A WAY TO UNITE GEOLOGY AND GEOTECHNICAL ENGINEERING THROUGH THE RECENT UNDERSTANDING OF SOIL BEHAVIOUR AND DEVELOPMENT IN MATHEMATICAL MODELS

Charles Ng¹⁾ and Yoshinori Iwasaki²⁾

¹⁾ Department of Civil and Structural Engineering, Hong Kong University of Science and Technology, Clear Water Bay, Hong Kong

²⁾ Geo Research Institute, Itachibori, Nishi - ku, Osaka, 550, Japan

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Abstract : Nearly all engineering structures are supported in some way on natural earth materials; this inevitably creates an interaction between geological conditions and foundation analysis, design and construction. Thus, both geologists and geotechnical engineers have a vital role to play in many engineering operations which interact with the ground. In this paper, the interplay between geotechnical engineers and geologists is discussed.

To analyse and design geotechnical engineering works in urban areas, the practice of geotechnical engineers includes the study of the mechanical properties of natural earth materials and, very often, the use of mathematical models to describe the fundamental behaviour of these materials. With the help of recent advances in techniques of site investigation and laboratory testing, new insights into soil stress-strain relationships at small strains and recent stress history effects have been gained and are introduced in this paper. In addition, some mathematical soil models such as the non-linear Brick model, which has the capability to capture these new observed soil behaviour, is discussed and examined. The use of geological information in assisting the back-analysis of a case history in the city centre of Cambridge is illustrated. The ongoing processes of field monitoring and back-analysis can help to refine and improve our understanding of the behaviour of soil structure interactions, which in turn provides guidance for future design. It is intended that through the introduction, discussion and illustration, geologists will have a better understanding of soil behaviour. It is hoped that this understanding will help to create a better partnership between geologists and geotechnical engineers in the future.

Key Words : Stress - strain relationship, small strains, recent stress history, Brick model

NOTATION

c_u	undrained shear strength	δ	small increment
E_u	undrained Young's modulus	Δ	increment
f, F	two non - linear functions	$\delta s'$	incremental mean effective normal stress
G	shear modulus ($1/3dq/d\varepsilon_s$)	δt	incremental shear stress
G_t	tangent shear modulus	δv_e	additional elastic volumetric strain capacity
N	standard penetration "N" value	δv_e	incremental elastic volumetric strain
K'	effective bulk modulus ($dp'/d\varepsilon_v$)	$\delta \gamma_e$	additional elastic shear strain capacity
K_o	coefficient of earth pressure at rest	$\delta \gamma_e$	incremental elastic shear strain
K_{onc}	coefficient of earth pressure at rest for normally consolidated soil	ε_L	axial strain
K_p	passive earth pressure coefficient	$\varepsilon_x, \varepsilon_y$	normal strains
LDT	linear displacement transducer	ε_s	shear strain ($2(\varepsilon_1 - \varepsilon_3)/3$)
OCR	overconsolidation ratio	ε_v	volumetric strain ($\varepsilon_1 + 2\varepsilon_3$)
p'	$(\sigma'_1 + 2\sigma'_3)/3$	$\varepsilon_1, \varepsilon_3$	principal strains
q	$\sigma'_1 - \sigma'_3$	ι	initial slope of unload/load line in $v - \ln s'$ space
s'	$(\sigma'_1 + \sigma'_3)/2$	κ	average slope of unload/load line in $v - \ln s'$ space
s_o	mean normal stress in a slurry state	λ	slope of 1D compression line in $v - \ln s'$ space
SBPM	self - boring pressure meter	μ'	effective Poisson's ratio
t	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$	σ'_h	horizontal effective stress
v	volumetric strain ($\varepsilon_1 + \varepsilon_3$)	σ'_v	vertical effective stress
v_o	volumetric strain in a slurry state	σ'_1, σ'_3	principal effective normal stresses
β	material constant allowing for overconsolidation effects	ϕ'	effective angle of friction
γ	shear strain ($\varepsilon_1 - \varepsilon_3$)		

INTRODUCTION

This paper discusses the relationship between engineering geologists and geotechnical engineers with emphasis on the importance on collaboration. To improve understanding between the two, the paper introduces the current knowledge of the stress-strain behaviour of soils at small strains and a state-of-the-art soil model, which has the ability to mimic the soil behaviour. This model is subsequently used to back-analyse an urban deep excavation for which the knowledge of both geology at the excavation and the soil behaviour at small strains is vital to understand the soil-structure interaction of the excavation.

INTERPLAY BETWEEN ENGINEERING GEOLOGISTS AND GEOTECHNICAL ENGINEERS

Nearly all engineering structures are supported in some way on natural earth materials, which inevitably creates a connection between geological conditions and foundation analysis, design and construction. To assist with the analysis and design of these engineering structures, geotechnical engineers study the physical and mechanical properties of natural earth materials and very often use mathematical models to describe the fundamental behaviour of these materials. On the other hand, geologists normally study the composition and arrangement of the earth's crust from which all these natural earth materials are derived.

There is a widespread recognition of the vital importance of geology to those who practise the art of geotechnical engineering. Under the name of soil mechanics, geology is also commonly included as a basic subject in the training courses for geotechnical engineers. Geological considerations are essential for the design of many civil engineering structures.

When geologists are called in to advise in geotechnical engineering work, they will act in conjunction with the engineers responsible for the work. Thus, the need for co-operation arises between geotechnical engineers and engineering geologists. Their co-operation may lead to a valuable partnership. In some ways, their partnership is a union of opposites, for the approach of the two to the same problem is psychologically different. The geologists analyse natural conditions as they are; the engineers consider how the existing natural conditions can be altered so that they will suit a specific safe, economical and environmentally friendly engineering design. The geologists draw on their analysis to cite problems that exist naturally and suggest problems that may arise; the engineers must solve the problems and overcome the difficulties. The final responsibility for decisions concerning the project must always rest with the engineers; but in reaching such decisions, the engineers will be guided by and will probably rely upon the factual information provided by the geologists.

It is important to steadily develop liaison and un-

derstanding between the scientific approach to the geology of rocks and soils and the engineering investigation of their properties to assist in building up this valuable partnership between engineering geologists and geotechnical engineers. It is inevitable that this interaction will progress to the mutual benefit of both geology and geotechnical engineering and to the continued advance of human understanding of the most common of all solid materials.

To ensure the valuable partnership, the joint work calls for a great degree of real co-operation. The engineering geologists must remember that what the engineers want is a clear picture, presented as concisely as possible, of the geological conditions related to the work, with a view to practical utilization. On the other hand, the engineers must remember that the geologists are not engineers, and they cannot be expected to deliver the kind of report that would come from other engineers. Under many circumstances, the most effective results can be achieved if the engineers are able to give the geologists at the outset a list of specific questions to be answered. The questions may relate to geological conditions, and to the necessity for and the locations of further exploratory works such as boreholes and exploratory pits.

SOIL MECHANICS AND GEOLOGY

Soil mechanics is the universally accepted geotechnical term for the engineering study of soils. However, what is soil and what is rock remains a debatable subject in many cases. Soil mechanics owes its origin to Karl Terzaghi, an Austrian civil engineer, who is generally recognised as the father of soil mechanics. His famous book on soil mechanics published in 1925, *Erdbaumechanik auf bodenphysikalischer Grundlage*, translated by A. Casagrande into the English *From theory to practice in soil mechanics*, is now credited as being the birth of soil mechanics. Since then, soil mechanics and geotechnical engineering have rapidly evolved in response to immense construction projects such as airfields in the 1940s, the vast network of highways and bridges built in the 1950s and 1960s, and the large amount of land reclamation, deep excavations and tunnels constructed in recent years.

"Unconsolidated materials" (geologists) and "soils" (geotechnical engineers) describe those natural materials found in the earth's crust that constitute so large a part of the actual surface of the earth that few geotechnical engineering operations apart from rock tunnelling can be constructed without an encounter with soil of some type. Since it will not always be economical to make foundations in solid rock, the founding of structures on "unconsolidated materials" or "soils" is probably the most important part of geotechnical engineering. Geologists and soil mechanicians generally maintain different levels of appreciation of soils. The geologists normally have an ingrained interest in origin, and the soil mechanicians are preoccupied with soil types and their par-

ticular mechanical properties.

Unlike most other engineering materials, soils originate from various geological processes such as deposition, erosion and weathering. Many engineered works come in close proximity to soils. Problems and difficulties in design and construction are many times more frequent when structures are to be founded on soil rather than on bedrock. Over recent years, increasing emphasis is being placed on geological engineering and mechanical evaluation of soils. When soil and rock mechanics have become vital parts of the scientific core of geotechnical engineering, the scientific study of rocks and soils must have contact with geology. Soil mechanics closely approaches a geological study when field investigations are involved. It would be virtually impossible to conduct field studies of soil without reference to the local geology. However, some soil studies have been conducted in the field without benefit from contact with geology in any form. Sometimes no harm has resulted, but this has been through good luck rather than through an appropriate management system. Poor results have often been obtained through patent neglect of geological features. In many cases, money has been spent unnecessarily because subsurface explorations were not co-ordinated with local geology, and occasionally the neglect of geology has had disastrous results. It can therefore be stated that studies of soil mechanics in the field are incomplete without some consideration of the appropriate local geology. Herein lies perhaps the chief contact between geology and soil mechanics.

OBSERVED NON-LINEAR STRESS-STRAIN BEHAVIOUR OF SOILS AT SMALL STRAINS

Non-linear stress-strain relationship

Weathering and surficial geological processes produce weathered rocks and surficial deposits, called "soil". An important aspect of geotechnical engineering is the understanding of the mechanical behaviour of soils, from which engineering calculations and mathematical models for soil behaviour are formulated.

The assumption of a linear stress-strain relationship of soil inside the state boundary surface (Schofield and Wroth, 1968) has been applied to almost all geotechnical engineering practice for years. Only recently has it been widely accepted that the stress-strain relationship is highly non-linear even at very small strains (see Figure 1), a fact that emerged from recent advances in both laboratory (Atkinson and Salfors, 1991; Goto *et al.* 1991) and field measuring techniques (Tatsuoka and Kohata, 1994). Typical strain ranges for retaining walls, foundations and tunnels constructed in stiff soils such as stiff clays and dense sands are over the range of strains where there is the greatest variation of stiffness. The stiffness of most soils decreases as strain increases and the stiffness at small strains is strongly dependent on

the current state and the recent stress or strain history of each soil element (Atkinson *et al.*, 1990).

Conventionally, the determination of the axial soil stiffness of a triaxial sample is based on external measurements of displacement which include a number of extraneous movements. The true soil strains developed can be easily masked by these movements, leading to a poor definition of stress-strain behaviour of the material under test, particularly over a small strain range. The important sources of error (Jardine *et al.*, 1984) are illustrated in Figure 2. Some of the deflections shown in the figure may be quantified by careful calibration, but large unaccountable errors remain due to:

Fig.1 Approximate strain limits for soil structures (Mair, 1993)

Fig.2 Sources of error in external strain measurements (Jardine *et al.*, 1984)

- (a) The difficulty of trimming a sample so that the end faces are perpendicular to the vertical axis of symmetry.
- (b) Play in the connection between the load cell and the sample top cap.
- (c) The inevitable "bedding down" at the ends of the sample, due to local surface irregularities or voids.

Therefore, most triaxial tests tend to give apparent soil stiffnesses far lower than those inferred from field observations.

The revolution of static and dynamic stiffness measuring devices (Tatsuoka and Kohata, 1994; Jardine, 1994) enabled accurate measurement of soil stiffness at small strains on soil samples subjected to different stress histories and stress paths. Figure 3 shows the small strain measuring device - LDT for use in a triaxial test. Local deformation is measured by two thin strain gauged strips of phosphor bronze directly attached to the sample on which two hinges are mounted. As the soil deforms, the distance be-

tween the two hinges changes, as does the curvature of the LDTs. The recorded bending strains can be converted to axial strains once the LDTs have been calibrated. The resolution of this type of device has recently been improved (Ng *et al.*, 1995a) and it is capable of measuring strain as low as 10^{-7} with an accuracy of 10^{-5} .

Figure 4 illustrates the stress-strain data from a test on a high quality intact London Clay sample. It is evident that the strains deduced from external measurements of deflection, even though corrected for load cell and apparatus compliance, gave much larger strains than values measured locally on the specimen. It is clear that the stress-strain behaviour of soils is highly non-linear over almost the entire strain range.

Figure 5 shows the measured normalized stiffness (E_u/c_u) versus axial strain (ϵ_L) for a range of soils, reported by Jardine *et al.* (1984). The plots demonstrate the following main points:

- (a) The stress-strain relationship for soils is highly non-linear over a wide range of strains.
- (b) The tests on Chalk gave the highest normalized stiffness, equal to those of the low plasticity clay at small strains but exceeding them at strains above 0.01%.
- (c) The London Clay tests showed stiffness characteristics similar to other heavily overconsolidated intact or remoulded low plasticity clays.
- (d) The stiffness characteristics for the Ham river sand form a lower bound to all results reported in the diagram.

Figure 6 shows a way to illustrate the variation in shear stiffness (G) with shear strain (γ) for the Italian Vallericca Clay tested in a triaxial apparatus

Fig.3 Local measurement of axial strain using a pair of LDTs (Goto *et al.*, 1991)

Fig.4 Stress - strain data (Jardine *et al.*, 1984)

Fig.5 Summary of normalized stiffnesses (Jardine *et al.*, 1984)

Fig.6 Comparison between resonance column (RC) and triaxial (TX) tests for a range of confining pressures: Vallericca Clay (Georgiannou *et al.*, 1991)

Fig.7 Effect of recent stress history on current stiffness (Atkinson *et al.*, 1990)

(TX) and a resonance column (RC). The effect of confining pressure on the measured shear stiffness (G) is clearly shown.

Recent stress history effects of overconsolidated soil

It is now well understood that soil stiffness is highly non-linear, being greatest at the start of loading when the increments of stress and strain are small. For a given soil at a given state with a particular stress history, the measured stiffness is often different for different stress paths such as compression or extension, drained or undrained, loading or unloading. Som (1968) observed that the compressibility of samples in oedometer tests was considerably re-

Fig.8 Stiffness of reconstituted London Clay measured in (a) constant p' tests; (b) constant q' tests (Atkinson *et al.*, 1990)

duced (i.e., the stiffness was increased) following an extended period of constant loading. Atkinson (1973) found that the stiffness of samples in triaxial and plane strain tests was increased following a sudden change in the direction of the stress path. These have often been referred to as threshold effects. Atkinson *et al.* (1990) conducted a series of tests using reconstituted London Clay specimens to systematically study the previously observed threshold effects. They concluded that in addition to the overall stress history, which is best described by the overconsolidated ratio, the recent stress history, which includes either a sudden change in the direction of the stress path or a period of time at a constant stress state, has a major effect on subsequent stiffness.

The effect of stress path rotation for loading paths which remain inside the state boundary is illustrated in Figure 7a. Samples brought to the same initial state p'_i and q'_i at O along the different paths CO and DO are then loaded along the same path OA. At O, the rotations of the stress paths relative to the new

stress path OA are θ_c and θ_a . Figure 7b illustrates the stress-strain curves for the same loading path OA. Since the samples had identical states and over-consolidation ratios at O, and providing that they were both held at this state for equal periods of time, the difference in stiffness is attributable to the different stress path rotations.

Figure 8 shows tangent stiffnesses $dq/d\varepsilon_s$ (shear) and $dp'/d\varepsilon_v$ (bulk) plotted versus the strains ε_s or ε_v , respectively. It is clear that rotation of the stress path has a significant effect of on stiffness, particularly at the start of the test. At small strains of the order of 0.01%, the stiffness for $\theta = 180^\circ$ is approximately an order of magnitude larger than the corresponding stiffness for $\theta = 0^\circ$, but at strains of the order of 0.5% the differences have been largely removed. Stiffnesses for $\theta = \pm 90^\circ$ fall between those for $\theta = 0^\circ$ and 180° . All these clearly demonstrate that the stress-strain behaviour of overconsolidated soil is non-linear and the stiffness depends on the current state, the overconsolidation ratio and the recent stress history.

MATHEMATICAL SOIL MODELS

Introduction

A most important development in the campaign to predict the stress-strain behaviour of soil was the development at Cambridge University of a group of plasticity models, introduced by Schofield and Wroth (1968) in their most philosophical and useful book "Critical State Soil Mechanics". The critical state framework, which was formulated so elegantly by the Cambridge soil mechanics group under the late Professor Roscoe, has provided a coherence which the subject previously lacked. It has also provided a logical framework for incorporating theories of plasticity, yield and flow for the mathematical modelling of soil behaviour (i.e., Cam-clay models). Over the last twenty years, critical state soil mechanics has been widely taught and increasingly applied to the solution of engineering problems (Burland, 1990). For many soil models, an explicit state boundary surface is essential for modelling soil behaviour. The state boundary surface, which is called the stable state boundary surface (SSBS), is used to separate possible from impossible states and it is usually assumed that for states inside the surface, deformations are elastic and for states on the surface, deformations are elasto-plastic, so that the state boundary surface is a yield surface.

Traditionally, the behaviour within the yield surface has been assumed to be either rigid linear elastic or non-linear elastic. The shortcomings of such assumptions are now widely appreciated, especially for predicting the behaviour of foundations constructed in stiff clays and dense sands where small strain deformations are expected under normal serviceability conditions. It is thus useful to adopt a framework that distinguishes the main phases of soil pre-failure

Fig.9 Regions of small strain and high stiffness at three states of stress history (Jardine, 1994)

behaviour in a more appropriate manner (Jardine, 1994).

Multiple kinematic yield surface models

Following the ideas of Mroz (1967), some researchers have proposed multiple kinematic yield surfaces (Jardine, 1985; Al-Tabba and Wood, 1989) to closely describe the observed and the recently understood elasto-plastic behaviour of soils within the large-scale yield surface. Figure 9 shows the basic elements of the modelling scheme (Jardine, 1994). The large-scale yield surface is represented by the Y_3 locus. Only small to moderate strains are developed by stress changes that remain within the Y_3 surface; the tangent stiffness of soil changes abruptly when a probing stress path reaches the surface. In fact, this surface denotes the current bounding surface which cannot be crossed by undrained tests. The more extensive SSBS lies outside this current surface, which can only be reached by following drained probing stress paths in which large volume strains are developed. The current effective stress point is surrounded by two inner sub-yield surfaces, called Y_1 and Y_2 , which move with the point (i.e., are kinematic). For calculation purposes, the strains are computed from origins defined at the initial stress point. The Y_3 surface remains relatively immobile and is only affected by comparatively large strain events. The Y_1 surface represents the limits to the region over which the soil response is linearly elastic when loaded. When stress changes are applied which cross the initial boundary, the Y_1 region is dragged with the stress point and elastic behaviour is only observed when the stress path direction alters so as to re-enter the Y_1 region. Once the Y_1 region is engaged, the response is initially hysteretic and non-linear and some insignificant plastic strains are predicted. Appreciable plastic straining is delayed until the stress path engages the surrounding Y_2 surface. The plastic yielding experienced at this point may be expressed by a marked shift in the directions of the strain increment vector.

A further feature of the Y_2 surface is that it defines the threshold at which drained or undrained cycling starts to significantly affect the soil.

As the stress path further increases, heading towards the Y_3 surface, substantial plastic deformation develops. As before, a sharp change in stress path direction can cause the stress point to re-enter the Y_1 and Y_2 regions.

The Brick model

An alternative but more elegant way of thinking of the concept of multiple kinematic yield surfaces for modelling the non-linear stress-strain behaviour of soil and the recent stress history effect was developed by Simpson (1992), who presented a novel way of modelling soils: "bricks on strings".

After careful study of laboratory test results of soil behaviour at small strains (see Figure 8), Simpson used a physical analogue, a man pulling a number of bricks inside a room, to illustrate the idea of modelling this observed soil behaviour. Simpson imagined a man walking around a room and pulling behind him a series of bricks, each on a separate in-extensible string. Some possible paths for the man and strings are shown in Figure 10. If he walks continuously in one direction, the bricks line up behind and follow him (see Figure 10a). This is an analogy for the plastic behaviour of soil. If the man turns back (Figure 10b), the bricks initially do not move; then those on shorter strings start to move, gradually followed by those on longer strings (Figure 10c). This analogy represents soil behaviour when its stress or strain path rotates 180° . If the man turns through 90° , the bricks initially keep moving in their previous direction but gradually swing round behind him (Figure 10d). The similarity between the man

Fig.10 The analogy - a man pulling bricks around a room (after Simpson, 1992)

with his bricks and soil behaviour is obvious.

Simpson regarded the "room" as strain space and used the movement of the "man" to represent the total strain of a soil element in the chosen strain space. Each brick represents a proportion of the soil element. The movement of each brick represents the plastic strain undergone by that portion of the soil element. The elastic strain of the element is given by the difference between the movement of the common point (the man) and the sum of the movements of the bricks, each weighted in accordance with the proportion of soil material it represents.

Figure 11 shows an "S-shaped" curve which has been idealized for a number of bricks. This type of "S-shaped" curve is commonly used to show the way in which stiffness of soils decreases with increasing shear strains. In the figure, the vertical axis is the normalized tangent shear stiffness (G_t/s'). The γ -abscissa of each brick is the string length, i.e., the maximum elastic shear strain that each portion of the soil element can undergo. The steps shown in the figure indicate the points at which strings become taut and a proportion of the soil element becomes plastic. The proportion of soil which each brick represents is given by h_i/h_o . At large strains, the "S-shaped" curve reaches the horizontal axis. This implies that the soil mass has failed. In the model, it is assumed that the soil is homogeneous and isotropic, and only elastic strain causes a change in stress.

A number of assumptions which are well-known in soil mechanics are incorporated in the model as follows:

(a) The elastic volumetric stiffness is assumed proportional to the current mean normal stress (s') and the elastic behaviour of soil at very small strains is governed by a new proportional constant ι (ι) as shown in Figure 12. The λ (λ) line is the virgin consolidation line whereas the κ (κ) line refers to unloading/reloading situations in v - $\ln s'$ space. It should be noted that the λ and κ shown in the figure are similar in principle but different in numerical values to the conventional λ and κ values

Fig.11 Idealization of the "S-shaped" stiffness-strain curve for the Brick model

Fig. 12 Proportional constants for the Brick model

used in the Cam - Clay models (Schofield and Wroth, 1968).

(b) For a plane strain formulation, the elastic shear stiffness can be derived from elastic volumetric stiffness with the assumption of a constant Poisson's ratio. This leads to the basic elastic stress - strain relationship for the Brick model given by

$$\begin{bmatrix} \delta s' \\ \delta t \end{bmatrix} = \frac{s'}{t} \begin{bmatrix} 1 & 0 \\ 0 & (1 - 2u') \end{bmatrix} \begin{bmatrix} \delta v_e \\ \delta \gamma_e \end{bmatrix} \quad (1)$$

(c) It is further assumed that during normal consolidation, an increase in mean normal stress (s') makes available an extra capacity for elastic volumetric strain. This assumption is based on the fact that when normal stress on a frictional body is increased, its ability to strain elastically (both normal and shear) without slip is also enhanced. This gives rise to the log - linear compression and swelling lines that are often clearly displayed by soils and that are represented by the parameters λ and κ in the Cam-clay models. In the Brick model, the additional elastic volumetric capacity for normal compression is given by

$$\delta v_c = \left(\frac{t}{\lambda} \right) \delta v \quad (2)$$

and during unloading and reloading, equation (2) is modified to

$$\delta v_c = \left(\frac{t}{\kappa} \right) \delta v \quad (3)$$

Similarly, additional capacity for elastic shear is also assumed and given by the following formula:

$$\delta v_c = \frac{1}{(1 - 2u')} \left(\frac{t}{s'} \right) \delta v_c \quad (4)$$

The effect of this assumption is to ensure radial straight lines occur in (s', t) stress space during anisotropic consolidation, i.e., $dt/ds = t/s'$. By combining equations 1-4, and by considering the strain history and location of the "man", the basic stress-strain equation (3) becomes

$$\begin{bmatrix} \delta s' \\ \delta t \end{bmatrix} = \begin{bmatrix} f & 0 \\ 0 & F \end{bmatrix} \begin{bmatrix} \delta v_e \\ \delta \gamma_e \end{bmatrix} \quad (5)$$

where f and F are two non-linear functions which cannot be explicitly expressed in analytical form.

(d) Soil stiffness at small strains varying linearly with the logarithm of OCR has recently been reported by some researchers (Viggiani, 1992). This observed soil behaviour is incorporated in the model by introducing a multiplier equal to $\{1 + \beta(v - v_o - \lambda \ln(s'/s'_o))\}$ to the soil stiffness, where v_o and s'_o indicate the initial state of the soil relative to which isotropic consolidation is to be measured. Normally, this state represents a slurry. For normally consolidated soils, $\beta = 0$. More details of the model are given by Simpson (1992).

Conventional parameters for numerical modelling such as ϕ' and K_o are not required for the model, but follow automatically from the brick concept and assumptions described.

IMPORTANCE OF GEOLOGICAL INFORMATION TO THE ANALYSIS OF AN URBAN DEEP EXCAVATION

The development

The Lion Yard site is situated in the city centre of Cambridge and it is approximately 65 m by 45 m on plan. The structure consists of a three-level underground carpark beneath a five-storey hotel and is supported on large diameter under-reamed bored piles founded 30 m below ground level in the Gault Clay. A cross-section of the site is shown in Figure 13. The 10-m-deep excavation is retained by a 17-m-deep perimeter concrete diaphragm wall 0.6 m thick.

Efficient design in geotechnical engineering is not easy. Nearly every geotechnical engineer must make some assumptions and he runs the risk of encountering surprises. These circumstances are the inevitable result of dealing with natural materials such as soil and rock. Field monitoring is therefore necessary to provide a means by which the geotechnical engineer can verify the design assumptions and the contractor can execute the work with safety and economy. More importantly, the field data may also be assembled into a comprehensive case record that is then available for use when checking the validity of any analytical and numerical models. The ongoing process of back-analysis can help to refine and improve our understanding, which in turn provides guidance for future designs. Without field monitoring and back-analysis to provide feedback, it is not possible to assess how conservative the design of a structure is. This is particularly so in geotechnics, where the effects of soil-structure interaction are still being explored. Figure 14 illustrates the relationship between design, construction and back-analysis.

In order to verify the design assumptions, to monitor the performance of the 10-m-deep excavation at Lion Yard and to provide valuable data for back-

Fig.13 Cross section of the site

analysis, various instruments were installed on site. Full details of the instrumentation together with interpreted results are reported by Ng (1992).

Geology and soil properties of the site

Folding, uplifting and subsequent erosion at the end of the Jurassic or in the early part of the Cretaceous period gave rise to an unconformity, so that the earliest Cretaceous formation, the Lower Greensand, rests on Kimmeridge clay in some places, and Corallian in others (Chatwin, 1961; Worssam and Taylor, 1969). The Lower Greensand was deposited in shallow water and subsequently the Gault clay was laid down as a result of a widespread marine incursion that spanned the Middle and Upper Albian stages. The conditions were not uniform throughout the period of deposition and so the Gault contains several beds of different lithology. At the end of Gault times, there was a temporary halt in deposition, accompa-

nied by submarine erosion and reworking of sediments on the sea floor.

Following the deposition of the Gault, the Chalk was laid down as the sea water cleared and the land areas dwindled so that less and less terrigenous sediment became available. During the Tertiary and Quaternary periods, uplift and extensive erosion took place and eventually produced the present landscape. An estimated 200 m to 400 m of Chalk has since been eroded.

At least two Pleistocene ice sheets passed over the Cambridge area (Chatwin, 1961; Worssam and Taylor, 1969). A subsequent interval of warm-temperature inter-glacial conditions is indicated by some of the higher river terrace deposits, followed once more by Arctic conditions during the last glaciation. Subsequently, the lowest river terrace deposits were formed.

In the Cambridge area, the thickness of Gault varies between 27 m and 42 m. At Lion Yard, a 38-m thickness of Gault was found during the installation of a standpipe piezometer in the Lower Greensand. The soil profile obtained during site investigation is shown in Figure 15. In general, groundwater conditions are approximately hydrostatic below a level of about +7.0 m OD (i.e., about 3 to 3.5 m below existing ground level).

The Gault Clay in Cambridge is highly calcareous, containing up to 30% by weight of calcium carbonate (Worssam and Taylor, 1969). A similar result (27.5 %) was found in samples taken from Lion Yard. The Gault in its natural state is heavily over-consolidated, with a natural water content close to the

Fig.14 The inter-relationships between Design, Construction and Back-analysis

Fig.15 Soil profile and geotechnical parameters

plastic limit. It consists of stiff to hard silty grey clay of high plasticity. The clay is laminated with closely spaced fissures and joints. The top few metres of clay show signs of weathering such as cryoturbation. Hard nodules of phosphatized marl are scattered through the clay.

Use of geotechnical information to assess the initial stress in the ground

It is extremely difficult to assess initial horizontal stresses in the ground correctly, but they are one of the most important parameters necessary for analysis and design of soil structures. Reloading on top of a clay deposit has a dominant effect on vertical effective stress but has only a relatively small effect on horizontal effective stress, as illustrated by many researchers (Burland *et al.*, 1979; Simpson *et al.*, 1979). They allowed for the effects of reloading by assuming isotropic elastic behaviour and using the equation given below:

$$\frac{\Delta\sigma'_h}{\Delta\sigma'_v} = \frac{u'}{1-u'} \quad (6)$$

Simpson *et al.* (1979) show that the ratios of $(\Delta\sigma'_h / \Delta\sigma'_v)$ due to reloading were 0.18 and 0.33 for isotropic and anisotropic assumptions, respectively, for London clay. Similar results are expected for the Gault. Clearly, reloading has a relatively small effect on σ'_h and therefore leads to a reduction in K_0 , especially in the top few metres of clay.

As described previously, the geological history is not simple at Lion Yard. The Gault clay has been reloaded by gravel and fill after the erosion of the Chalk. A simple calculation indicates that at 1 m below the top surface of Gault clay, an initial K_0 value

of 3.0 prior to redeposition will be reduced to less than 1.0 as a result of a 3-m-thick redeposition. This is substantially different from the in-situ self-boring pressuremeter measurements (see Figure 15). The results of these tests suggest that very high initial horizontal stresses are present in the upper 7 m of the Gault clay at Lion Yard, corresponding to K_0 values ranging from 1.5 to 3.5 when related to the current vertical effective stresses. This upper part of the clay is the most relevant, since the entire basement is located within 10 m below ground level. It is thus important to compare the results of the pressuremeter testing with other tests, and with semi-empirical and theoretical calculations.

Figure 16 shows the variation of total horizontal stresses with depth measured in Gault Clay at Lion Yard and Madingley (Powell, 1990), and estimations using the semi-empirical correlation originally proposed by Schmidt (1966), $K_0 = K_{onc} OCR^m$, where $m=1.2 \sin \phi'$. Laboratory test results ($\phi'=24^\circ$) on reconstituted specimens (Ng, 1992) have been used in the calculation. Since the maximum preconsolidation pressure at Lion Yard is not known, a range of possible preconsolidation pressures (2000 kPa to 4000 kPa) has been considered. The Madingley site is situated about 4 km west of Lion Yard and the top of the Gault clay is at the ground surface, i.e., no reloading by gravel has occurred. Figure 16b shows the corresponding distribution of K_0 with depth. It should be noted that prior to reloading, the top few metres of the clay was in a state of passive failure with K_p value of 3.5, a value based on the results of laboratory tests ($\phi'=34^\circ$) on undisturbed block samples taken from the site (Ng, 1992).

It can be seen from Figure 16a that the measured

Fig.16 Comparison of the measured initial horizontal stresses and K_0 at Lion Yard and Madingley

horizontal stresses at Lion Yard are substantially higher than the measurements using a self-boring pressuremeter at Madingley for the top 7 m of clay, even if the lateral stresses are adjusted to allow for the assumed effects of reloading by a 3-m thickness of gravel. Similarly, the measured values at Lion Yard are significantly higher than the estimated horizontal stresses using Schmidt's semi-empirical correlation for the possible range of preconsolidation pressures considered. The simplified geological history of Lion Yard has also been simulated numerically using the Brick model (Simpson, 1992). The results of the simulation agree with the predictions using Schmidt's correlation and the measurements at Madingley if reloading by gravel is considered. More details of the simulation are given by Ng (1992). Thus, it is believed that the initial horizontal stresses in the ground are likely to have been overestimated by the self-boring pressuremeter tests at Lion Yard.

Simulation of the geological history at Lion Yard

Correct modelling of the initial stress conditions in the ground and wall installation effects is vital for the back-analysis of the main excavation at Lion Yard (Ng, 1992; Ng *et al.*, 1995b). Thus, numerical modelling of the main excavation was started by simulating the geological history of the site, followed by modelling the diaphragm wall installation effect and finally the three stages of excavation at the three levels of the underground carpark.

To model the initial K_0 condition at this site, an additional analysis stage in the numerical simulation procedure was set up to mimic the geological history

Fig.17 Initial stresses in the ground

Fig.18 Displacement vectors at the end of excavation

Fig.19 Shear strains around the site at the end of excavation

of the site by consolidating the Gault clay one-dimensionally to its estimated preconsolidation pressure, followed by erosion, and then reloading by made ground and gravel. The computed initial horizontal stresses are compared with field measurements (SBPM) and other empirical predictions in Figure 17. Based on these comparisons and the geological study, it is believed that the initial K_0 values varied between 1.0 and 2.0 for the top 7 m of the clay.

Comparison of the measured performance and the computed results

Figures 18 and 19 show the computed general ground deformation vectors and shear strain contours around the site at the end of main excavation, respectively. Due to vertical stress relief inside the site, soil on the retained side moved towards the excavation whereas the soil underneath the excavation heaved upwards as depicted in Figure 18.

It can be seen in Figure 19 that the shear strains developed around the excavation were small and generally less than 0.3%, except in front of the wall just below the final excavated level where the soil was yielding. A similar pattern and magnitude of shear strains has been predicted by Arup Geotechnics for the multi-propped excavation in London clay at the British Library at Euston (Atkinson and Salfors, 1991). These indicate that the moduli adopted for any calculations based on the theory of linear elasticity should be appropriate for the average small strain level anticipated during main excavation. By studying the strains developed around excavations, an appropriate soil stiffness corresponding to the average small strains can be selected for design from laboratory test results (Ng *et al.*, 1995c).

Figure 20 shows the comparison between the computed and measured displacements of the wall for the three stages of main excavation. At the first stage of excavation, the wall behaved essentially as a cantilever wall with only small propping effects at the top of the wall. This was because the installation of some of the top level props had not been completed

before the start of the excavation. An allowance was made in the analysis by using a factored prop stiffness.

During the subsequent two stages of excavation, the deflected profiles of the wall were typical for a multi-propped wall constructed in stiff clay, i.e., deep-seated inward movements occurred (Burland *et al.*, 1979). Generally, the predicted and measured deflections of the wall are in good agreement although the Brick model overpredicted the lateral inward movement in the region of the toe, which resulted in smaller predicted wall curvature than the measured value.

Inward toe movement as a result of vertical stress relief is clearly depicted in Figure 20 by the field measurements and numerical predictions. The conventional approach of deducing wall displacements from field data by assuming fixity at the toe would have underestimated the true wall deformation.

A comparison of the measured mean and calculated prop forces at the final stage of excavation is shown in Figure 21. The predictions appear to correspond reasonably well with the field observations despite the large overprediction of prop force at L3. This is probably because the fixed end moment provided by the concrete column at the top of the wall was ignored in the analysis. The introduction of a negative moment at the top of the wall would have resulted in a smaller predicted prop force at L3. Good agreement between the measured and computed prop forces is important as this increases the confidence in the computed earth pressures, which are of course in force equilibrium with the predicted prop forces. This seems to imply that the estimated initial stresses in the ground, based on geological information and numerical simulation, are likely to be correct.

CONCLUSIONS

a) Nearly all engineering structures are supported in some way on natural earth materials, and this inevitably creates a connection between geological conditions and foundation analysis, design and con-

Fig.20 Comparison of the measured and computed displacements of the wall during excavation

struction. This implies an inevitable interaction between geologists and geotechnical engineers. Since the approach of the two parties to the same problem is psychologically different, understanding and co-operation between the two are essential for the success of a project. For geotechnical engineers, some basic knowledge of geology, and for geologists, some understanding of mechanical behaviour of soils, will certainly help to build up a valuable partnership to the mutual benefit of both geology and geotechnical engineering.

b) The stiffness of most soils decreases as strain increases and the stiffness at small strains is strongly dependent on the current state and the recent stress or strain history of each soil element.

c) The development of mathematical models in geotechnical engineering has been very rapid. Currently, constitutive laws are sufficient to describe the observed static mechanic behaviour of soils. Generally, complex soil models tend to give better predictions than simple soil models. However, these complex soil models clearly require more involvement in terms of soil parameters, time and understanding of non-linear soil behaviour and techniques of numerical modelling. Thus, the selection of complex or simple analysis depends on what resources are available and how good a prediction is required.

d) Field monitoring is necessary to provide a means by which the geotechnical engineer can verify the design assumptions and the contractor can execute the work with safety and economy. More importantly, the field data may also be assembled into a comprehensive case record that is then available for use when checking the validity of any analytical and numerical models. The ongoing process of back-analysis can help to refine and improve our understanding, which in turn provides guidance for future designs. The available geological information at Lion Yard contributed significantly to the success of back-analysis and understanding of the performance of the basement at Lion Yard.

Fig.21 Comparison of the computed and measured propping forces

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