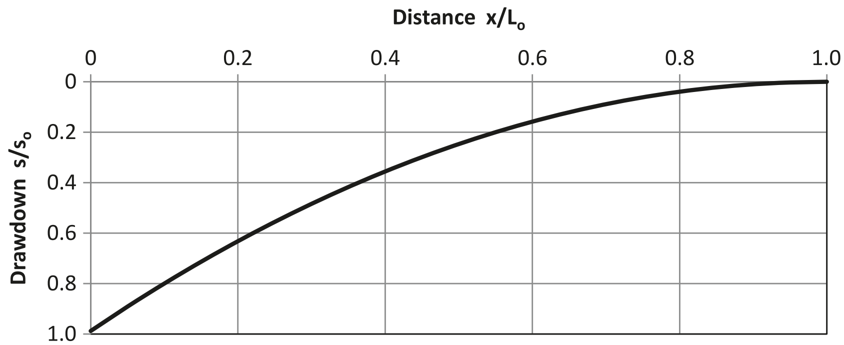
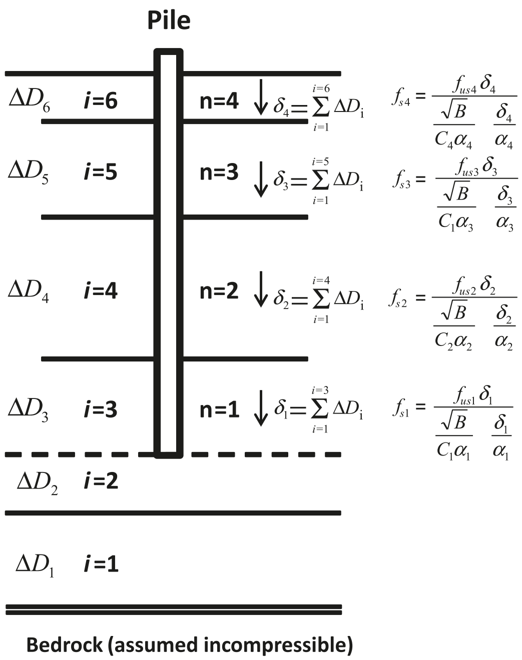


Fig. 3. Formulating shaft and base resistance curves from ground settlements. B, pile diameter; C, curve modelling parameter; f_s, shaft resistance; f_{us}, ultimate shaft resistance; α, curve modelling parameter.



where f_{usn} is the ultimate unit shaft resistance for that layer, B is the pile diameter, and C and α_1 are curve modelling parameters (f_{usn}, C, and α_1 are related to the soil strength and stiffness properties already input as SI data into the program or calculated internally). It is worth noting that eq. [5] was developed in empirical terms using curve fitting methods to recreate hyperbolic t–z curves that matched observations from an instrumented bored pile formed in weak, weathered rocks. The pile was subjected to pull-out loading tests to measure the relationship between local unit shaft resistance and relative pile–soil displacement. By combining different values of C and α_1, a series of possible t–z curves, with varying stiffness values (tangent slope at origin) was developed to represent any soil–pile situation. Therefore, eq. [5] can be used for any pile by simply selecting the appropriate values of f_{usn}, C, and α_1. Usually, f_{usn} would be estimated from soil mechanics equations of pile shaft resistance, but the best way to assess C and α_1 for a given pile site is to develop t–z curves using a site investigation probe (Lambson et al. 1992) or model pile tests in a laboratory. The model formulated using eq. [5] can be used for both positive and negative shaft resistance because the direction of the relative pile–soil movement has little effect on the rate of shaft resistance development.

As ground settlement around the pile induces negative shaft resistance, the program distinguishes this from positive shaft resistance using dialogue box that is provided for the user to specify whether δ_n values are to be generated externally or internally. When analyzing pile settlement under positive shaft resistance (downward loading on the pile head), δ_n values are internally generated by the program by imposing small incremental base displacements and solving force equilibrium and displacement compatibility equations involving all pile–soil segments. For base transfer (q–z), the following equation has been developed for a circular pile shaft, based on the original hyperbolic model, with all quantities in SI units, suggested by Fleming (1992):

$$[6] \qquad f_b = \frac{4f_{ub}BE_b\delta_b}{(0.6\pi B^2 f_{ub} + 4BE_b\delta_b)}$$

where f_b is the unit base resistance mobilized due to base movement of δ_b; f_{ub} is the ultimate unit base resistance; B is the base diameter; E_b is the soil stiffness at pile base level. Here again, E_b and f_{ub} are related to the soil properties already input into the program as SI data. The program predicts the complete load–settlement curve for the pile subjected to negative shaft resistance due to settlement induced by ground water lowering.

Using FEA to generate the required ground settlements for pile analysis

In contrast to the simplified method of Preene et al. (2000), the PLAXIS 2D finite element prediction of settlement takes into consideration flow in two dimensions and allows different permeability values to be defined for the various strata through which flow takes place. The method is based on Darcy's law, which may be expressed as follows:

$$[7] \qquad \begin{cases} v_x = -k_x \dfrac{\partial H}{\partial x} \\ v_y = -k_y \dfrac{\partial H}{\partial y} \end{cases}$$

where v_x and v_y are the discharge velocities in the x (horizontal) and y (vertical) directions, respectively, and k_x and k_y are the coefficients of permeability in the x and y directions, respectively. Further, considering the continuity of flow and assuming isotropic permeability, the variation of the total head H can be expressed as follows:

$$[8] \qquad \frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} = 0$$

In the modelling of ground water flow, the total head H is the primary variable. Thus, for each element, the values of total head at the element nodes (matrix $\mathbf{H}^e$) are related to the nodal flows (matrix $\mathbf{Q}^e$) and the element flow matrix $\mathbf{F}^e$ as follows:

$$[9] \qquad \mathbf{F}^e\mathbf{H}^e = \mathbf{Q}^e$$

The procedure for developing the element flow matrix is similar to that used for the stiffness matrix in the FEA of deformation. In addition, the individual element flow matrices are assembled in the same way as usually done for the stiffness matrix in the FEA of deformation. Thus, the global flow equation is written in the form $\mathbf{F}\,\mathbf{H} = \mathbf{Q}$, where $\mathbf{F}$ is the assembled flow matrix, $\mathbf{H}$ is the matrix of nodal groundwater head for the full mesh, and $\mathbf{Q}$ is a flow matrix containing terms associated with the flow boundary conditions. To solve for the nodal groundwater heads, the global flow equation is inverted (e.g., using Gauss elimination procedure). Finally, the flow rate within a given element is determined from the respective element's flow equations.

An interactive program for pile capacity and settlement prediction

An interactive program has been developed by the author for use as a robust and flexible tool for rapid analysis of the following: (*i*) interpretation of the soil profile at a site using cone penetration test (CPT) input data files, (*ii*) calculation of the bearing capacity of different pile types using either in situ or laboratory soil data (or a combination of both), and (*iii*) prediction of the load–settlement behaviour of different pile types under vertical axial compression loading.

Use of the program for pile capacity prediction

The following in situ data may be used for prediction of both pile capacity and load–settlement response: (*i*) standard penetration test (SPT), (*ii*) pressure-meter test (PMT), and (*iii*) CPT. A variety of laboratory soil data (used in various combinations) may also be used, e.g., undrained cohesion, consistency limits, angle of internal friction, relative density, mean particle size, and consolidation properties. The program computes the shaft and base capacities of the particular pile type defined by applying all the "built-in" calculation methods appropriate to the input soil profile, pile type, and soil properties. A total of 23 pile capacity calculation methods are provided in the program. Most of the methods have been coded from national design methods presented by piling experts representing various European countries (France, the Netherlands, United Kingdom, Finland, Belgium, Germany, Italy, Norway, and Ireland) at a previous symposium. The methods, which are listed in Table 1, were compiled by De Cock and Legrand (1997). Other methods included in the program are from published journal articles and technical reports of professional institutions, e.g., the Marine Technology Directorate, UK. As shown in Table 1, the methods are grouped under the relevant type of soil input data required. For all the calculation methods, the outputs of shaft and base capacities of the pile being analysed can be viewed, saved, printed or exported to another application.

Use of the program for load–settlement prediction

The author's program has been developed further for the prediction of the complete load–settlement curve of a pile, using the following five methods taken from various publications:

1. A numerical method (Delpak et al. 2000) previously developed by the author and published jointly. This is a complete pile analysis method that can only be implemented as a computer program. Due to the large number of equations and iterative processes involved, there is insufficient space to describe the method within this paper.
2. A method suggested by Fleming (1992) that defines shaft and base transfer using hyperbolic functions. The functions are of the form shown in eq. [10], but become more complex when combined with expressions for pile compression and effective length (centre of friction transfer) to arrive at the load–settlement relationship. The method, which is too long to be reproduced here, uses empirical constants assessed from soil–pile properties. The author has programmed the method with flexibility so that the empirical constants (shaft and base stiffness parameters) can be either determined internally by the program from SPT input data or entered manually by the user.
3. A general load-transfer (t–z) procedure (Coyle and Reese 1966), which has been also coded by the author in a flexible way, that allows the program user to customize hyperbolic t–z curves by defining site-specific curve modelling parameters.
4. A second generalized t–z solution procedure in which the ultimate local shaft resistances involved in the shaft load-transfer curves are expressed in terms of PMT data, so that the program automatically generates t–z curves from input PMT data.
5. A simplified CPT-based method described by Everts and Luger (1997).

Application of the integrated FE and load-transfer analysis to a real geotechnical project

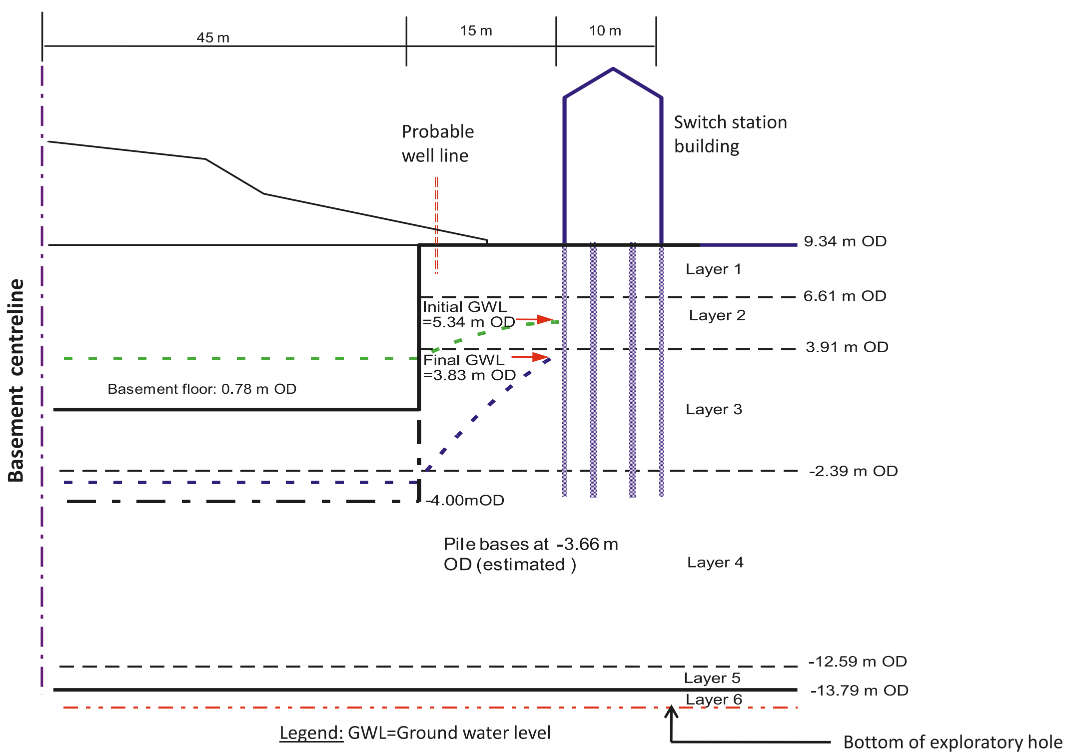
An old basement structure 9 m deep and having rectangular plan dimensions of 90 m by 600 m had existed at a site in South Wales (UK) for many years and had been back-filled with pulverized fuel ash (PFA). The intention was to remove the backfill to allow the basement to be converted into underground space for machine installations for a new power station. The problem was that the ground water table in and around the basement remained high for long periods every year and thus, removal of the backfill could lead to the uplift failure of the basement structure. Therefore, the plan was to excavate the backfill in stages while lowering the water table sufficiently and slowly enough to avoid the risk of instability. Another problem was that because of the close proximity of the basement structure to an existing switch station building, the piled foundations of the building were at risk of settlement due to negative shaft resistance induced by the process of ground water lowering. In this paper, the focus is on the analysis of the negative shaft resistance and settlement of the piles.

Table 1. Load capacity prediction methods based on CPT/CPTu records.

Data type	References and calculation methods
CPT/CPTu	1. Eslami and Fellenius (1997)
	2. Everts and Luger (1997)
	3. Bustamante and Frank (1997)
	4. Meyerhof (1983) – relating shaft capacity to sleeve friction, f_s
	5. Meyerhof (1983) – relating shaft capacity to cone resistance, q_c
	6. De Ruiter and Beringen (1979)
	7. Almeida et al. (1996); Simonsen and Athanasiu (1997)
	8. Holeyman et al. (1997)
	9. Katzenbach and Moormann (1997)
	10. Jardine and Chow (1996) – undrained analysis for base capacity in clay
	11. Jardine and Chow (1996) – drained analysis for base capacity in clay
	12. Kay (1997).
SPT	1. Svensson et al. (1997)
	2. Mandolini (1997)
	3. Lehane (1997)
PMT	1. Bustamante and Frank (1997)
LAB	1. Finlay et al. (1997) – undrained lower bound method
	2. Finlay et al. (1997) – drained lower bound method
	3. Finlay et al. (1997) – undrained upper bound method
	4. Finlay et al. (1997) – drained upper bound method
	5. Heinonen and Hartikainen (1997)
	6. Mandolini (1997) – undrained method
	7. Mandolini (1997) – drained method

Note: CPTu, piezocone tests; SPT, standard penetration tests; PMT, pressuremeter tests; LAB, laboratory soil tests.

Fig. 4. Geotechnical model for the ground water lowering problem.



Idealized geotechnical model of the problem

Because the basement structure was very long in one direction in comparison to the other, the drawdown of ground water due to pumping from the lines of wells installed on either side of the basement can be idealized as a plane horizontal flow according to the illustration in Fig. 1. Figure 4 shows the idealized geotechnical model for the analysis of this problem. As the model is symmetrical about the centerline of the

Table 2. Soil properties interpreted for settlement analysis (FEA method).

Parameter	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6
Layer description	Red brown silty sandy gravel	Firm brown sandy gravelly clay	Soft grey slightly sandy organic clay	Grey silty fine-medium sand (Alluvium)	Brown silty sandy fine to coarse gravel	Very weak to weak red brown mudstone (zone II/III)
γ_{sat} (kN/m^3)	19	19	15	21	20	22
N	36	10	6	21	32	50
c_u (kN/m^2)	—	22	15	—	—	1000
ϕ' (°)	33.5	—	—	35	40	?
k_h (m/s)	3.7×10^{-7}	1.5×10^{-8}	1.5×10^{-8}	8.5×10^{-6}	9.7×10^{-7}	?
E'_o (kN/m^2)	85×10^3	7.7×10^3	5.25×10^3	30×10^3	85×10^3	?
c_{hv} (m^2/s)	3.15×10^{-3}	1.16×10^{-5}	7.88×10^{-6}	2.55×10^{-2}	8.25×10^{-3}	?

Note: γ_{sat}, saturated unit weight, N, SPT blow count; c_u, undrained strength; ϕ', angle of internal friction; $\{k_h \approx k_v\}$, permeability coefficients (for horizontal and vertical) directions, respectively); E'_o, soil stiffness in one-dimensional compression; c_{hv}, consolidation coefficient for vertical compression with horizontal drainage flow; ?, data that is unknown but not required. For FEA, isotropic consolidation is assumed ($k_h = k_v$).

basement, it is only necessary to consider one-half of the basement, extending from the centerline to as far to the right as necessary to include the switch station building. The depth of the water table within the backfill was measured to be 3.44 m OD (ordnance datum), while the bottom of the basement was at 0.78 m OD. The process of ground water lowering was to be carried out in two stages. In the first stage, the backfill in the basement was to be excavated to 3.91 m OD. In the second stage, well pumping was to be carried out for a period of 12 weeks to lower the ground water level around the basement from 3.44 to –3.00 m OD. Ultimately, a deeper basement was to be created by extending the bottom of the existing basement from 0.78 to –4.0 m OD, as shown in Fig. 4. A site investigation borehole sunk close to the switch station building established the ground water level to be at 4 m depth (5.34 m OD) and revealed the soil profile illustrated in Table 2, which lists the soil data interpreted from various standard tests.

The design and execution of the ground water control project, instrumentation, and measurement systems was to be awarded to specialist contractors who would also monitor the vertical movements of the switch station building. Unfortunately, because some of the project information was deemed by the client as being confidential, the full details of the project, instrumentation, and monitoring programme were not available to the author. Nevertheless, it was reported that the ground water lowering was to be carried out using well-point pumping with 22 kW rotary suction and with the provision of an air exhauster. Typically, the working head would have been on the order of 15–17 m and maximum discharge rates up to 35 L/s for 150 mm diameter outlet pipes. The well-point spacing around the basement structure would typically be around 1–2 m, depending on the required drawdown, the low permeability of the strata, the geometry and size of the basement, and the anticipated discharge values. As for settlement monitoring, the main instrumentation was likely to comprise theodolites along with reflecting prisms as targets mounted on the walls of the switch station building. In addition, inclinometers and tiltmeters were also reported to have been used. The measurements, which were acquired electronically with a data logger over a continuous period of 12 weeks, produced three-dimensional (3D) positioning records of the monitoring points. This data was used to evaluate the vertical movements of the foundations of the building as well as any in-plane translations. The author was furnished with only the average settlement of the building after 12 weeks of pumping and maintenance of ground water at the desired level. The average settlement of the piled foundations according to the monitoring data was approximately 12.7 mm, while tilting was described as being minimal.

In the PLAXIS analysis, all strata were modelled as Mohr–Coulomb materials with assumed drained behaviour, while the basement wall and floor structures were modelled as elastic plates. The soil was modelled using 2649 six-node triangular elements, and the plates were represented by 66 three-node line elements. Also, 66 three-node line elements were used to model the interface between the soil and the plates to allow soil–structure interaction analysis. The area beneath the switch station building was modelled with finer mesh to increase the accuracy of analysis. In line with the proposed sequence of operations, the FEA was designed to model staged construction in the following order:

- *Stage zero* (initial conditions): This stage captured the in situ state of stress in the ground, the initial horizontal phreatic surface at 5.34 m OD, and hence, the prevailing pore pressure field.
- *Stage 1*: The PFA mound in the basement structure was excavated to 3.91 m OD. This altered the stress and pore pressure fields, which were calculated and updated for use in the next phase.
- *Stage 2*: The ground water level around the basement structure was lowered to –3.0 m OD by pumping at a constant rate (m^3/(s·m)) determined by the FE calculation such that if the rate was maintained and steady-state conditions prevailed after 12 weeks, the desired ground water level under the basement would remain constant at –3.0 m OD.

In all calculation phases, the soil was modelled as a Mohr–Coulomb material, while the plates were analysed as elastic materials. Figure 5 shows the deformed mesh from the PLAXIS output. As for the input conditions for ground water flow calculation, first, the initial situation was defined by a horizontal phreatic level at 5.34 m OD (known from site information) outside the basement structure and to the right-hand side of the wall. Second, the bottom boundary of the problem was defined as a closed flow boundary. To draw a new general phreatic level before performing ground water flow calculations, the ground water head at the extreme right boundary was fixed at 5.34 m OD (to represent an "influence distance"), while that at the extreme left boundary was fixed

Fig. 5. Deformed mesh and phreatic surface from FEA output.

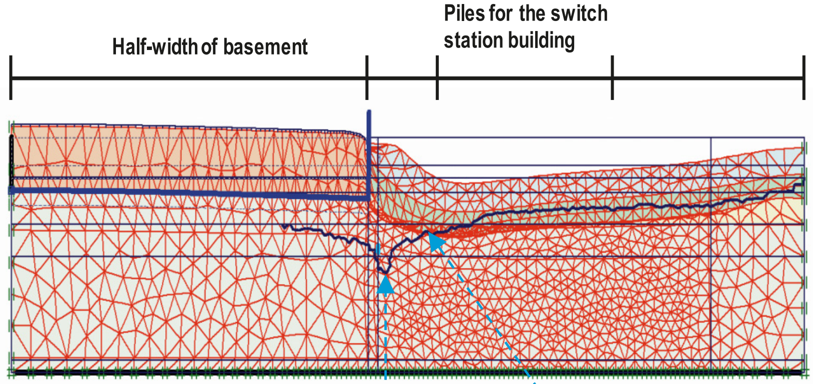


Table 3. Soil input data for pile settlement analysis using the author's program.

Stratum description (BH15A): ground level at +7.91 m OD	Soil type	Depth to base (m)	Thickness (m)	γ_{sat} (kN/m^3)	ϕ' (°)	c_u (kPa)
Made ground – red brown silty sandy gravel, N = 36	Granular	1.3	1.3	19	33.5	0
Made ground – firm brown sandy gravelly clay, N = 10	Cohesive	4	2.7	19	0	22
Vey soft dark grey brown sandy organic clay, N = 6	Cohesive	10.3	6.3	15	0	15
Grey silty fine to medium sand (alluvium), N = 13	Granular	13	2.7*	21	35	0
Grey silty fine to medium sand (alluvium), N = 13	Granular	—	—	21	35	0
Brown silty sandy gravel – river terrace, N = 32	Granular	—	—	20	40	0
Very weak and weak friable mudstone II/III, N = 50	Cohesive	—	—	22	0	1000

*Up to base of a pile.

at –3.0 m OD, i.e., the desired drawdown under the basement. To set the basement walls and floor structures as impermeable, the interfaces on both sides of the walls and floor were activated in PLAXIS. Pumping was simulated by defining a constant rate of extraction at a point in the geometry just to the right of the basement wall and within stratum No. 4. The input rate of extraction was varied in small steps, and the complete analysis of steady-state flow was repeated until the calculated results of ground water levels were consistent with the target profile. The required pumping rate was found to be 1.14 m^3/(day·m), which was well within the probable maximum pumping capacity of 35 L/s for a 150 mm outlet pipe of a well-point system. The vast output of the 3D displacements at various positions is not included due to space limitations here. The outputs of the ground settlements at points along a typical pile shaft of the switch station building were transferred into a pre-formatted data file, which was automatically read into the pile analysis program.

Table 3 lists the soil input data used in the program for pile settlement analysis, based on the geotechnical model of the site. To check the FEA results of ground settlements induced by ground water lowering, manual calculations were carried out using eq. [3], and the soil properties are shown in Tables 2 and 4. The results of maximum shaft resistance at various levels were obtained from the pile capacity calculation module of the author's program and are shown in Table 4, which also lists the ground settlements calculated manually using eq. [3]. Using the method of Finlay et al. (1997) coded in the author's program, the total shaft resistance was found to be 337 kN (sixth column of Table 4).

Interpretation of negative shaft resistance and additional settlement of pile at working load

Once the ground settlements at different depths were determined by using parallel FEA and manual methods (Table 4) in turn, it was imperative to deduce the resulting negative shaft resistance and combine it with the working load to be able to re-run the author's load–settlement analysis program, to establish how ground water lowering would affect the piles. From manual methods, using the cumulative ground settlements and maximum shaft resistance values in the fifth and sixth columns, respectively, of Table 4, the following hyperbolic shaft transfer function proposed by Fleming (1992) was used to determine the values in the seventh column of Table 4:

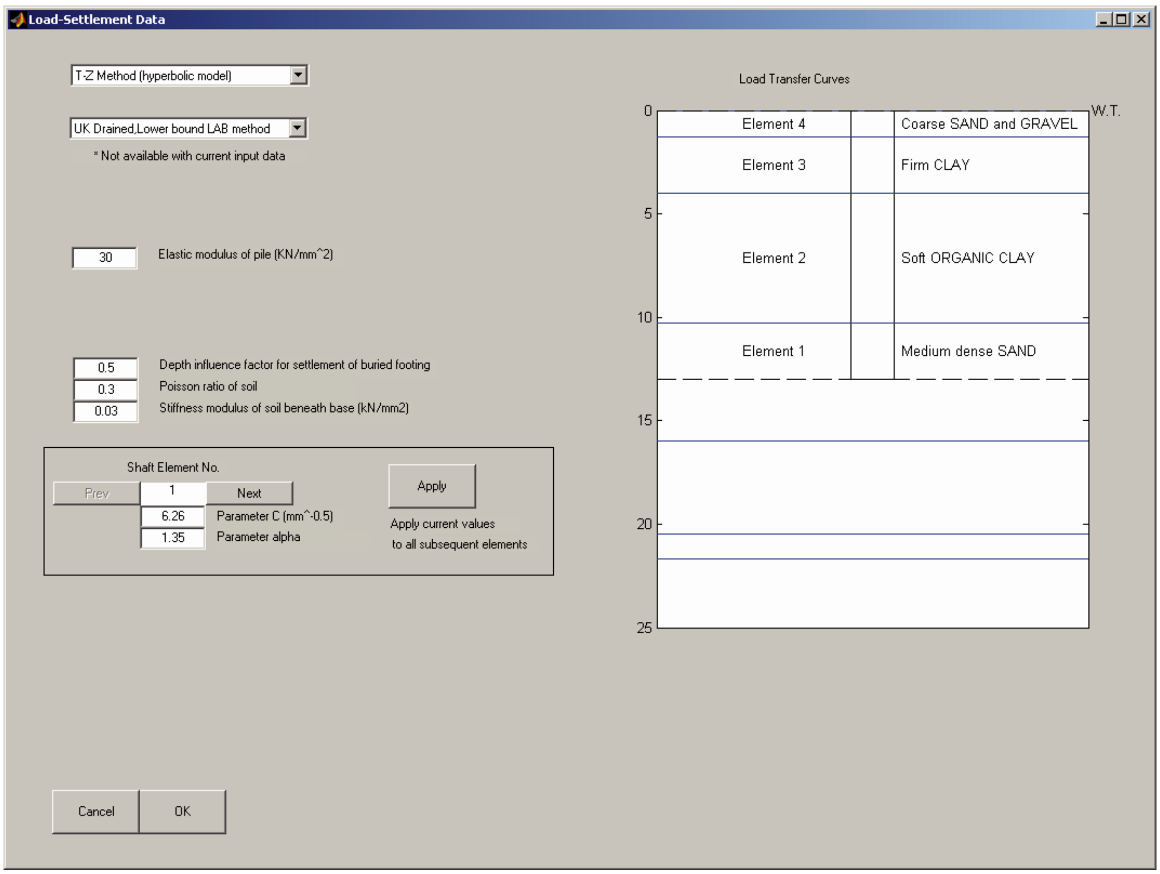
$$[10] \qquad f_s = \frac{f_{us}\Delta_s}{M_s D_s + \Delta_s}$$

Table 4. Manual calculation of ground settlement (eq. [3]) and shaft resistance results from the methods of Finlay et al. (1997) and Fleming (1992) coded in the author's program.

Layer number*	Drained modulus, E_o' (kPa)	$\Delta\sigma_v'$ (mid-stratum) (kPa)	Compression of layer (mm)	Cumulative settlement at mid-layer (mm)	Max. shaft resistance (kN) (Finlay et al. 1997 method)	Negative shaft resistance mobilized (kN) (Fleming (1992) method)
1	85 000	0	0	41.6	4	4
2	7700	0	0	41.6	53	52
3	5250	31.5	37.8	41.6	170	166
4	30 000	42.1	3.8	3.8	110	89
					Total = 337 kN	**Total = 310 kN**

*As in Table 2.

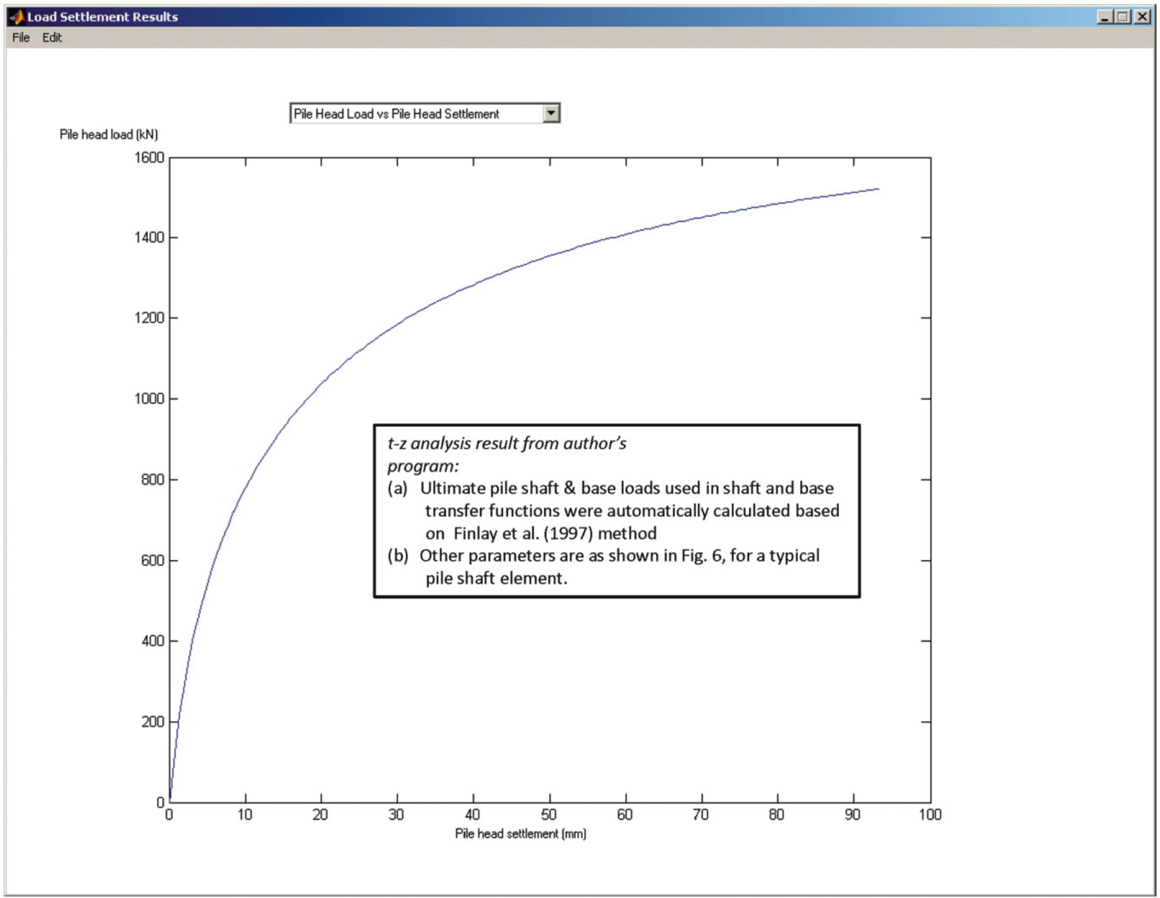
Fig. 6. Part of the user interface in the author's program (*t–z* analysis option).



where f_s is the negative unit shaft resistance mobilized at a certain depth interval, f_{us} is the maximum unit shaft resistance in compression loading (sixth column of Table 4) over the same depth interval, Δ_s is the relative pile–soil slip (cumulative settlement at mid-layer, as given in the fifth column of Table 4), D_s is the pile diameter, and M_s is the pile shaft flexibility factor (assessed as 0.002 for the pile–soil conditions). The sizes of the piles were known from construction records to be 450 mm × 450 mm sections, dimensions that had to be converted to an equivalent diameter for the purpose of this analysis. It can be seen that, as expected, none of the negative shaft resistance values estimated based on eq. [10] exceed the maximum positive shaft resistance values (sixth column of Table 4).

The deformed mesh and final phreatic surface used for obtaining the FEA results are shown in Fig. 5. This analysis accounts for many of the factors ignored when using the simplified method described by Preene et al. (2000). When the FEA results file was input into the author's program for pile analysis, it was found that the ground settlements due to water table lowering were enough to induce a drag-down force of 320 kN on a typical pile. This value resulted from the use of eq. [5] within the author's program. The value agrees well with the value of 310 kN obtained from manual calculations based on Fleming (1992), as given in seventh column of Table 4. Also, both of the above values are less than the maximum positive shaft resistance of 337 kN calculated by the author's program using the method of Finlay et al. (1997). From the pile installation records, it was found that the working load for a typical pile was approximately 482 kN. Thus, as a result of the ground water lowering process, a typical pile would be carrying a new downward force

Fig. 7. Load–settlement curve predicted by author's program (*t–z* analysis option).



of (482 + 320) = 802 kN, which would obviously cause additional settlement of the pile. The *t–z* analysis option in the author's program (see Fig. 6) produced the load–settlement curve shown in Fig. 7. The curve shows that the settlement corresponding to the increased pile load (802 kN) was approximately 12.0 mm. This value is close to the value of 12.7 mm obtained by monitoring the switch station building over a period of 6 months. Regrettably, for reasons that have already been stated, the full monitoring records showing settlement–time data were not available to the author. However, it was revealed that the data was related to the settlement of the switch station building itself, not the ground surface around the building. To check the result independently, the following widely recognized formula for pile settlement suggested by Bowles (1994) was used:

$$[11] \qquad \Delta H = \Delta q D \frac{1 - \nu^2}{E_s} m I_s I_F F_1$$

where Δq is the base pressure (sum of working load and negative shaft resistance divided by area of the pile base), ν is the Poisson's ratio of soil (assumed to be 0.35), E_s is the stress–strain modulus of soil below the pile base (assessed as 30 000 kPa from soil description and SPT data), mI_s is the shape factor (assumed equal to 1.0), I_F is the Fox (1948) embedment factor (taken as 0.5 because the pile length to width ratio exceeded 5), and F_1 is the reduction factor (taken as 0.5 because the base load mobilized at the working load was significant, i.e., 482–337 = 145 kN). Using this simple method, the pile settlement — accounting for down-drag caused by ground water lowering — was calculated to be approximately 14.3 mm. Therefore, it is seen that the pile settlements from all of the following methods are reasonably consistent: (*i*) measured value = 12.7 mm, (*ii*) result from the integrated FEA and *t–z* analysis = 12.0 mm, and (*iii*) result from Bowles (1994) formula = 14.3 mm.

Conclusions

The method for integrating FEA results of ground settlement with pile load-transfer analysis was shown to be successful. The pile analysis program developed by the author allows the user to customize *t–z* and *q–z* curves by defining different parameters as appropriate to the pile site. Unlike other simplified analytical methods, the FE method takes into account ground water flow through multiple strata with different permeability values. It also considers flow in two dimensions, which is more realistic than in the simple methods. To demonstrate the applicability of the method, a case record was analyzed for the effects of ground water lowering on piles. The predicted pile settlements caused by down-drag forces of soil settlement were found to be reasonably consistent with the measured ones.

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