2 Site Investigation and Geotechnical Parameters

2.1 Site Investigation

Adequate site investigation is essential to ensure safe and economic design and construction of ELS works for civil engineering and building developments. The main objectives of site investigation are to acquire knowledge of site characteristics that affect such works and plan for their safe execution, with due consideration given to the nearby buildings/structures/services. When planning ELS works, site investigation should normally include a detailed desk study, site reconnaissance, topographic survey, ground investigation (GI) for establishing a suitable ground model, collecting soil and rock samples, carrying out in-situ tests for selecting design parameters and undertaking field monitoring to determine the groundwater conditions. Geoguide 2: Guide to Site Investigation (GEO, 2017a) provides guidance on the planning and execution of site investigation works. This Chapter discusses additional considerations for planning the site investigation and selection of geotechnical parameters that are pertinent to the design and construction of ELS works.

2.1.1 Ground Investigation

GI should be properly planned in order to develop a suitable ground model for the design of ELS works, with due regard to the anticipated extent and depth of excavation. An adequate number of boreholes should be extended to the competent soil stratum or bedrock when determining the ground stratigraphy. In case the competent soil stratum is located at a considerable depth below the excavation level, boreholes should be extended to a depth where the passive resistance of the soil is anticipated to be mobilised. This is usually at least two to three times the excavation depth. Besides the stability consideration of the embedded wall, deeper boreholes may also be needed to determine the ground conditions at greater depth for seepage analysis.

The number and spacing of boreholes should be determined with due consideration of variability in the spatial distribution and thickness of each stratum, the materials to be excavated, the materials in which the wall is embedded and the bearing materials for ELS works. As recommended by Geoguide 2, borehole spacing of between 10 m and 30 m is considered adequate in general. If specific geological features are encountered that are critical to the design, such as the presence of corestones, weak or fault zones, clastic marble and highly variable rockhead, the location and spacing of the boreholes should be specifically arranged to investigate the effects of these features on ELS works.

2.1.2 Groundwater Conditions

Site-specific groundwater and drainage conditions should be ascertained within and in the vicinity of the site, and their likely response to, for example, storms, seasonal rise, artesian conditions or tidal action. Field tests normally yield more reliable parameters of the mass permeability of the ground than laboratory tests. However, it is seldom necessary to carry out pumping tests to establish the mass permeability of the ground in urban environments during the GI stage, as the tests may cause possible adverse effects on the nearby buildings/structures/services.

The measurement of groundwater levels in boreholes is usually carried out using standpipes and piezometers. Sometimes, it may be necessary to install piezometers strategically in order to measure the presence of any artesian and non-hydrostatic pressures, particularly for confined aquifers that could result in unexpected changes in groundwater pressure in underlying impermeable soil layers.

An investigation of a settlement incident of highway structures adjacent to a deep excavation project in reclaimed land in the Kai Tak area illustrated the significance of changes of piezometric head in a confined aquifer due to continuous dewatering of the nearby ELS works. The ground stratigraphy at the site concerned comprises reclamation fill, marine clay, alluvial sand and saprolite, which is typical in most reclaimed land in Hong Kong. The embedded wall was terminated at a depth slightly below the interface of the alluvial sand and saprolite. The marine clay had a low permeability which provided a water cut-off layer between the groundwater in the fill and the underlying alluvial sand. Thus, the groundwater level in the fill layer remained stable during dewatering of the excavation. However, steady-state seepage in the alluvial sand layer occurred and led to the piezometric pressure in the sand falling by a few metres. The presence of the differential piezometric pressure between the marine clay and alluvial sand layers led to dissipation of water from the clay and triggered consolidation settlement in the clay layer as illustrated in Figure 2.1. As a result, on-grade highway structures nearby had settled and tilted. In similar ground and seepage conditions, piezometers should be installed at appropriate depths on the unexcavated sides of the embedded wall (e.g. at a depth below the interface between the permeable and impermeable soils), in order to monitor the changes in groundwater pressure and to facilitate estimation of the potential consolidation settlement.

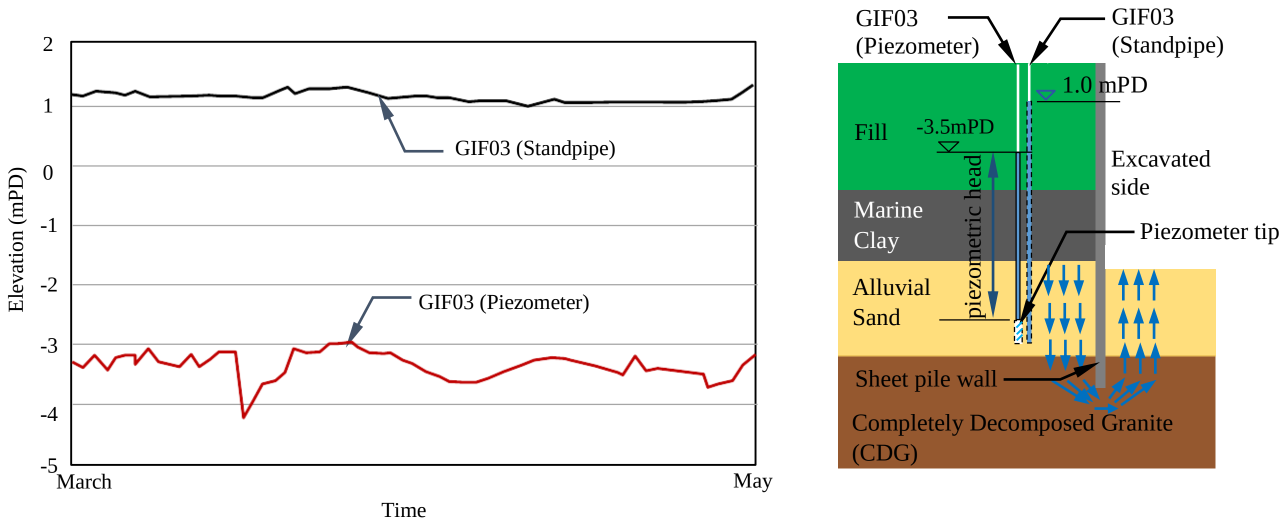

Figure 2.1 Variation of Groundwater Pressure in a Confined Aquifer

2.1.3 Soil Shear Strength and Stiffness

The sampling methods recommended in Geoguide 2 should be used to obtain good quality undisturbed soil samples (e.g. Mazier and piston samples) and representative specimens should be selected for subsequent laboratory testing to determine soil shear strength. It is preferable to collect sufficient samples for conducting single-stage consolidated triaxial tests at different confining pressures. Multi-stage consolidated triaxial tests may also be used in cases where it is difficult to obtain a sufficient number of representative and suitable specimens. Methods of determination of soil strength parameters from multi-stage triaxial test results using the Mohr-Coulomb strength model are discussed by Wong (1978). Endicott (2020) discussed the possible effects of alteration and disturbance of the soil fabric during multi-stage triaxial tests.

A sufficient number of triaxial tests (e.g. at least five soil samples for each soil stratum) should be conducted to provide representative results if practicable. Also, it is important that the ground model and geotechnical parameters should be reviewed during construction, especially for large-scale deep excavations with complex ground conditions and construction sequences.

Apart from collecting samples during the GI for laboratory tests, it is often useful to conduct field tests to obtain information to supplement the selection of design parameters. The in-situ standard penetration test (SPT) is commonly adopted to determine the stiffness of granular soils. In reclaimed land, the cone penetration test (CPT) and vane shear test are useful for determining the undrained shear strength and soil stiffness of clayey materials. The CPT provides a fast and continuous profiling of alluvial and marine deposits. However, pre-boring may be required to penetrate fill layers with substantial gravel and cobble contents, before conducting the CPT. In the determination of undrained shear strength, s_u, calibration between the CPT and representative laboratory tests is required to derive the site-specific correlation factor. Robertson & Cabal (2022) provided guidance on CPT interpretation to determine different geotechnical properties and applications of CPT results. Where necessary, dissipation tests can also be conducted as part of the CPT to determine the consolidation and compressibility characteristics of fine-grained soils. Alternatively, the engineering properties of clayey material can also be determined from one-dimensional consolidation tests (i.e. oedometer test) on undisturbed samples.

2.1.4 Adverse Rock Discontinuities

Where it is anticipated that part of the embedded wall would be installed in rock, GI should include a discontinuity survey using either the impression packer or borehole televiewer method. Depending on the results of the discontinuity survey conducted in the initial boreholes, additional boreholes and discontinuity survey may also be carried out to establish the presence of any persistent adverse rock joints that could affect the design of rock sockets and stability assessment of rock faces below the embedded wall. Adverse joint sets may be identified from concentrations of the pole density of different discontinuities in a stereoplot. Significant savings can be achieved if representative and realistic rock discontinuities are identified using suitable GI methods, as opposed to assuming the presence of the most persistent adverse discontinuities within the rock mass in the design (Cheung et al, 2023).

2.1.5 Condition Survey of Existing Ground Conditions

In urban areas, it is common to have many utilities laid within or adjacent to a development site. The space for accommodating these underground utilities is usually congested and frequent trench excavations and backfilling may result in the presence of loose fill layers surrounding them. Where utilities include water-carrying services, it is plausible that some leakage may have taken place over time and voids could occur when fine materials are washed away by water seeping through the soil mass. Sinkholes are sometimes reported in cases where soil arching over the void collapses. The excavation and associated installation of the embedded wall could then aggravate the problem of sinkhole formation and excessive settlement.

GEO (2023b) has concluded a review of these incidents associated with deep excavation and documented the common contributory factors. In conducting the desk study for the excavation works, the existing conditions of the underground water-carrying services and buried drains should be established. Relevant government departments, including the Water Supplies Department, Drainage Services Department and Highways Department, should be approached for records of any reported pipe bursting or leakage incidents. Such records may indicate the possible presence of voids in the fill layer. It is now common practice for a Closed Circuit Television (CCTV) survey to be conducted as part of the precondition survey of a site, so as to assess the condition of existing underground drains.

In addition to ascertaining the conditions of existing utilities, GI should also include measures to identify the presence of any voids, especially if loose fill layers, a high groundwater table and buried water-carrying services are present. Ground penetration radar (GPR) may be used to detect the presence of voids at shallow depth. Lai et al (2018) reported a blind test using GPR to detect predetermined underground voids. The investigation concluded that GPR was effective in detecting voids given careful application of de-noise, signal filtering and function gains by the commercial operators. However, GPR is less reliable in the detection of water-filled voids, as the signals are blurred by the dielectric properties of the groundwater. Alternatively, GCO probing may be used to detect the presence of pre-existing cavities in shallow fill materials.

2.2 Selection of Geotechnical Parameters

Guidance on the determination and evaluation of relevant geotechnical parameters is given in Geoguide 1. Nevertheless, it is of paramount importance that engineering judgement and experience should always be exercised in the determination of the geotechnical parameters for excavation design. The determination of selected values should take into account the following major factors:

(a) Quality of GI works (e.g. quantity and quality of soil samples and in-situ test data);

(b) Scale and duration of the excavation works (e.g. deep excavation encountering highly variable rockhead and long duration of dewatering works in association with consolidation of clayey material); and

(c) Drainage conditions for the excavation (e.g. drained or undrained conditions).

The selected values of geotechnical parameters for the design of ELS works should be based on suitable estimates that best represent the deformation performance of the works. At the construction stage, the performance of ELS works should be checked by measuring the actual deformation and comparing it against the estimated deformation. An effective I&M plan should be developed to initiate remedial and strengthening measures in cases when the measured values approach the trigger limits and stakeholders should be consulted on the plan. This provides the first line of safeguard for ELS works. More guidance on I&M is given in Chapter 10.

2.2.1 Design for Drained and Undrained Conditions

The circumstances and considerations to determine whether drained or undrained conditions applying in ELS design depend upon the speed with which the drained condition is achieved (Gaba et al, 2017). Drained conditions should be considered to apply if the rate of loading and unloading is sufficiently slow relative to soil permeability such that no significant excess porewater pressures are generated. In Hong Kong, sandy deposits and saprolite are generally permeable soils that behave as completely drained during excavation works. In contrast, the undrained condition applies to soil strata predominately containing silty and clayey soils (e.g. marine and alluvial clay) which has a much lower soil permeability.

Depending on the scale and complexity of the development project, ELS works can take a few months to a few years. Therefore, the assessment of drained and undrained conditions should consider the rate of dissipation of excess porewater pressure over the entire excavation period, particularly for sites with a thick layer of silty or clayey materials and lengthy construction programme. The assessment should also take into account factors that could affect the hydrogeological conditions of the site, e.g. any ground improvement works completed in a reclamation that may affect the groundwater conditions, such as installation of vertical band drains and deep cement mixing columns, as well as the presence of perched or confined aquifers between layers of transported soils and sources of water recharge. In practice, designs are carried out assuming either the undrained or drained condition and it is seldom necessary to consider the transient stage between these two conditions unless such a stage is critical.

2.2.2 Soil Shear Strength

In general, the Mohr-Coulomb strength model with effective stress parameters of apparent cohesion, c' and shear resistance, ϕ' of soil is commonly adopted. Consolidated triaxial compression tests are normally used to determine the shear strength parameters. Two types of triaxial compression tests are commonly carried out, namely the consolidated undrained (CU) test with porewater pressure measurement and the consolidated drained (CD) test with measurement of volume change. In general, total stress parameters (including the undrained shear strength) of the saturated specimen (at a given initial moisture content or void ratio), corresponding to a known initial effective stress, and the porewater pressure changes during shearing can be obtained in the CU test, from which the effective stress parameters can be determined. In a CD test, the drained shear strength of the saturated specimen and the volume change characteristics during shearing can be obtained. The testing time for both tests is also governed by the maximum allowable rate of axial displacement and the soil permeability, as specified in Geospec 3 (GEO, 2017b).

In determining the Mohr-Coulomb shear strength parameters, consideration should be given to the relevant design stress range where the parameters are obtained in both CU and CD tests. The values of the parameters should also be assumed constant within the range of stresses for which they have been evaluated. In the CD test, the maximum shear strength obtained depends on the confining stress specified and the magnitude of confining stress will affect the dilation angle of the shear strength parameters. The derived shear strength parameters obtained in CU and CD tests could be different, but the difference is usually not significant for the design of ELS works.

For undrained conditions, the shear strength of soil can be expressed in terms of total stress by the undrained shear strength, s_u. Laboratory undrained triaxial compression (UU) tests can be used to determine s_u values. However, the soil samples should be consolidated to the appropriate confining stresses and in-situ stress state as recommended in Geoguide 1. The s_u of clayey soils determined from UU tests may not be representative due to possible disturbance and de-saturation of soil samples during sample collection and testing (GEO, 2017b). On the contrary, in-situ field tests, such as the CPT and vane shear test (e.g. Robertson & Campanella, 1983), may give a more reliable estimate of s_u values for clayey materials. General guidance on performing CPT in Hong Kong is given in Geoguide 2.

Evans (1995) reported that most of the Holocene marine clays in Hong Kong are normally consolidated and their empirical values of s_u/σ_v' generally vary from 0.22 to 0.39, where σ_v' is the effective overburden stress. However, this range of values may not be applicable when the degree of consolidation is less than 95%. In the absence of a detailed consolidation assessment and past settlement records, the Code of Practice for Foundations 2017 (BD, 2017) gives some practical and pragmatic recommendations for estimating the completion of consolidation based on the age of reclamation in years and thickness of the clayey layers. In addition, where sensitive clay is identified, the value of s_u could be significantly reduced from peak values if the micro-structure of the clay is disturbed by site activities, such as the formation of mud waves, piling operations and wall installation. In such cases, adoption of the peak value of s_u may not be appropriate and the sensitivity of clay should be considered in design. For over-consolidated clays (e.g. Pleistocene alluvial clays), site-specific representative field tests calibrated with laboratory tests can be adopted to determine the empirical correlation of s_u/σ_v'.

CIRIA C143 (Clayton, 1995) refers to other empirical correlations of s_u with in-situ SPT 'N' values for soils with different plasticity. The selection of appropriate empirical correlations should be based on reliable case histories with similar ground conditions.

2.2.3 Soil Stiffness

Soil stiffness is the key geotechnical parameter needed to estimate ground deformation associated with ELS works. GEO (2020) discussed methods for obtaining soil stiffness parameters and the factors that influence their selection. For example, relatively high soil stiffness would be expected for sites with substantial overburden removed at the site formation stage. The stress-strain behaviour of soil is generally non-linear. However, it is often convenient in design to assume a linear or log-linear relationship between stress and strain for soil behaviour within a limited range of stress and strain. Also, soil stiffness generally varies with the strain level and therefore the depth of excavation.

The prevailing practice is generally to adopt a linear elastic-perfectly-plastic model for soil, with the Young's moduli at working strain level correlated with in-situ test results such as SPT 'N' values:

$$E' = f \times N \quad \text{(2.1)}$$

where E' = Young's modulus for the drained condition (in MPa)
N = SPT 'N' value
f = Correlation factor

The above correlation has been widely adopted in local practice and the correlation factor (f) has been derived from back-analyses of well instrumented and reliable published case histories of wall deflection (e.g. Lui & Yau, 1995; Chan, 2003; Wong, 2013). Based on local practice and experience, f is generally in a range of 1.0 to 1.5 for fill and alluvium and 1.5 to 2 for completely and highly decomposed rock (e.g. saprolite). However, where loose soils or soft soil with a high fines content is encountered, the correlation factor is usually taken as unity.

Apart from the correlations of soil stiffness with SPT 'N' value for drained conditions, some other commonly-used empirical relationships to obtain Young's modulus for the undrained condition (E_u) by correlation with s_u, plasticity index (PI) and over-consolidation ratio (OCR) can be found in Duncan & Buchignani (1976) and Jamiolkowski et al (1979) (e.g. E_u/s_u varies from 300 to 600 for values of PI between 30% and 50% and OCR less than 2).

Alternatively, nonlinear stress-strain behaviour can be directly simulated using various constitutive models. Special in-situ and laboratory tests, e.g. pressuremeter tests, field geophysical tests and triaxial tests with local strain gauges attached directly to the sample (Jardine et al, 1984), are often used to determine the soil parameters.