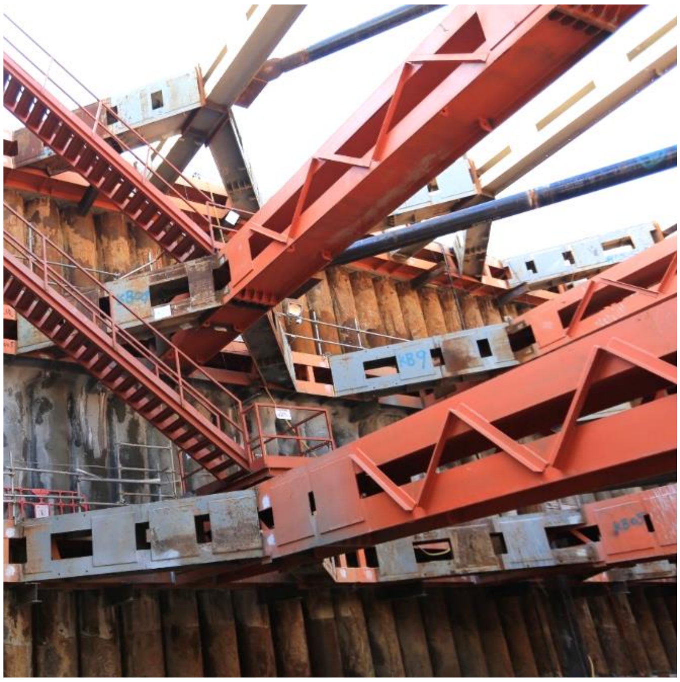
GEO Publication No. 1/2023

Deep Excavation Design and Construction




Geotechnical Engineering Office
Civil Engineering and Development Department
The Government of the Hong Kong
Special Administrative Region

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Cover Photographs:

Top Left:	Excavation using an Interlocking Pipe Pile Wall for the Central Kowloon Route Project
Top Right:	Excavation using a Tied-back Wall for a Residential Development Project at Stubbs Road
Bottom Left:	Excavation using a Diaphragm Wall for a Kai Tak Development Project
Bottom Right:	Excavation using a Multi-cell Circular Shaft for the Cut-and-cover Tunnel Construction under the Tuen Mun-Chek Lap Kok Link Project

Foreword

The GCO Publication No. 1/90 - Review of Design Methods for Excavations was published in 1990. Although the review was written primarily as a reference document, upon publication it immediately became the *de facto* standard of good practice for design of excavation and lateral support (ELS) works in Hong Kong. Since then, practitioners have accumulated much experience in the design and construction of deep excavations in Hong Kong. There have also been many advances in design methods and modelling techniques, as well as an increasing trend in international practice of using the partial factor method for limit state design. On the other hand, the construction industry has introduced modern equipment and invested heavily in digital construction in recent years.

In view of these developments, the GEO saw the need to revise Publication No. 1/90 to consolidate the experience gained and the improvements made in the practice of ELS works. The scope of the publication has also been expanded to its present form to cover the key aspects relating to the construction of ELS works. Hence, a new publication title is used. The recommendations given in this new publication aim to achieve significant gains for the economic design of ELS works, savings in construction time and enhancements in ground deformation monitoring and control.

This publication was prepared under the overall direction of a Working Group. The membership of the Working Group, given on the next page, includes representatives from the relevant government departments, the Hong Kong Institution of Engineers and the Piling Contractors Committee of the Hong Kong Construction Association. In 2021, the Geotechnical Division of the Hong Kong Institution of Engineers produced a technical review report on the latest practice of geotechnical design and construction of ELS works, which identified key technical areas deserving consideration in the drafting of this new publication. Copies of draft version of this publication were circulated to local professional bodies, consulting engineers, contractors, academics and government departments with experience in the field of ELS works. Numerous individuals and organisations made useful comments, many of which have been adopted in finalising this document. Their contributions are gratefully acknowledged.

Given the variable nature of the geological and hydrogeological conditions in Hong Kong, geotechnical design is always carried out under certain assumptions and simplifications. It is therefore important to collect field data for verifying the assumptions made and to confirm the performance of geotechnical works. For this reason, field monitoring data such as ground settlement related to ELS works have been collected and presented in this publication. Practitioners are encouraged to help expand this database by undertaking purposeful instrumentation and monitoring of ELS works and providing us the relevant field data and details of the works. This information can be provided to the Technical Secretary of the GEO.

Practitioners are encouraged to provide comments on the contents of the publication to the GEO at any time, so that improvements can be made in future editions.

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December 2023

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1 Introduction

1.1 Purpose and Scope

Deep excavations are common in the urban area of Hong Kong and often adopted for construction of multi-level basements, railway and road tunnels. Excavation in such environments invariably presents challenges to planners, engineers and contractors, as there are always nearby buildings, structures and services to be protected. The consequences of any excavation collapse could be serious and costly. For this reason, excavation in soil generally requires sufficient lateral support to minimise any adverse effect on the surrounding environment.

The GCO Publication No. 1/90 "Review of Design Methods for Excavations" (GEO, 1990), published in 1990, gave the then state-of-the-art methods for the design of excavation support systems, as well as the prediction of ground deformation around excavations. The publication has been widely used by practitioners as a reference document for excavation design in Hong Kong. Since then, there have been many advances in the knowledge, technology and modern methods for designing and executing excavation and lateral support (ELS) works. In view of these developments, a Working Group on Revision of GCO Publication No. 1/90 was set up to update the publication. The purpose of this document is to give guidance for the design and construction of ELS works in Hong Kong, taking cognizance of latest advancements in pertinent subjects. This document also consolidates local practice and experience in the construction of ELS works in Hong Kong and provides recommendations for mitigating the geotechnical risks associated with excavations. The publication is intended for use by readers who have some general knowledge of ELS works.

New permanent earth retaining walls are not covered by this document and readers should refer to Geoguide 1: Guide to Retaining Wall (GEO, 2020) published by the GEO.

1.2 Overview

The content of this document is intended to cover the execution of temporary excavation works on land using embedded wall that facilitates the construction of underground structures. In this document, the term "excavation" is applied generally to cover all ELS works, whereas the term "deep excavation" refers to excavations deeper than 4.5 m, in conformity with the current distinction for the enhanced statutory control of ELS works under the Buildings Ordinance (PNAP APP-57) (BD, 2012).

Good practice for site investigation and selection of geotechnical parameters that are crucial for the design of ELS works and associated key considerations are presented in Chapter 2.

A review of common types of excavation support systems and technical considerations for the design and construction of ELS works are discussed in Chapters 3, 4 and 5. In particular, the discussions given in Chapter 5 highlight the construction aspects that should be carefully assessed and considered in the execution of ELS works.

Chapter 6 discusses the limit state design of ELS works. It outlines relevant requirements for checking the ultimate limit state (ULS) and serviceability limit state (SLS). The application of global factor and partial factor methods in limit state design are also discussed, along with recommended factors to be used in both methods. Methods for ULS and SLS design are presented in Chapters 7 and 8, respectively.

Chapter 9 discusses mechanisms for the control of ground deformation due to ELS works, including the recommended empirical and engineering approaches and the trigger values for initiating response actions to minimise adverse impact on sensitive receivers.

Adequate instrumentation and monitoring (I&M) are essential for the safe execution of ELS works. Chapter 10 sets out the considerations necessary to formulate an appropriate I&M plan and introduces new technology that could promote further advancement in local practices.

1.3 General Guidance

In this document, reference has been made to published codes, reference papers, textbooks, and other relevant information. Readers are strongly advised to consult the original publications for full details of any particular subject and to consider the appropriateness of using such methods in the design and construction of ELS works.

The various stages of site investigation, design and construction of ELS works require coordinated input from experienced designers and contractors. Continuous involvement of the designer of the ELS works is essential for verifying both the validity of the assumed ground conditions and the expected performance of the ELS works. The installation methods used to construct the embedded wall may significantly affect the performance of the ELS works, and the subsequent works that require strict adherence to agreed procedures. It is important that competent specialist contractors are employed and their works should be adequately supervised by suitably qualified and experienced engineers who are familiar with the design.

In common with all types of geotechnical works, professional judgement and engineering common sense must be exercised when designing and executing ELS works.

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.

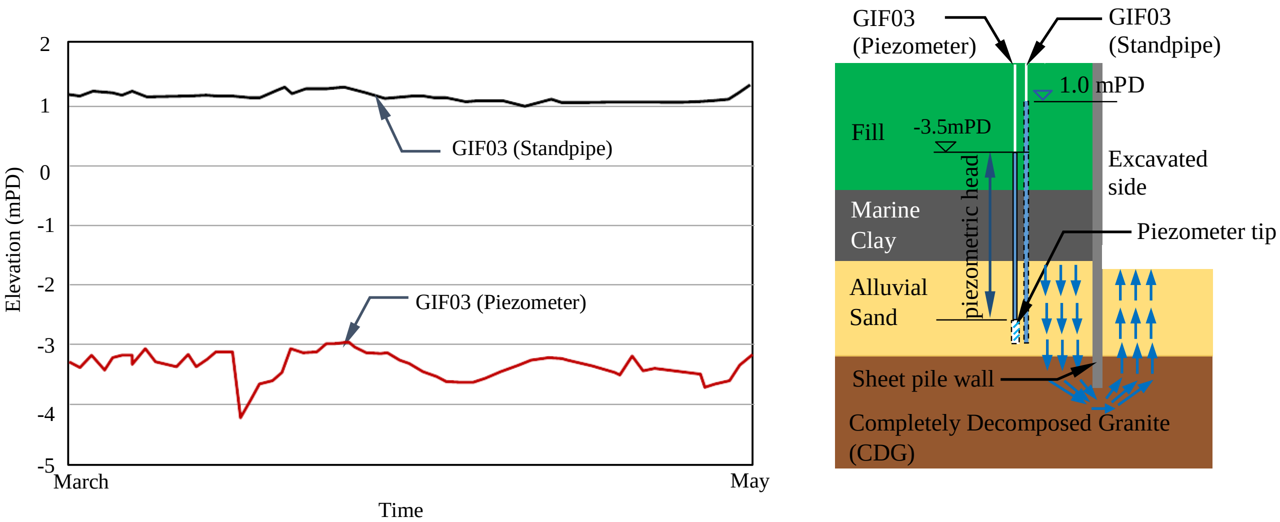


Figure 2.1 Variation of Groundwater Pressure in a Confined Aquifer

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:

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

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.

3 Excavation Support Systems

3.1 General

Excavations for building developments and civil engineering works often require ELS works to support the adjoining ground and construct the substructures.

The following types of embedded wall are commonly used in Hong Kong to support excavations:

(a) Channel planking wall;

(b) Sheet pile wall;

(c) Soldier pile wall;

(d) Pipe pile wall;

(e) Bored pile wall; and

(f) Diaphragm wall.

The types of excavation support systems can be divided into the following four major categories according to the form of support provided:

(a) Cantilevered wall;

(b) Strutted wall;

(c) Tied-back wall; and

(d) Circular shaft.

3.2 Types of Embedded Wall

3.2.1 Channel Planking Wall

A channel planking wall comprises steel channel sections driven, pressed or vibrated into loose to medium dense soils (e.g. SPT 'N' values less than 30). The commonly used section sizes range from 150 mm (depth) × 90 mm (width) to 300 mm (depth) × 100 mm (width). A typical channel planking wall arrangement is shown in Figure 3.1. There is no interlocking between the channel sections, and therefore this wall type does not provide water tightness for the excavation. The steel channel sections are very often driven into the ground in a group of welded sections. Site welding at joints would normally be carried out on the excavated side in order to minimise water seepage for excavations in ground with a high groundwater table. The stiffness and moment capacity of the channel sections are usually small, and therefore a channel planking wall is suitable for shallow excavations of generally less than 4 m in depth.

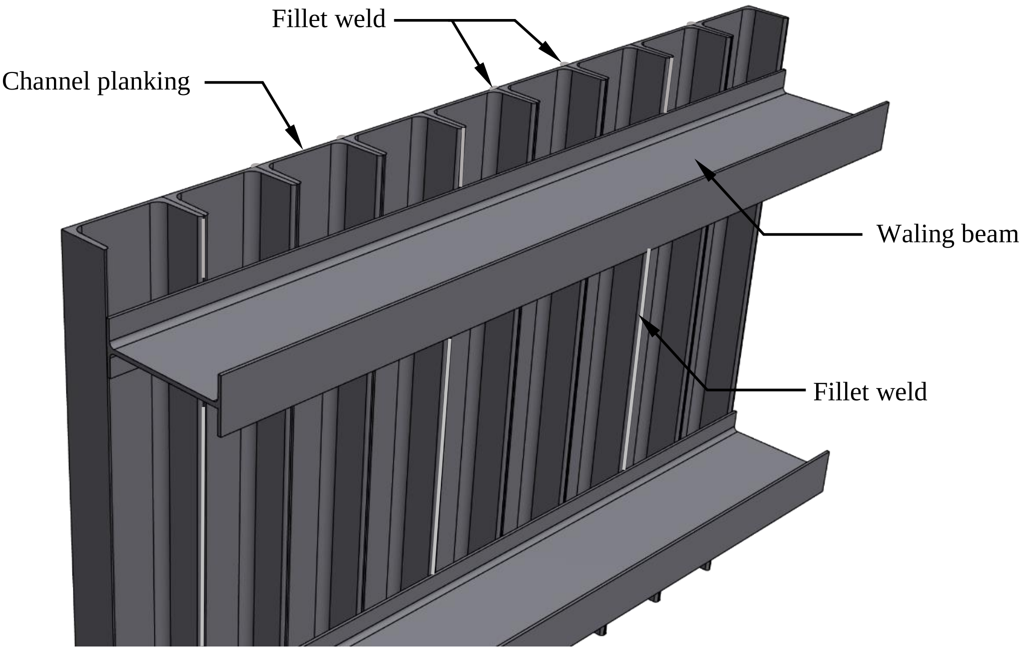


Figure 3.1 Typical Channel Planking Wall

3.2.2 Sheet Pile Wall

A steel sheet pile wall is the most common type of embedded wall used in Hong Kong. A typical sheet pile wall arrangement is shown in Figure 3.2. Sheet piles are relatively flat and wide in cross-section such that they can be installed side by side to form a continuous wall with interlocks which generally provide reasonably good water tightness.

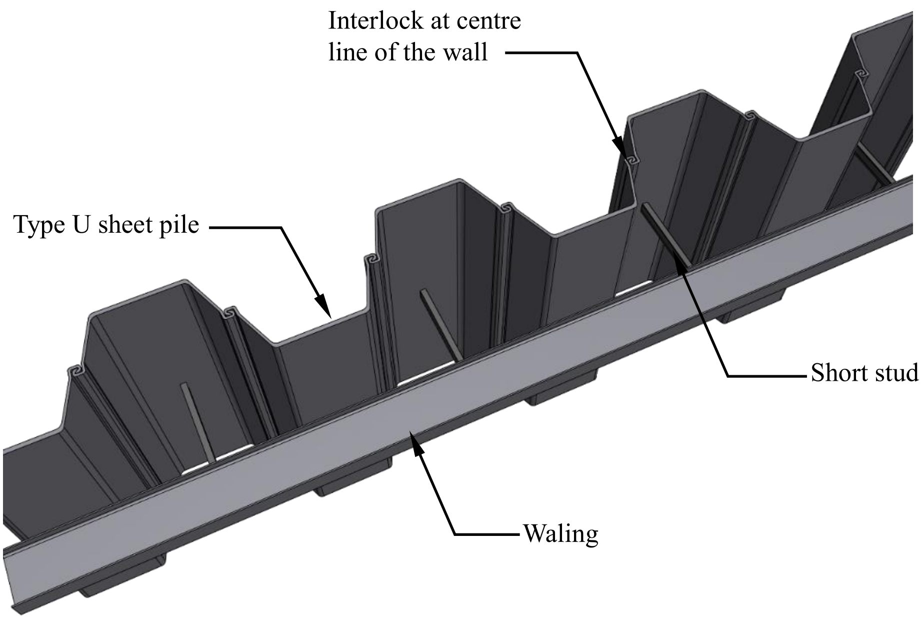


Figure 3.2 Typical Type U Sheet Pile Wall

The common shapes of sheet pile sections include Type U and Type Z, with different positions of the interlocks (Figures 3.2 and 3.3). There are many sectional types of Type U sheet piles to suit varying space and strength requirements. Common sizes range from 400 mm to 500 mm in width and 100 mm to 200 mm in depth. Type Z sheet piles have a typical width of about 700 mm and depth of about 500 mm.

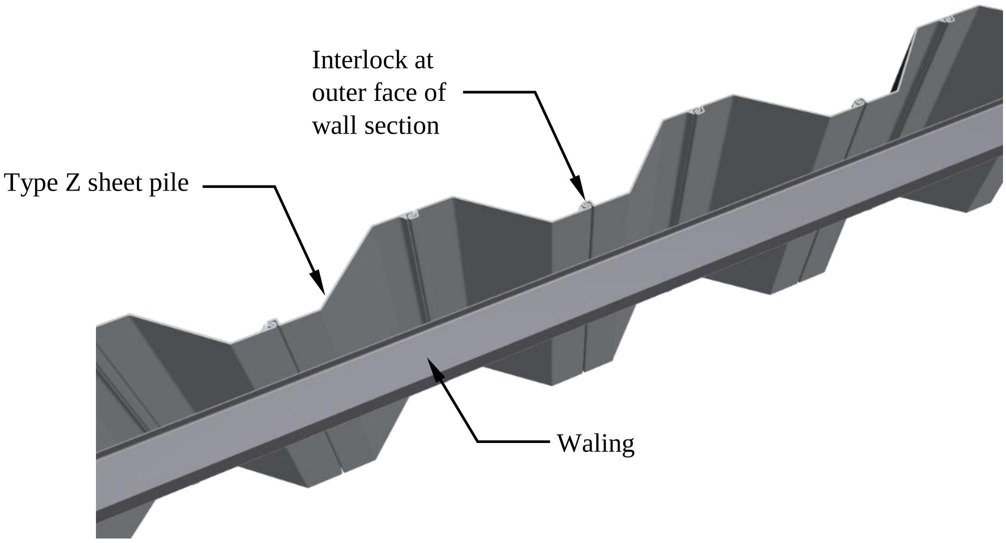


Figure 3.3 Typical Type Z Sheet Pile Wall

Type U sections are more widely used in Hong Kong because of the ease of stacking, driving and transportation. The typical excavation depth using a Type U sheet pile ranges from 3 m to 15 m. However, the interlocks are located at the centre line of the sheet pile wall where maximum shear stress develops. A reduced bending stiffness and moment capacity of the connected Type U sheet piles is usually adopted to allow for slippage at the interlocks. The reduction factors are discussed in CIRIA Special Publication 95 (Williams & Waite, 1993), CIRIA C760 (Gaba et al, 2017) and Eurocode 3 (BSI, 2007). In this connection, a Type Z sheet pile wall has interlocks at the outer and inner face of the section and is more effective in resisting bending moment as combined sections. In addition, a Type Z sheet pile wall has a greater moment of inertia and sectional modulus than that formed using Type U sheet piles with the same mass of material. Hence, Type Z sheet piles are becoming more commonly used in deep excavation projects in Hong Kong. However, driving a single Type Z sheet pile section requires better control of wall alignment as compared to a Type U section and it is common practice to install Type Z sections in doubly clutched sheets with a larger clamp for wall installation.

Sheet pile walls are usually installed by driving, vibration or pressing the steel sections into the ground. When selecting the profile and section size of the sheet piles, it is important to consider their drivability and penetrability in the anticipated ground conditions.

In general, where the ground is mainly comprised of loose to medium dense granular soils (e.g. SPT ‘N’ values less than 30), it is common to install the sheet piles by vibration. It is difficult to use the vibration method to install sheet piles through dense soil (e.g. SPT ‘N’ values larger than 30), even with heavier sections. Moreover, vibration may induce ground settlement during pile installation or extraction, especially where the ground comprises loose sandy soils or rock fills. For very dense soils with SPT ‘N’ values up to 120, a hydraulic or drop hammer may be used to drive sheet piles through dense soil strata. When hard driving is anticipated, heavier sheet pile sections are required to sustain the driving force. Significant noise and vibration will be generated when a heavy hammer is used for driving.

Obstructions in the ground (e.g. corestones, boulders, existing pile caps and basement, and old seawalls) can pose difficulties to proper installation of sheet piles by driving. Inadequate penetration of sheet piles is one of the common causes of excavation collapse in Hong Kong (GEO, 2002). Hard driving of sheet piles through such obstructions should be avoided, as it may damage the pile sections and cause declutching that affects the water tightness of the sheet pile wall. Pre-boring to overcome underground obstructions and enhanced site supervision are usually adopted to ensure proper installation of sheet piles to the intended depth. Extra space may be needed to allow for the pre-boring works when planning the alignment of a sheet pile wall close to the site boundary.

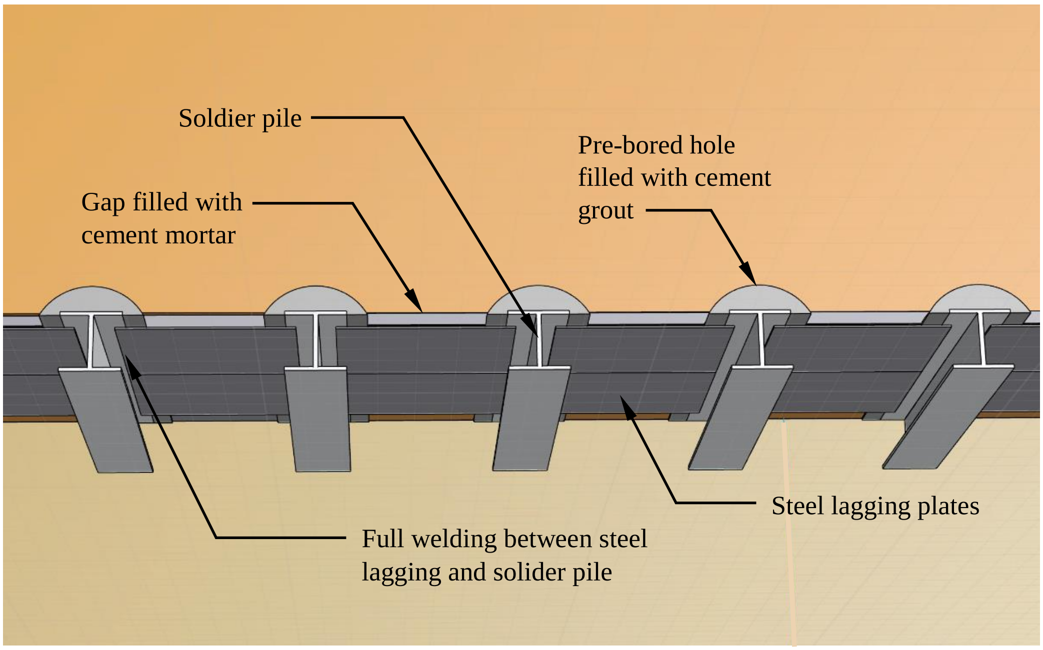
As sheet piles can be reused, they should be inspected for any damage due to excessive wear and tear. Damaged interlocks may be declutched during driving and cause ground loss and ground settlement due to ingress of groundwater and loose soil. A sealant (e.g. material containing hydrophilic polyurethane or wood resin), can be applied at the interlocking joint to improve the water tightness of a sheet pile wall. However, it will also reduce the wall stiffness and bending moment capacity. In some cases, site welding at the joints is carried out to prevent water seepage, but this will make subsequent extraction of the sheet piles difficult. Alternatively, a precautionary grout curtain wall may be installed behind the sheet pile wall in case there is a potential issue of quality of interlocks where the wall is installed in the ground with a high groundwater table.

The press-in method of installation may also be adopted so as to reduce the noise and vibration generated when compared with other installation methods. However, the penetration of a press-in sheet pile wall is generally limited to loose to medium dense soils with SPT ‘N’ values of around 30 or less. The press-in method may be used in conjunction with water jetting or an auger to install sheet piles in harder strata. The use of water jetting may increase potential ground loss surrounding the sheet piles and lead to excessive ground settlement. Thus, water jetting should be used with caution and a tight supervision and monitoring scheme should be implemented.

3.2.3 Soldier Pile Wall

A soldier pile wall consists of embedded piles with horizontal lagging spanning between them to retain the soil. Steel H-sections or I-sections are often used as the soldier piles, which provide intermittent vertical support and are installed before the commencement of excavation commences. Typical arrangements of soldier pile walls with steel lagging are shown in Figure 3.4. The common size of steel section ranges from 305 mm to 610 mm in depth. The spacing of solider piles should be determined based on the proximity of any structures and utilities and the soil arching effect. In competent soils, a relatively wide spacing of not more than three times the width of the steel sections could be adopted (GEO, 2020). However, it should be cautioned that closer spacing is required where the ground condition comprises loose fill and a high groundwater table near the surface, which is often the case in urban sites.

Soldier piles are either driven or placed in pre-bored holes, which are backfilled with grout or lean concrete, in deep excavations generally up to 20 m. Similar to sheet piles, soldier piles can be driven into soil for faster installation. However, pre-boring is often required and preferred to overcome underground obstructions, such as boulders or old seawall masonry blocks, especially in urban areas, and to minimise the vibration and noise arising from pile driving. In addition, driving of steel sections may cause heave and settlement of nearby buildings/structures/services (D'Appolonia, 1971) which should be duly considered in selecting the appropriate embedded wall system.



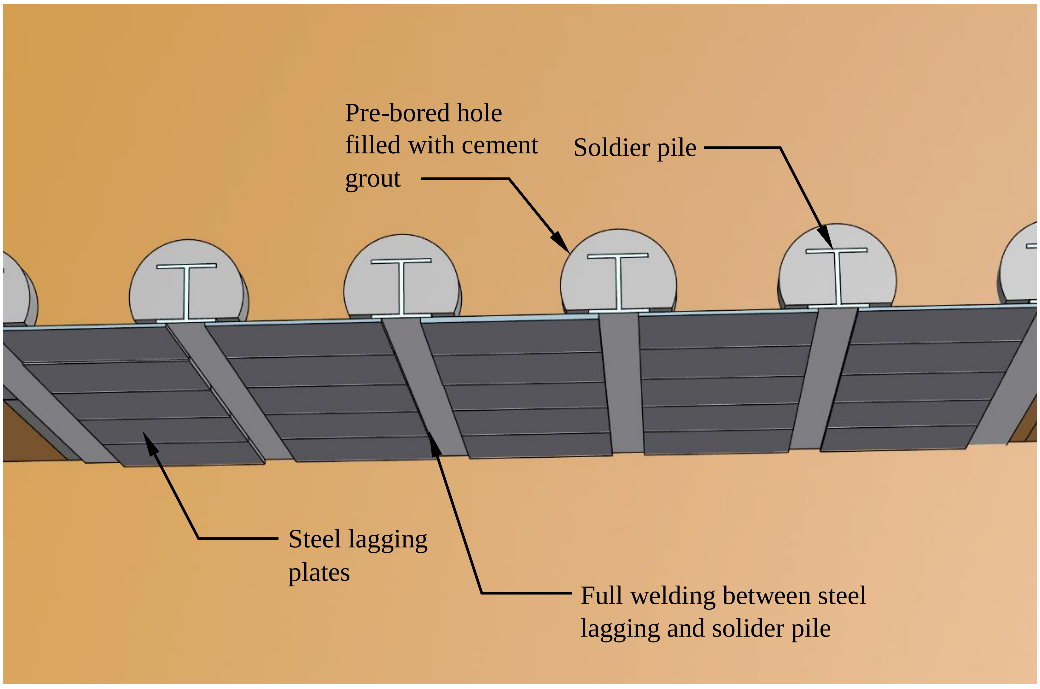


Figure 3.4 Typical Soldier Pile Wall with Steel Lagging

Steel plates or channel sections connected by welding to the soldier piles, or shotcrete with wire mesh, are usually adopted as lagging to support the soil face and prevent progressive deterioration of the soil arching effect between the piles. Lagging is often installed in lifts of 1.0 m to 1.5 m, depending on the strength of the retained soil and the groundwater conditions.

Soldier pile walls are well suited to competent ground, typically dense soils (e.g. SPT ‘N’ values greater than 30), with a low groundwater table. When the base of the excavation is below the groundwater level, a grout curtain wall is commonly provided to minimise water seepage into the excavation. On the contrary, where there is a concern about the build-up of groundwater pressure behind the lagging, drainage holes are provided at appropriate levels to maintain the groundwater level, or to lower it if drawdown is permitted. In such cases, filter materials such as synthetic fabrics may be used to prevent loss of soil behind the wall.

3.2.4 Pipe Pile Wall

A pipe pile wall is similar to a solider pile wall, except that the steel casing used in the pre-boring works is also used as the vertical element to the excavation. Sometimes, steel H sections are inserted inside the casing to increase the bending stiffness and capacity of the embedded wall. Pipe pile walls are commonly used for excavations more than 10 m deep, with typical casing sizes in Hong Kong ranging from 219 mm to 813 mm in outer diameter. The spacing of the pipe piles should be determined based on similar considerations as for a solider pile wall. Steel lagging is installed between the pipe piles to retain the excavated soil face and prevent progressive deterioration of the soil arching between the piles, especially under the groundwater table. In such cases, the grout curtain is often provided on the retained side of the pipe pile wall for water tightness before the commencement of excavation. A typical pipe pile wall with a grout curtain is illustrated in Figure 3.5.

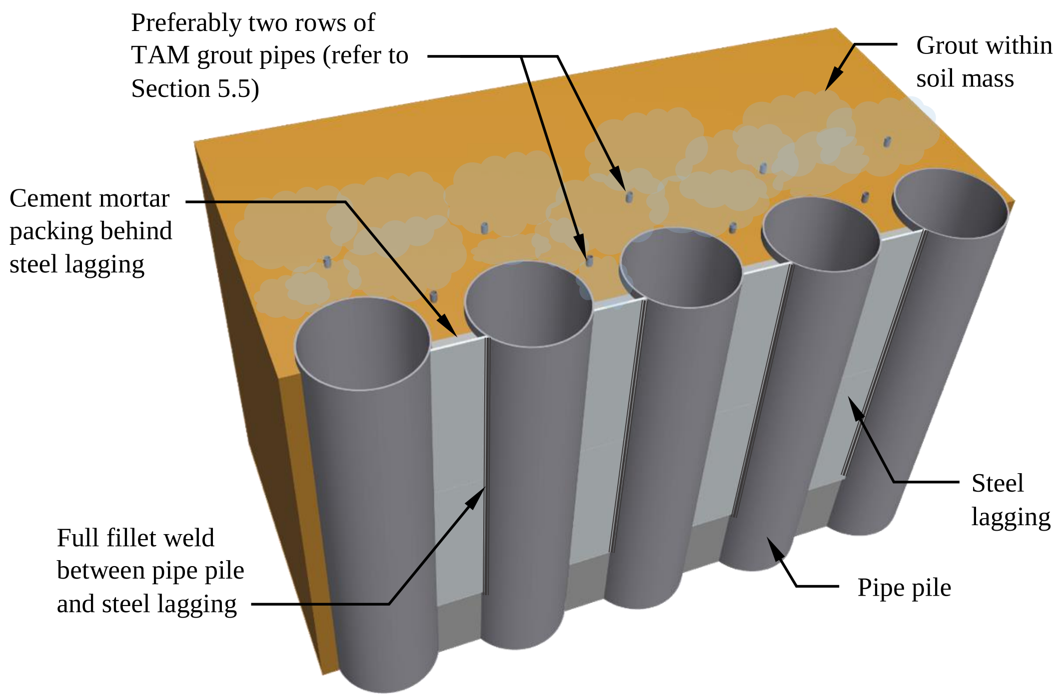


Figure 3.5 Typical Pipe Pile Wall with Grout Curtain

Interlocking pipe piles as shown in Figure 3.6 provide a reasonably good water cut-off and are usually regarded as an impermeable embedded wall. The interlocking joints act as a physical barrier against groundwater seepage, although minor seepage through the joints may still occur if the difference in hydraulic head is significant. In such cases, sealant can be used in the interlocking joints to further enhance water tightness (Li et al, 2018).

Due to their water tightness, interlocking pipe piles have been gaining popularity in recent excavation projects in Hong Kong. A few ELS projects (e.g. the Central Kowloon Route crossing Kowloon Bay, and the Lyric Theatre in Tsim Sha Tsui) using interlocking pipe pile walls have been successfully executed and the walls proved to be effective in providing a groundwater cut-off barrier. Interlocking pipe pile walls also have the advantage of readily overcoming underground obstructions. However, particular attention is needed when interlocking pipe piles are being advanced through mixed ground conditions (e.g. saprolite with corestones), as even a small deviation of the wall alignment or installation tolerance could cause interruption or clashing of the piles at depth. Retraction and reinstallation of the clashed pipe piles may cause significant ground loss and hence induce undue ground deformation in the vicinity. An oversize pre-bored hole is also required to accommodate the interlocks.

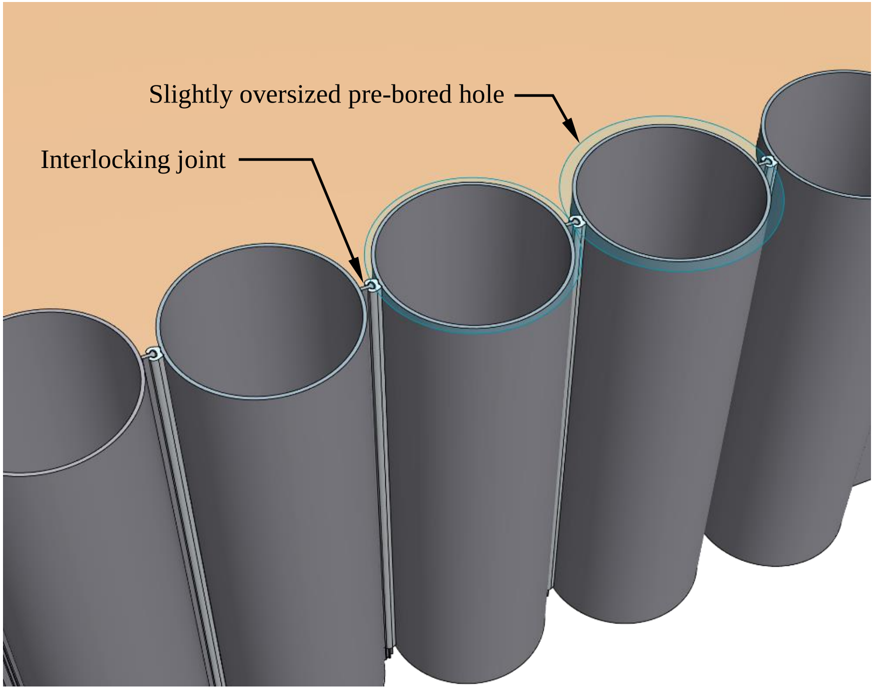


Figure 3.6 Typical Interlocking Pipe Pile Wall

3.2.5 Bored Pile Wall

Bored pile walls generally have a relatively large wall stiffness and are better in limiting wall deflection during excavation as compared with the foregoing wall types. They are usually used in excavations more than 15 m deep. A bored pile wall is formed by a row of either contiguous bored piles or secant bored piles constructed along the periphery of the excavation. Contiguous piles (Figure 3.7) do not intersect with each other and the gaps between piles are typically around 100 mm to 500 mm to allow for construction tolerance and avoid overlapping at depth, although the space between the piles can be larger in more competent soils. The wall tolerance should be specified based on the type of installation method, as well as deflections occurring during wall installation. A grout curtain is commonly used to prevent water seepage between the gaps during excavation. The bored pile wall is usually used as a permanent structure, with a secondary internal wall tied to the bored pile wall in order to improve water tightness and provide a wall surface with a better aesthetic finish.

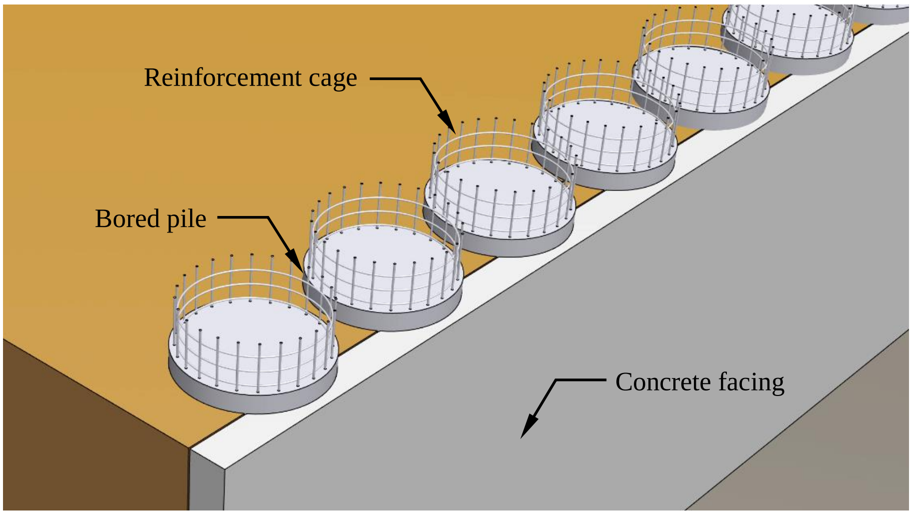


Figure 3.7 Typical Contiguous Bored Pile Wall

On the contrary, secant bored piles (Figure 3.8) are concrete piles that are cast in-situ but overlap with a greater contact between adjacent piles. The piles are usually arranged as alternate soft piles and hard piles. The soft piles are commonly formed as bored piles with weaker plain concrete, although jet grouting has also been used in individual projects. The hard piles, cutting into the soft piles when bored, are the main structural elements and are properly reinforced. The soft piles are intended to prevent water seepage by supporting the ground between the hard piles.

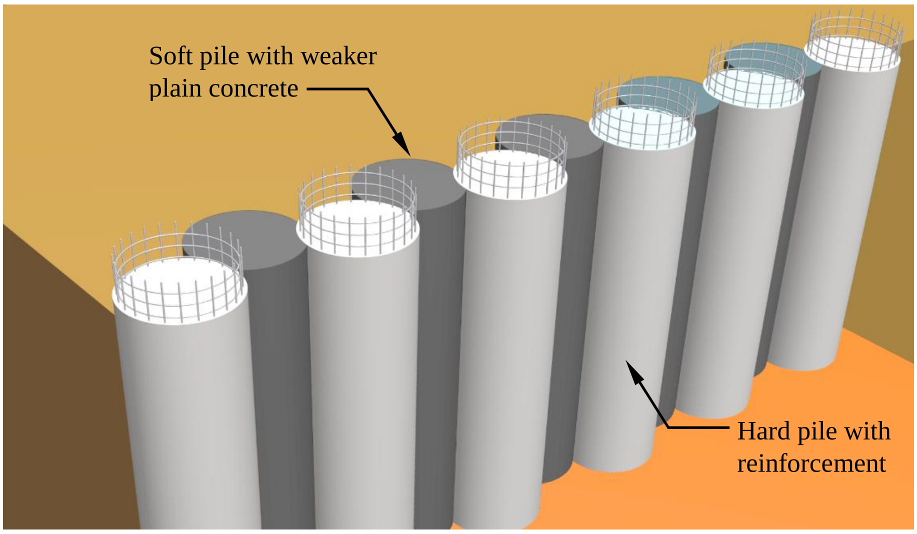


Figure 3.8 Typical Secant Bored Pile Wall

Bored piles are usually excavated by a high rotary table rig, or by grab and chisel within a steel casing, which is advanced progressively with the use of an oscillator or a rotator. Reverse circulation drilling (RCD) incorporating rock roller bits may be used when needed to penetrate rock or boulders (GEO, 2006). Bored pile shafts can also be excavated by means of a rotary auger or a rotary drilling bucket under a bentonite slurry which supports the sides of the excavated shaft (GEO, 2020). The considerations for using a bentonite slurry for excavation of a bored pile without temporary casing are similar to those for a diaphragm wall and are discussed in Chapter 5.

Where excavation is to be carried out beyond the casing, it should be supported by an excess water head or bentonite slurry. Alternatively, steel casing may be advanced below the excavation level to provide the support (GEO, 2006). Excavation ahead of the toe of temporary casing in loose soil strata may cause excessive ingress of groundwater and soil into the borehole, which is one of the probable causes of deep sinkholes. In loose soil strata and/or near sensitive structures, no excavation should be allowed unless an adequate toe-in of the temporary casing is achieved during bored piling operations.

Similar problems of ground instability may also arise where a steep rockhead is encountered during RCD to form a rock socket. In such cases, pre-grouting at the toe of temporary casing down to the rockhead may be considered. Where the deep rockhead is encountered, it may be difficult to install the grout holes in precise positions surrounding the bored pile. Alternatively, plugging of the toe with concrete or soil mixed polymer fluid (e.g. Supermud) could be used to enhance the stability of the empty bore as excavation proceeds.

3.2.6 Diaphragm Wall

A diaphragm wall is formed by a series of aligned discrete rectangular reinforced concrete panels (Figure 3.9). It was first introduced in Hong Kong in the development of the New World Centre (Tamaro, 1981). Since then, the technique was used extensively in the construction of the Mass Transit Railway (MTR) underground stations (e.g. Budge-Reid et al, 1984) and for the deep basements of high-rise buildings (e.g. Liu et al, 2010). A diaphragm wall is suitable for most ground conditions, except for very soft ground due to the high risk of instability in the trenches used to form the panels. Sometimes, pre-grouting may be required to improve the ground condition before the installation of a diaphragm wall.

Apart from rectangular panels, T-, Z- and I-shaped panels are sometimes used, which provide higher stiffness and larger moment capacity. However, construction of non-rectangular panels is relatively difficult and close site supervision is required (Fernie et al, 2012). They should be used with caution, as it is more difficult to maintain the trench stability.

Diaphragm wall panels commonly have a thickness ranging from 0.8 m to 1.5 m and length between 2.8 m and 6.4 m. Individual panels up to 6 m long are usually excavated by two or three bites of the excavating equipment. Besides strength considerations, selection of panel size should also consider constructability aspects, including adequate space for placing tremie pipes, reservation tubes and stop ends, and for concrete flow between steel reinforcement bars. Excavation and concreting of adjacent panels should be carried out in alternate panels sequentially (e.g. starting panels first, then closing panels), which permits soil arching to develop around the panel excavation.

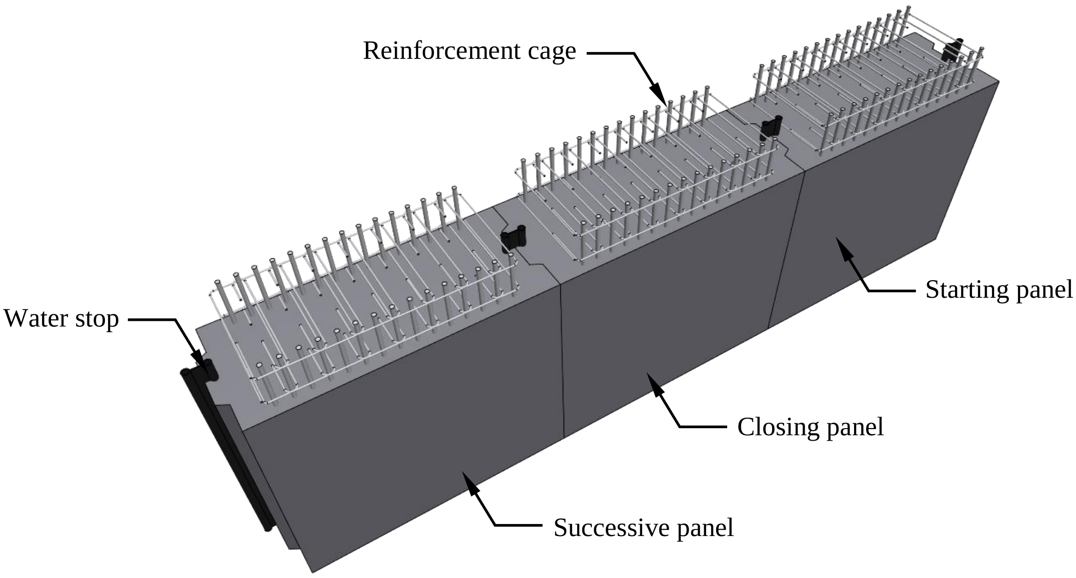


Figure 3.9 Typical Diaphragm Wall

Rectangular trenches are excavated with the use of grabs or hydromills (also called trench cutters or hydrofraises). As compared to grabs, hydromills are generally more powerful, efficient and versatile and can work continuously to lift the excavated material to the surface (Endicott, 2020). The reinforced concrete panels are usually cast in-situ in Hong Kong. Precast panels are seldom used due to construction difficulties in connecting the panels together on site.

A diaphragm wall generally provides good water tightness between cast in-situ panels. The commonly used joint systems in Hong Kong include vertically pulled and peel-off steel systems, which enable a vertical water stop to be adopted. Accurate placing of stop-ends is vital for the control of panel dimensions and water tightness at panel joints. Local experience in successful placing and removing such stop-ends is limited to about 50 m depth. If the adoption of stop-ends with a water stop is considered impractical, or the performance of a water stop is ineffective, grouting should be applied at and around the construction joints between the diaphragm wall panels.

Trench excavation of a diaphragm wall is usually supported by bentonite slurry. The hydrostatic pressure of the slurry should be controlled so that it is always greater than the combined water and earth pressures, with due allowance given for the soil arching effect (GEO, 2020). Further details of the construction considerations for a diaphragm wall are presented in Chapter 5.

A concrete capping beam is usually cast on top of the diaphragm wall panels, which allows more even load distribution on the diaphragm wall and hence reduces the differential wall deflection. For situations where part of a diaphragm wall is to be bored through or saw cut to create a wall opening (e.g. Tunnel Boring Machine launching or retrieval), the capping beam could also take up the self-weight of the hanging diaphragm wall panels above the wall opening.

Diaphragm walls founded on rock are usually designed to have a shallow rock embedment. Otherwise, larger power hydromills and a longer period of excavation will be needed for deep penetration in rock. Where excavation is extended below rockhead, shear pins are commonly installed and penetrated below the final excavation level in order to ensure the toe stability of the diaphragm wall.

Rock fissure grouting is often carried out below the toe of a diaphragm wall where it is necessary to control groundwater seepage along rock joints or other discontinuities into the excavation. Sze & Young (2003) described the construction of a deep basement for Chater House which involved the adoption of toe grouting in the form of chemical grout in soils and fissure grouting in rock down to a depth of 5 m into Grade III or better rock, in order to ensure the effectiveness of the water cut-off barrier and limit the drawdown of groundwater level outside the site. Steel reservation tubes can be provided in the reinforcement cage for toe grouting and installation of shear pins. In addition to lateral stability at the wall toe, vertical stability also needs to be considered as diaphragm wall panels impose large surcharge loading on vertical or inclined rock excavations. Field mapping of the exposed rock face and stability assessment should be carried out for different types of potential rock slope failures (e.g. plane mode, wedge mode and toppling mode), and appropriate stabilisation measures added if necessary (e.g. rock bolts/anchors).

3.3 Forms of Excavation Support

3.3.1 Cantilevered Wall

A cantilevered wall is a wall driven or bored to a depth considerably greater than the depth of excavation and derives its support from the resistance provided by the embedded section of the wall. It has a simple construction sequence with no strutting and can provide a large unobstructed area for construction of permanent structures within the excavation. Nevertheless, in order to reduce wall deflection, a light waling is sometimes installed to even out variations of ground pressures along the wall (Williams & Waite, 1993).

Cantilevered walls are generally limited to relatively shallow excavations. For deep excavations, the wall section requires much higher stiffness and bending moment capacity (e.g. as provided by a large diameter bored pile wall) in order to maintain the wall deflection and ground deformation within tolerable limits. As such, cantilevered walls are economical only for moderate retaining wall heights (usually less than 5 m), as the required stiffness and structural capacity of the wall increases rapidly with increase in retained height.

3.3.2 Strutted Wall

A strutted wall is the most popular system for deep excavation in Hong Kong and its layout should be planned with due consideration of ease of access for excavation plant, export of excavated materials and configuration of the permanent works. Different strutting systems, including the cross-lot and raking strut systems, are shown in Figures 3.10 and 3.11. Pre-loading may be applied to struts to reduce wall deflection, and thereby the associated ground settlement. However, wall deflection during preloading may affect the water tightness of completed grout curtains. Strut removal can also give rise to additional wall deflection and the construction sequence should be carefully planned. The space between the permanent structure and the embedded wall should be properly backfilled and compacted.

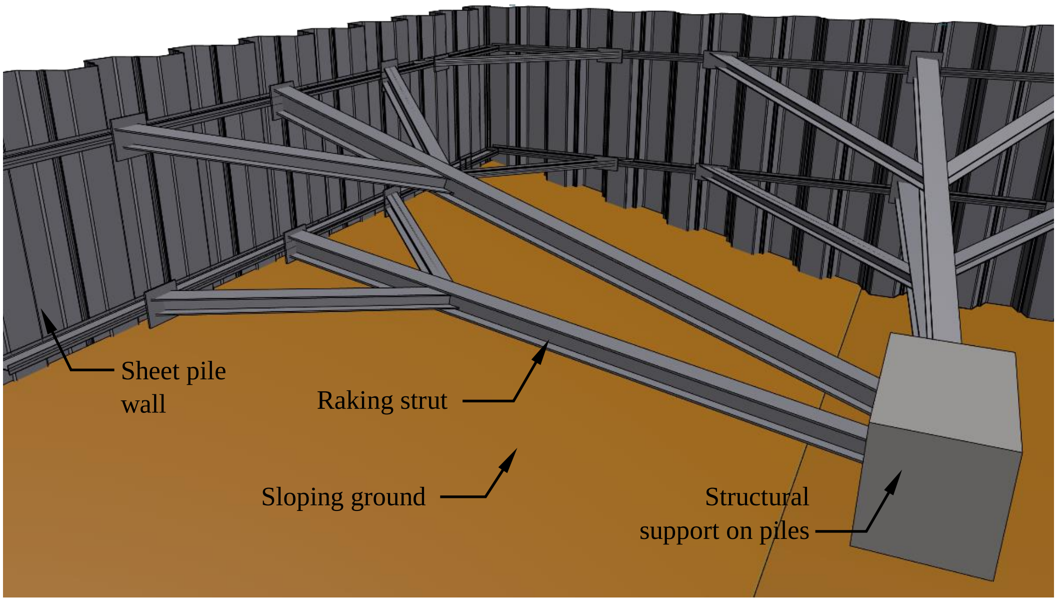


Figure 3.10 Strutting System with Raking Struts

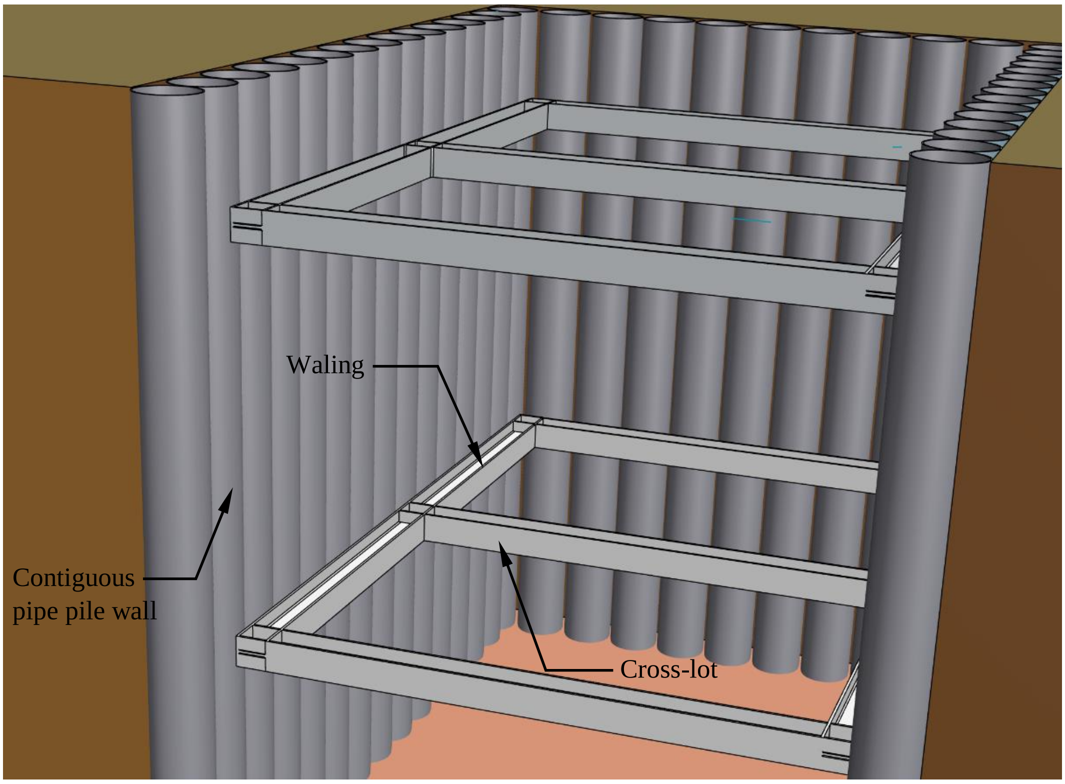


Figure 3.11 Strutting System with Cross-lot

The disadvantage of a cross-lot strutting system is that the working space can be severely restricted. The diagonal strutting system as shown in Figure 3.12 is more suited to small and preferably square excavations, e.g. shafts. Diagonal strutting is also used near the corners of wide excavations and serves to leave a relatively large portion of the excavation open. Raking struts are commonly used for unbalanced excavations where the excavation depth and loading are different on the opposite sides of an excavation, or where the excavation is particularly wide and makes cross-lot strutting impracticable. A combination of cross-lot, raking and diagonal struts is sometimes used to suit the specific site conditions.

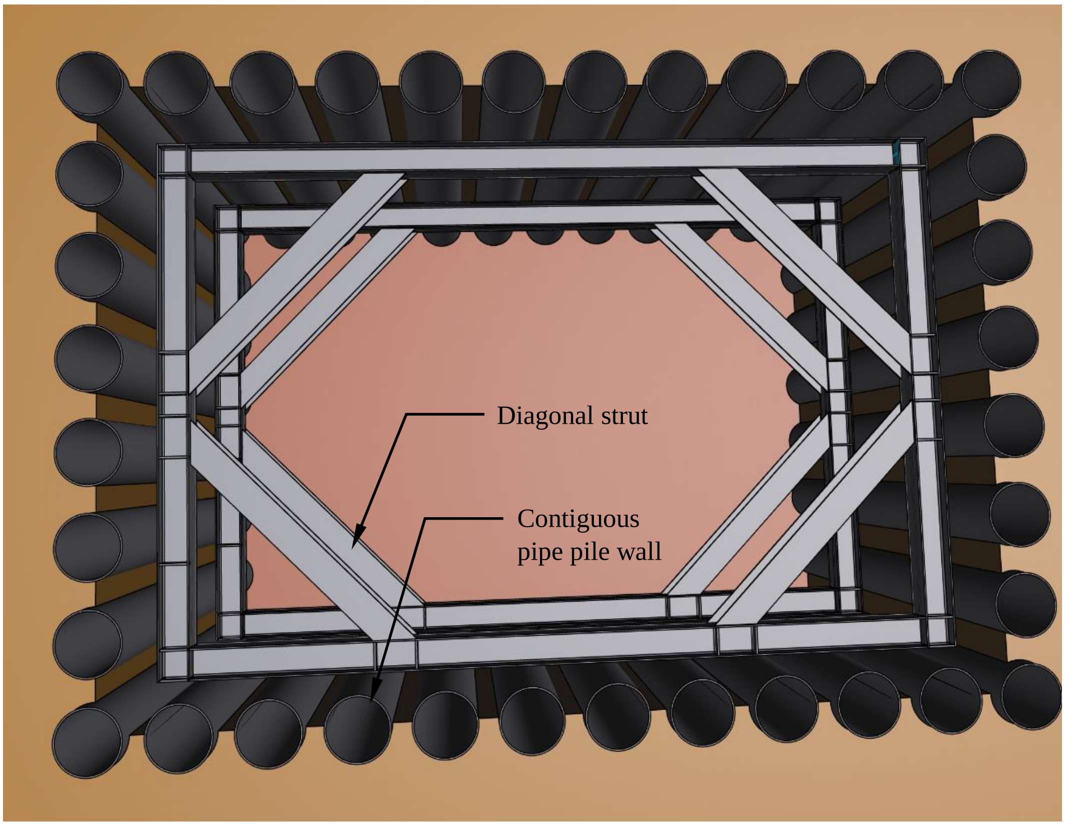


Figure 3.12 Strutting System with Diagonal Struts

Two types of construction sequence (i.e. bottom-up and top-down) for strutted walls are common in Hong Kong. Figure 3.13 illustrates the bottom-up sequence, in which the excavation is first completed before the permanent structure is built from the bottom upwards. The bottom-up sequence allows the permanent structure to be built from the base and independent of the temporary works. Hence the permanent structure, if kept separate from the temporary structures, will have no locked-in deformation and stresses due to sequential loading during excavation (Endicott, 2020). However, cross-lot or raking struts used to facilitate the bottom-up sequence can obstruct part of the excavation space and impede the construction of permanent structures. Besides, steel decking is usually erected on top of the struts to provide temporary platforms for site construction works.

The top-down construction sequence, as illustrated in Figure 3.14, uses the permanent internal structure as part of the strutting to the embedded wall, with the top basement slabs cast before further excavation down to lower-levels as the works progress. This sequence allows the superstructure to be constructed simultaneously with the basement structure. This method of construction is common for deep excavations in Hong Kong, particularly when it is planned as part of an accelerated construction of the superstructure. The top basement slab enables flexible usage of the ground (e.g. for road traffic) and provides cover to the site against adverse weather. Nevertheless, a disadvantage of this construction method is that it limits the head room for large construction plant to excavate materials underneath the constructed floor slabs. Large openings in slabs, e.g. for the mucking out of excavated materials, can locally influence the support stiffness to the wall.

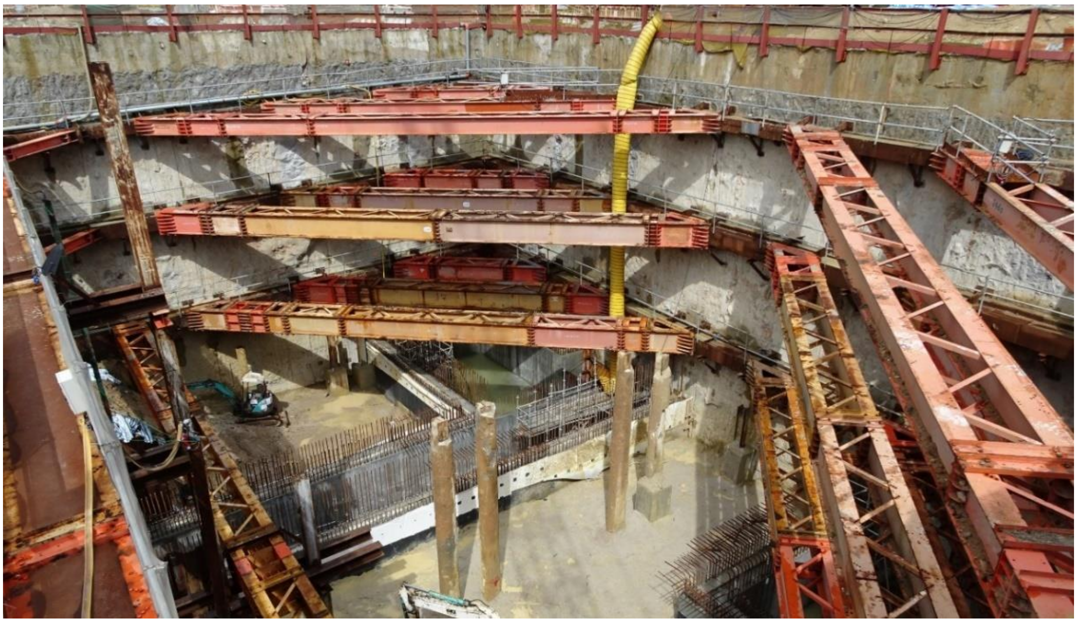


Figure 3.13 Bottom-up Construction

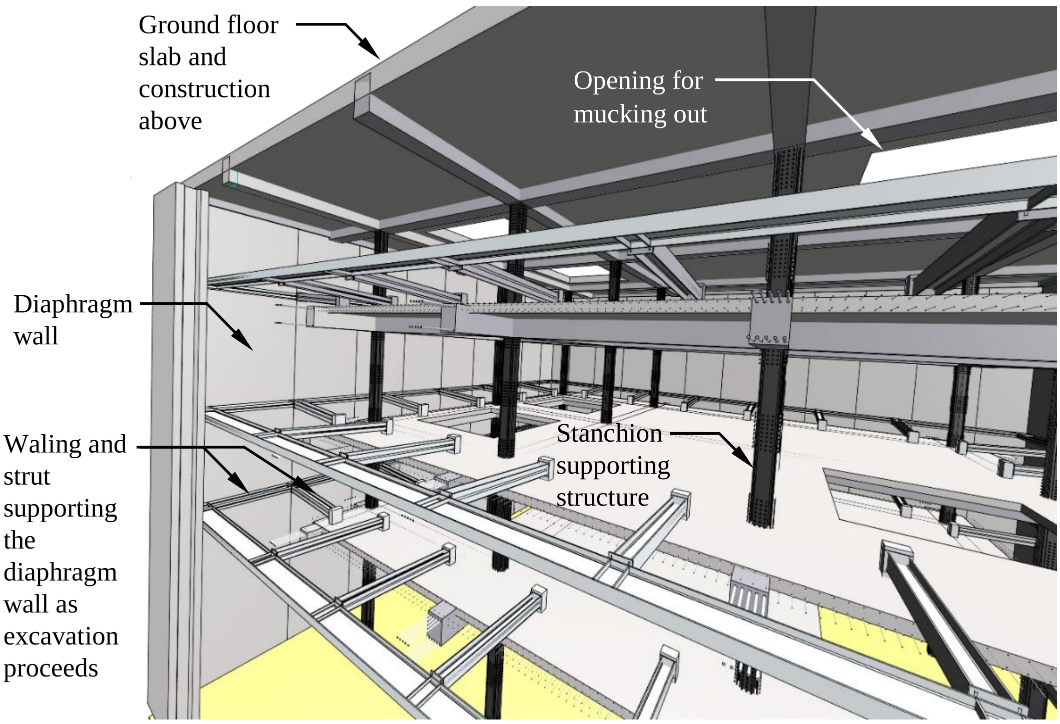


Figure 3.14 Top-down Construction

From programme and constructability perspectives, it is preferable to have lateral support provided at relatively large intervals so as to minimise restrictions on the working methods. Besides the strength and stiffness of the excavation support system, practical issues related to the erection of large struts and waling sections should also be considered (Gaba, 2012). The typical horizontal spacing between steel struts used in bottom-up construction in Hong Kong is between 3 m and 9 m, while the typical vertical spacing is between 2 m and 4 m. In the top-down construction method, a reinforced concrete slab can provide stiffer support than a steel strutting system, thus a larger vertical spacing can be allowed, typically ranging from 3 m to 6 m.

A diaphragm wall is commonly used as a permanent basement wall that is propped by the structural floor slab. Buildability aspects should be considered in designing the structural connection between the wall and basement slab. Starter bars can be cast in the diaphragm wall, and later exposed and bent out when needed to be lapped with the slab reinforcement. Proper and strong fixing is required to ensure the left-in starter bars are not damaged or displaced during concreting of the diaphragm wall. The size of starter bars should be suitably selected to avoid difficulty in bending out the bars. Sometimes, an additional row of starter bars is provided to allow for any misalignment in the elevation of the bars.

Embedding couplers in diaphragm walls is an alternative technique that has been used in some basement projects in Hong Kong. However, the quality of workmanship and site supervision of the quality of coupler connections are essential to avoid defective connections (e.g. improper connection and inadequate thread engagement), which may result in significant structural remedial works and also affect durability of the permanent structures. Adequate space for threading of reinforcement bars into embedded couplers should be provided. Localised post drilling may be adopted as a remedial measure for rectifying missing or misaligned couplers or starter bars. However, their installation may damage the wall reinforcement and affect the water tightness of the diaphragm wall panel.

3.3.3 Tied-back Wall

Tied-back walls, in which the wall is anchored or tied back into unexcavated ground outside the excavation, are less commonly used in Hong Kong, as it is not always possible to install the tie-backs in adjoining ground, particularly where it would involve encroachment into private land and properties. However, where encroachment into the adjoining ground is acceptable, a tied-back wall has the advantage of providing an excavation area free of strutting and facilitates construction of the permanent works. For excavations on sloping terrain, where there is large unbalanced excavation across the site and lack of space for construction works, a tied-back wall often provides a practical and feasible solution. Tied-back walls have been successfully employed in a number of local projects as temporary support measures by using ground anchors such as soil nails or prestressed anchors. Lam (2018) reported the application of a tied-back wall for a hillside excavation project in Stubbs Road, Hong Kong, as shown in Figure 3.15. Choi et al (2021) also reported the construction of the Lung Shan Tunnel portal using a temporary tied-back wall to support a composite retaining wall.

In Hong Kong, prestressed ground anchors (Figure 3.16) are sometimes used in a tied-back wall. The connecting elements are either tie rods or cable strands. These elements are commonly made of high strength steel and therefore a relatively small sectional area of anchor is required to provide a sufficient anchorage force to support the excavation. Prestressed ground anchors are seldom used as permanent structural support to the retaining wall, as this imposes a long-term monitoring commitment on the maintenance parties which usually involves appreciable recurrent cost and, should deficiencies be found at a later time, remedial works may be difficult and expensive. The monitoring requirements for soil nails and prestressed ground anchors are given in Geoguide 7 (GEO, 2023a) and Geospec 1 (GEO, 1997) respectively.

Figure 3.15 Tied-back Wall Support System for the Excavation Works at Stubbs Road

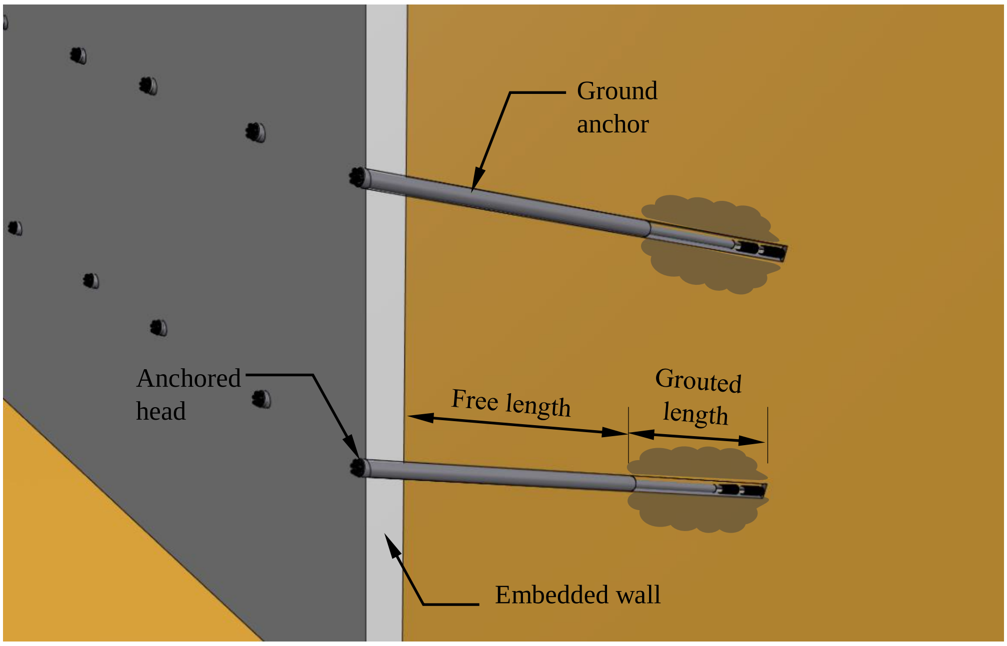


Figure 3.16 Tied-back Wall Support System Using Ground Anchors

Where tie-backs are used as temporary support, they are usually abandoned after completion of the substructure works. The left-in steel bars or wire strands may obstruct subsequent construction works in adjoining ground. Removable or retractable ground anchors have been adopted for deep excavation projects in Hong Kong, such as the Phase II Development of Hopewell Centre, a residential development in Tsing Yi, the MTR viaduct in Wong Chuk Hang (Chan et al, 2017), a commercial development at the Hong Kong International Airport, and the redevelopment of Grantham Hospital as shown in Figure 3.17. In these cases, the steel strands were pulled out, leaving only the plastic sheaths.

Swann et al (2013) described the use of glass fibre reinforced polymer (GFRP) bars as soil nails in the temporary excavation works for construction of the Ho Man Tin Station. GFRP bars can be cut with conventional drilling equipment and do not pose a significant obstruction to future construction works. Besides, the use of light weight GFRP bars is appealing for constrained sites where it may be difficult to deploy heavy lifting equipment.

Figure 3.17 Retractable Prestressed Ground Anchor Support System at the Redevelopment of Grantham Hospital

3.3.4 Circular Shaft

Circular shafts are gaining popularity in local large-scale projects where excavation deeper than 30 m is required. Diaphragm wall panels are commonly used to form the circular shaft, where the panels themselves act as compression members to resist the lateral earth load. In cases where the wall panels could not provide sufficient support through the hoop action, continuous reinforced concrete ring beams are constructed. This type of support system requires fewer or no internal strutting as compared to other systems, thereby allowing more free working space within the site. Also, the hoop compression can improve the water tightness between diaphragm wall panels. However, it is important to ensure effective overlapping of the wall panels in order to ensure full development of the hoop action. Thus, the tolerance of the alignment of the wall panels and joints should be carefully controlled (Gaba et al, 2017).

Pappin (2011) presented case histories of using a diaphragm wall to facilitate circular shaft excavation in Hong Kong and Singapore, including the Cheung Kong Centre, International Finance Centre 2 and the International Commerce Centre (Figure 3.18). The latter two excavations used diaphragm wall panels of 1.5 m thick, with the internal diameters of the circular shafts ranging between 61 m and 76 m and excavation depth up to 35 m. The Singapore experience showed that it was practicable to construct a circular shaft of up to 120 m in diameter and excavation depth up to 18 m. Despite such large-scale excavation, the lateral displacements recorded were small and generally less than 20 mm (Pappin, 2011).

More recently, multi-cell shaft excavation in a "peanut" shape, which is a series of interlocking circular shafts, has been used to construct a launching platform for tunnel boring machines (Figure 3.19) in the Trunk Road T2 and Cha Kwo Ling Tunnel Project. Chan et al (2020) described the use of a fifteen-cell cofferdam for the construction of the southern approach road of the Tuen Mun-Chek Lap Kok Link (Figure 3.20).

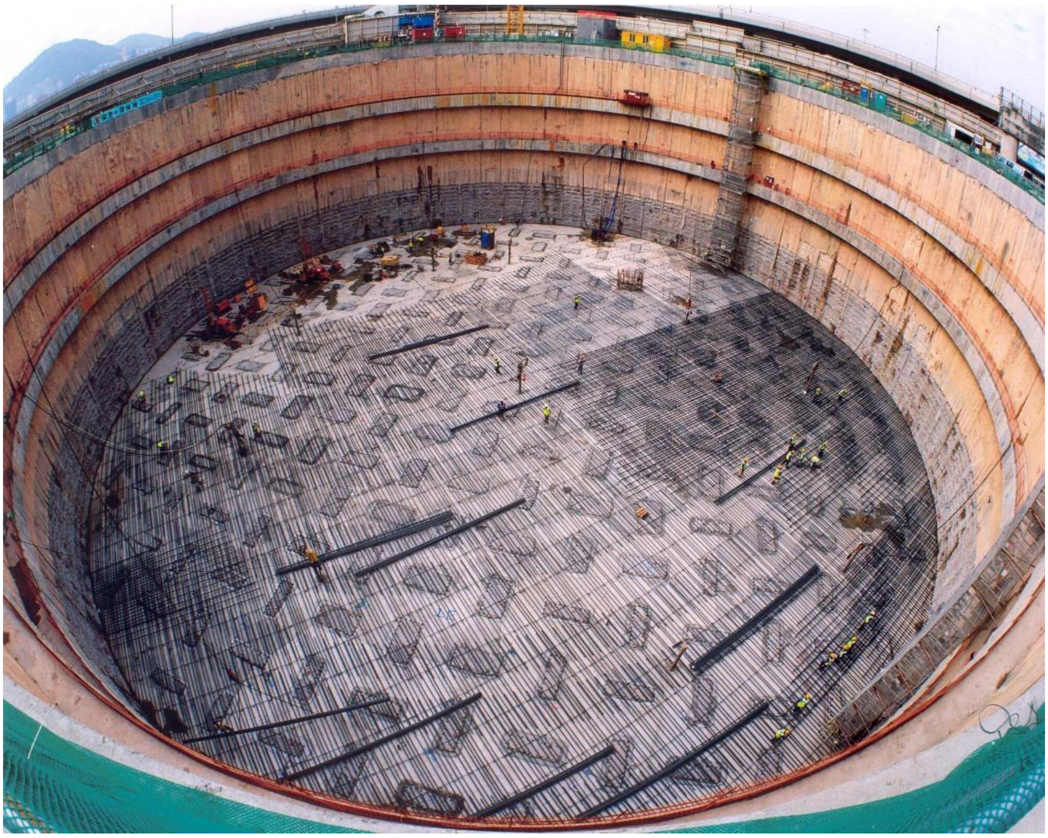


Figure 3.18 Circular Shaft for Basement Construction at International Commerce Centre

Figure 3.19 TBM Launching Peanut Shaft in Trunk Road T2 and Cha Kwo Ling Tunnel Project

Figure 3.20 Multi-cell Circular Shaft for Cut-and-Cover Tunnel Construction under the Tuen Mun-Chek Lap Kok Link Project

4 Design Considerations

4.1 General

This Chapter provides guidance on some of the key design considerations that are relevant to excavation support systems. Whilst the design needs to fulfil the fundamental requirements of stability and serviceability of the selected system, buildability aspects should also be properly considered to ensure that the works can be constructed efficiently, safely and effectively. THB (2020) highlighted the importance of addressing buildability aspects of the design, with a view to identifying and resolving any major construction issues at an early stage of the project.

4.2 Loading Conditions

Loads exerted by the retained ground, groundwater and surcharge should be duly considered in the design to ensure the stability and serviceability of ELS works. Guidance on loadings for the design of retaining wall as given in Geoguide 1 is relevant. Specific considerations on loading conditions for ELS works are discussed below.

4.2.1 Earth Pressure

The stability of the embedded wall can be evaluated either by the limit equilibrium or soil-structure interaction (SSI) method. In the limit equilibrium method, the soil is assumed to be fully mobilised to limit states based on admissible collapse mechanisms. The lateral earth pressure is typically assumed to increase linearly with depth. Geoguide 1 provides different methods for determining the lateral earth pressure at different limit states, including the pressure coefficients based on Caquot & Kerisel (1948) and trial wedge analysis. The latter method has the advantage of providing a better estimation of active pressure in case of uneven or steep ground profiles behind the embedded wall, but it can be a long and complicated iteration process. Alternatively, the uneven or steep ground profile can be represented as a series of surcharges acting behind the wall (Gaba et al, 2017). In this method, the shear strength of the soil above the top of the wall is ignored, which otherwise would reduce the active pressure acting on the embedded wall. The wall deflection profile usually cannot be predicted by the limit equilibrium method, except for the simple case of a cantilevered wall.

The SSI method can derive a more realistic distribution of earth pressure acting on an embedded wall. This is because it can estimate the wall deflection at each stage of excavation, which has a considerable influence on both the magnitude and distribution of the lateral earth pressure acting on the wall. The stiffnesses of soils and structural elements can be considered independently and the effects of the construction sequence can also be allowed for in the SSI analysis.

Different theories and computer programs are available for undertaking SSI analysis of an excavation support system, including the subgrade reaction, pseudo finite element, finite element and finite difference methods. However, it is important to understand the technical basis of these theories to ensure that correct assumptions and inputs are used in the analysis.

In examining the results of an SSI analysis for a complicated case, it is always useful to compare the results with those of a simplified limit equilibrium analysis of a similar problem in order to understand the reasonableness of the lateral earth pressure developed, and the magnitude and mode of deformation. Convergence in the SSI analysis is sometimes taken as achieving the minimum embedment for the wall. However, the convergence also depends on the number of iterations and magnitude of the tolerance specified in the computer program (Dunnicliff et al, 2012). Therefore, it is important to understand the tolerance used and to check whether excessive wall deflection occurs at each stage of the SSI analysis. Chapter 6 provides more discussion on analytical methods of SSI.

4.2.2 Surcharge

The design of an embedded wall should consider the surcharges applied to the ground, which may arise from the foundations of adjoining buildings, roads, construction plant and stockpiled materials. The guidance given in Geoguide 1 for the assessment of surcharges behind a retaining wall is also applicable to the design of an embedded wall, with due consideration given to the temporary and transient nature of the surcharges. Geoguide 1 provides methods for modelling the effects of uniformly and non-uniformly distributed loads on an embedded wall, such as loading over a limited area, line loads and point loads, which will give acceptable results in a limit equilibrium analysis. Methods that are developed from the modified Boussinesq solutions are based on elastic theory and assume a rigid wall with no deformation (GEO, 2020). Georgiadis & Anagnostopoulos (1998) showed that even a small lateral deflection of an embedded wall would significantly reduce the lateral earth pressure resulting from a surcharge to values smaller than those determined from elastic theories. In this regard, an SSI analysis provides a better evaluation of earth pressure arising from surcharge loading on an embedded wall.

4.2.3 Water Pressure

Retained groundwater has a marked effect on the force applied to an embedded wall. Therefore, it is necessary to assess the groundwater pressure distribution acting along the embedded wall, which depends on whether a hydrostatic condition or steady state seepage exists during excavation. Where it is necessary to control seepage, the ELS works are usually designed with a water cut-off barrier to minimise the groundwater flowing into the excavation. This can be achieved by installing the embedded wall, together with a grout curtain if needed, down to a relatively impermeable stratum. In these circumstances, hydrostatic water pressure can be assumed to be acting on the embedded wall.

The groundwater level outside the excavation can be affected by various sources of replenishing water, e.g. infiltration of rain on surrounding permeable ground surfaces, seawater intrusion at waterfront sites and leakage of water-carrying services. Given that the ELS works are usually in place for a few months or up to a couple of years, in most cases the design groundwater level (DGWL) can be determined based on monitored groundwater levels during the GI stage and provided with an added margin to allow for variations and fluctuations. For excavation at sloping sites, an additional consideration is the potential damming effect of the water cut-off barriers installed and its impact on the groundwater flow at depth. In such cases, the magnitude of groundwater rises could be estimated using design charts given by Pope & Ho

(1982) or computer-based seepage analysis. Chapter 6 discusses the determination of DGWL.

In some site settings, it may not be practicable or economical to install the water cut-off barrier to deeper and less permeable strata and the ELS works should be designed to allow for groundwater flowing into the excavation. In this case, a flow net analysis is usually required in order to establish the change of groundwater pressure under steady stage seepage. Site investigation should be carried out to establish the site topography, ground stratigraphy, permeability of the underlying soil and rock strata, and the presence of groundwater prior to excavation. It should be noted that the piezometric pressure within a confined aquifer may be different from the hydrostatic pressure. Suitable instrumentation, e.g. piezometers, should be installed at appropriate soil strata in order to determine the groundwater pressure distribution in different aquifers.

Water pressure distribution in a rock mass is often controlled by geological structures such as faults and dykes, as was evident in the deep excavations carried out for the Harbour Area Treatment Scheme (GEO, 2007). Where an embedded wall penetrates rock, the water pressure distribution will usually depend on the presence of any rock joints or fissures, as well as their permeability. Water pressure distribution in such situations may deviate from hydrostatic and can be determined with the aid of seepage analysis. In the event that rock fissure grouting has been carried out to seal a rock formation, it may be assumed that the rock is in a dry condition and no water pressure in the rock needs to be allowed for.

4.2.4 Seismic Loads

Hong Kong is situated in a region of low to moderate seismicity (GEO, 2015) and seismic loads are generally not critical for temporary works with a short design life. Consideration of seismic loads in the design of temporary works for construction of an excavation support system is usually not required.

4.3 Control of Groundwater

4.3.1 Water Cut-off Barrier

For excavation below groundwater level, it is preferable to install a water cut-off barrier to a sufficient depth such that it can minimise the groundwater inflow in a controllable manner and keep the excavation in reasonably dry condition. The cut-off barrier can be formed by vertical piles such as interlocking pipe piles or sheet piles. Where contiguous piles are used, the barrier is normally provided by a grout curtain formed between the vertical piles. The grout curtain can also help to maintain temporary face stability of the unsupported vertical cutting necessary for installation of the lagging wall.

Continuous seepage flow may cause gradual erosion and loss of fine particles from the soil matrix. The erosion could induce adverse local effects such as a reduction in soil volume, which over time may accumulate to cause cavities in the soil mass and excessive settlement at the ground surface (GEO, 2023b). For deep excavations that could affect the nearby sensitive buildings/structures/services with stringent tolerable movement limits, such a risk may be mitigated by designing a water cut-off barrier down to a relatively impermeable soil or rock stratum at greater depth, even though satisfying the conditions of wall stability and hydraulic stability may allow a shorter embedment depth.

4.3.2 Pumping Test

A pumping test may be specified for validating the soil mass permeability assumed in the groundwater seepage analysis and the results can also be used to estimate subsequent settlement caused by dewatering. However, the necessity for conducting a pumping test prior to bulk excavation should be carefully assessed, as it may induce a large differential piezometric pressure across the embedded wall at a stage when the lateral support needed to minimise ground deformation is not yet in place. The consequential wall lateral deflection caused by the pumping test could be significant, particularly for a deep excavation.

In an urban setting where an excavation is surrounded by sensitive structures (e.g. old buildings on shallow foundations, MTR facilities), it is more desirable and prudent to adopt a construction sequence that would minimise any ground deformation. Hence, dewatering in tandem with staged excavation with struts properly installed is a more sensible and preferable arrangement. The effect of groundwater drawdown outside the excavation should be safeguarded by monitoring the adjoining buildings and services and promptly taking the agreed response actions to avoid any adverse impact on them. In terms of contingency actions for preventing excessive groundwater drawdown outside the excavation, it is advisable to provide durable and reusable grout pipes for any subsequent remedial grouting, as well as the provision of recharge wells. In addition, it is highly preferable to adopt an automated system to monitor the groundwater level at regular intervals during excavation.

Pumping tests are generally unnecessary in any of the following circumstances:

(a) The mass permeability of the soils is low or the anticipated groundwater level at the site is below the final excavation level, such that dewatering is not required during excavation (with due consideration given to the anticipated groundwater level).

(b) Where the water cut-off barrier is installed down to soil strata with low permeability (e.g. highly decomposed saprolite with a permeability less than 10^{-7} m/s), or to a rock formation with suitable rock fissure grouting (e.g. usually to a minimum of 5 m depth from the rockhead level), such that the mass permeability of the strata would not be a key design concern and only minimal water flow into the excavation is anticipated.

(c) There are no nearby sensitive receivers that could be affected by groundwater drawdown (e.g. green field sites).

Where the ELS works are designed to allow steady state seepage to be maintained during the excavation and permeable soil strata exist below the toe of the embedded wall, a pumping test may be considered in order to validate the design assumptions, such as soil permeability, groundwater drawdown outside the site and sufficiency of the dewatering well. However, it should be noted that successful completion of a pumping test does not necessarily guarantee the water tightness of a grout curtain during the excavation stage. The performance of a grout curtain may deteriorate with time or be disturbed during bulk excavation (GEO, 2023b). Any local defect in the grout curtain identified during excavation should be rectified by regrouting in the defected zone.

A suitable pumping test can be conducted based on the constant drawdown test as given in BS ISO 14686:2003 (BSI, 2006). Local practice in conducting the test varies from the BS procedures by continuing the dewatering for a further 72 hours after reaching the steady state condition. Steady state seepage is considered to have been reached when the water level within the dewatering wells is drawn down to the target level and the rate of change of water level is less than 0.1 m per hour.

Cheung et al (2023) reported a review of the pumping tests conducted in recent deep excavation projects in Hong Kong. All the tests indicated that whenever steady state conditions were achieved, there was practically no change to the water level in the subsequent 72 hours period. Therefore, it is considered adequate to maintain the dewatering for a minimum of 24 hours after achieving steady state seepage, before commencement of the recovery stage. Table 4.1 shows the recommended intervals for measuring water levels during a pumping test.

Table 4.1 Recommended Intervals for Measuring Groundwater Levels in Observation Wells during a Pumping Test

Stage of Pumping Test	Interval between Readings
Baseline monitoring	4 hours (usually for a period of 3 to 14 days)
Dewatering before steady state	30 mins
Steady state seepage	1 hour (usually for a period of 24 hours)
Recovery stage	4 hours

Note: Steady state is reached when the rate of pumping is constant and the groundwater level inside the dewatering and observation wells varies by less than 0.1 m per hour.

4.4 Lateral Support

4.4.1 Strut Layout and Detailing

The load transfer mechanism between struts, walings and the embedded wall should be properly examined when planning the layout of the lateral support system, particularly in cases where the layout involves irregularly shaped excavations of large extent, unbalanced loadings and asymmetric ground levels across the excavation. The layout and details of the strutting should provide sufficient stability and robustness to the ELS system.

Inclined or diagonal corner struts induce in-plane axial load on the waling, which can be substantial if the corner struts are supporting a wide excavation face. It is good practice to arrange the strutting layout such that in-plane axial load on the waling does not rely solely on the wall friction to provide the resisting force. Tight connections between the waling and the embedded wall should be provided to ensure proper mobilisation of wall friction. Davies (1990) discussed a case of inadequate consideration of in-plane axial load in the waling arising from corner struts that related to the collapse of a strutted diaphragm wall in Hong Kong. CIRIA C517 (Twine & Roscoe, 1999) also elaborates the considerations on the structural requirements of the strutting system for deep excavations.

Where it is necessary to rely on wall friction to provide sufficient support to the in-plane load, mobilisation of wall friction should take into account possible disturbance caused by the installation of the wall. For an embedded wall in dense soil, the interface shear friction angle, δ_s, could be taken as the shear friction angle of the soil at the critical state, ϕ_{cv}'. Geoguide 1 recommends using lower bound values of ϕ_{cv}' of about 34° and 30° for Hong Kong granitic and volcanic (tuff and rhyolite) soils respectively. Wall adhesion, c_w, is usually neglected.

The adhesion between soft clay and an embedded wall has not been well studied in Hong Kong and it is common practice to use 10 kPa as a nominal value. Alternatively, the adhesion could be computed based on total stress analysis, and c_w is taken as a fraction of s_u.

$$c_w = \alpha\, s_u \qquad (4.1)$$

where α is a reduction factor related to the soil strength, the construction method and roughness of the wall surface.

Ou (2006) carried out back analysis of excavations in soft clay in Taipei, Singapore, San Francisco and Chicago, and reported that c_w is equal to $0.67s_u$ for a diaphragm wall and $0.5s_u$ for a sheet pile wall. The CIRIA C760 recommended taking α as 0.5 in stiff clay and smaller values in soft clay. If an adhesion greater than 10 kPa is to be used, adequate field or laboratory tests and analyses should be conducted.

For excavation in sloping ground with a large difference in ground levels, raking struts are often used to prop across the site in order to provide the support. In some cases, the site topography may prevent installation of struts across the whole site and local excavation is necessary. Besides, the unbalanced earth pressures may induce large loads on the installed wall at the lower side, which could cause downward movement and backward sway into the retained soil of the lower level wall. In such circumstances, it may be more practical to arrange for raking struts to be propped against intermediate support within the site, such as partially constructed pile caps, basement structures or large concrete blocks. In this regard, the effect of lateral load on partially completed pile caps or basement structures should be considered, including the lateral stability and structural adequacy of the foundations, and the potential for locked-in stresses on piles. Where warranted, assessment of the stiffness of the lateral support should also allow for deflection of the foundation piles or concrete blocks. A more attractive solution is to adopt temporary tie-backs to support the excavation, provided that permission to install the temporary support outside the lot boundary is obtained from the adjoining land owners. More specific details of a temporary support system and projects adopting such a scheme are discussed in Chapter 3.

Preloading is sometimes used to reduce wall deflections and hence ground deformation. However, excessive preloading should be avoided as it may be counter-productive by causing damage to grout curtains, adjacent utilities and underground structures. Also, long and slender steel strut members are more prone to buckling when jacking and wedging are carried out at the strut end.

4.4.2 Soil Berm

Soil berms formed inside the excavation are often used to provide support at intermediate stages. Besides providing lateral support to the embedded wall, they also act as a surcharge to the soil below the formation level. For limit equilibrium and simple SSI analyses, such as the 'beam on elastic foundation' approach, there are a few methods available to estimate the effect of a soil berm on the earth pressure on the passive side of the wall. The CIRIA C760 reviewed three such methods for a cantilevered wall which are the multiple Coulomb wedge method; the raised effective formation level method and the modified raised effective formation level method. The multiple Coulomb wedge method (NFEC, 1986) is applicable to both total stress and effective stress analyses (Daly & Powrie, 2001; Smethurst & Powrie, 2008). In this method, a series of Coulomb wedges of potential failure planes along the wall are assumed in order to obtain the lateral pressure distribution on the wall. It should be noted that the error due to assuming a planar failure surface in Coulomb theory increases rapidly with increases in wall friction δ (Morgenstern & Eisenstein, 1970). Therefore, as recommended in Geoguide 1, the wall friction, δ should be limited to $\phi'/3$ when using this method (Terzaghi, 1943). Further guidance on the use of Coulomb theory in assessing the passive earth pressure is given in Geoguide 1.

Another method for simulating the effect of soil berms in routine limit equilibrium or simple SSI analyses is to represent the berm by a strip load distribution equivalent to its self-weight, which provides a conservative estimate of the required wall embedment when compared to the multiple Coulomb wedge method. The equivalent surcharge method neglects the lateral pressure exerted by the soil berm on the wall and the shear resistance along any slip surfaces passing through the berms. Where there is a need to better estimate the effect of soil berms, finite element analysis could be carried out to model the soil berm directly.

The configuration and geometry of the soil berm normally depends on the construction sequence and the space required for installation of the struts. The soil berm should remain stable during the excavation stage and have an adequate safety factor in order to prevent any failure that could affect the stability of the embedded wall and cause unacceptable risk to adjoining properties, utilities and the public. On the other hand, for berms that are formed far away from the excavation boundary, other factors may be considered when determining a suitable factor of safety, such as the consequence of failure, the stand-up time of the berm and the loading close to the berm crest. Pallet & Filip (2019) describes the site-specific factors that are relevant to the design of temporary slopes in construction sites. Nevertheless, safety remains a paramount consideration and no site personnel should be subject to unacceptable risk while working in the vicinity of temporary slopes.

4.4.3 Wall Friction

δ is the angle of friction between the retained soil and the embedded wall and is often expressed as a ratio of the shearing resistance of the soil, though it is not a material property (GEO, 2020). The magnitude and direction of δ depends on the relative movement between the soil and the wall (GEO, 2020). δ in the active state will only be mobilised where the retained soil moves downwards relative to the soil/wall interface, while wall friction in the passive state is mobilised where the soil in the passive zone moves upwards relative to the soil/wall interface. If a vertical load-bearing wall or tied-back wall with prestressed ground anchors is founded on compressible ground, upward shear stresses need to be mobilised on the wall-soil interface in order to support the applied loads. In this case, it may be considered that the wall and the soil at the retained side move downward together such that no net friction is generated at the interface, therefore δ for active earth pressure should be assumed to be zero. Similarly, if the soil in the passive zone may settle under external loads or the wall may move upwards under raking strut forces, the δ for passive earth pressure should also be assumed to be zero.

4.5 Buildability

Buildability should be duly considered as an integral part of the design of ELS works. Temporary works are part of the process to construct the permanent structure and therefore it is important to visualise and optimise the interaction between the permanent and temporary works design and methodology, so that hazards and construction problems can be identified and addressed at an early stage of the design. DevB (2016) introduced the concept of design for safety in construction works and some of the guidance in that document on identifying risks and hazards may also be relevant to the geotechnical design and execution of ELS works.

The strutting system should be designed such that the site operations are not adversely constrained, e.g. the spacing between support members should allow sufficient clearance for the works. In particular, in strutted excavations, the lifting of heavy temporary steel members (e.g. struts, waling, brackets and other structural steel) and steel cages for permanent structures may induce various risks such as collision, working at height and falling objects. The design tolerances should be specified and should be based on feasible construction methods, as well as any predicted wall deflection which may occur during excavation.

It is not unusual for existing utilities to run through the site of an excavation project. When existing utilities or underground structures clash with the alignment of the embedded wall, it is not always possible to divert the utilities prior to installation. Special design provisions are needed to ensure that the excavation underneath the utilities and underground structures is carried out safely.

Ground improvement works, such as grouting, may be carried out to strengthen the soils underneath the utilities. Where excavation around the utilities is required, lagging panels are often installed to span between the vertical wall on both sides of the utilities. The design of the lagging panels should consider the space occupied by the utilities, the intersection angle between the utilities and the wall, the vibration and any other adverse effect on the ground during wall installation and the tolerance required for wall installation. Extra clearance should be allowed for in the opening width when there is high uncertainty in the alignment of utilities and when significant vibration or excessive settlement is anticipated to occur during wall installation.

Digital construction techniques have become a prerequisite in many projects and are also applicable in the temporary works design process. The buildability of the excavation support system is best communicated with the aid of Building Information Modelling (BIM). The BIM model can virtually simulate the construction processes for integrating the permanent and temporary works design, eliminating construction errors, and detecting any potential clashes between temporary works, construction plant and permanent works. Figures 4.1 and 4.2 illustrate the applications of BIM technology in excavation projects. The use of BIM is particularly important for sites where high coordination amongst different stakeholders is necessary and where there are many constraints on the construction works.

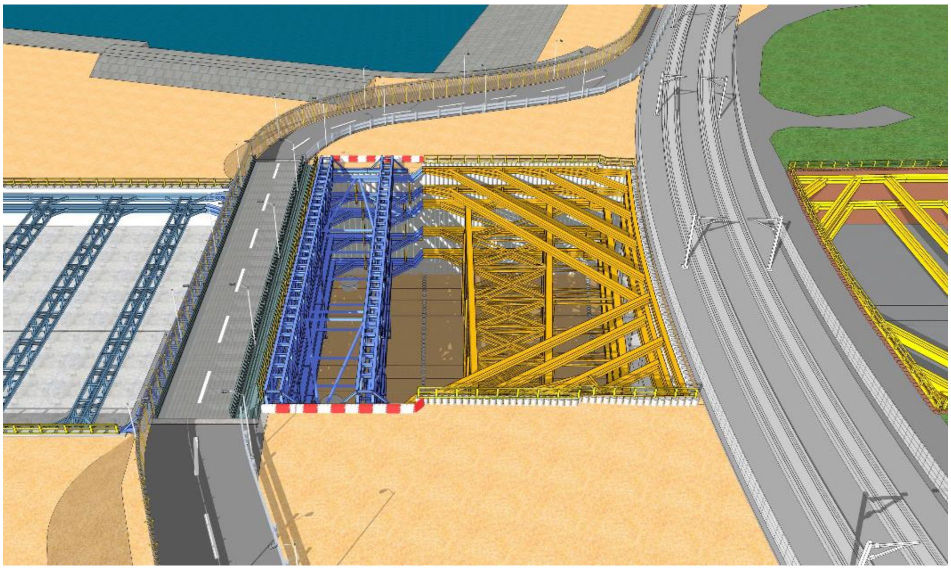


Figure 4.1 Use of BIM for the Cut-and-cover Tunnels in the Hong Kong International Airport

Figure 4.2 Use of BIM for Excavation Works at a Residential Development

Design of ELS works should be optimised from a holistic perspective to assure the cost-effectiveness of the project while maintaining the functionality, quality and safety of the works. The design should take into consideration different aspects, such as choice of a suitable type of embedded wall, an effective strutting layout and configuration, water cut-off measure, and construction sequence.

In recent years, shoring systems adopting integrated modules and components, that are manufactured and assembled in a controlled factory environment and delivered to site for installation, have been gaining popularity. Where site conditions permit, the adoption of modular construction could enhance productivity, quality and safety in the execution of ELS works. The advantages of using modular shoring systems can be best leveraged through advance planning in aspects such as the strategic selection and standardisation of member sizes as well as maximising the reuse potential of the modular shoring components. Toh et al (2023) discussed the concept of modularisation and its implementation in ELS works in Hong Kong.

Apart from the buildability aspect, adequate supervision is essential to avoid the occurrence of site problems or irregularities, such as non-compliance with agreed construction sequences, poor workmanship and defects in structural connections. More discussions on construction considerations are presented in Chapter 5.

5 Construction Considerations

5.1 General

Construction uncertainties cannot be completely identified at the site investigation and design stages. However, with well-planned construction procedures, provision of adequate precautionary measures, close supervision and monitoring, and timely implementation of remedial and contingency measures, it should be feasible to minimise construction problems due to unforeseen site conditions. Whilst complete collapse of ELS systems has rarely occurred in recent years, cases of excessive ground loss and sudden formation of sinkholes caused by deep excavation works are not uncommon. Some incidents have caused injuries to the public as well as damage to properties and facilities. This Chapter provides guidance on suitable precautionary measures to minimise the risks associated with the common types of construction problems encountered in ELS works.

5.2 Site Conditions Particularly Vulnerable to Ground Loss

The presence of loose fill renders a site more susceptible to excessive ground loss during piling operations and bulk excavation, particularly where the groundwater table is near the ground surface. This is commonly the case in reclaimed land where the fill was placed by the hydraulic filling method. In urban sites, utilities and services are often laid in congested spaces and the fill surrounding them is difficult to compact to a dense state. Leakage of water from utilities and pressurised water-carrying services may aggravate the problem, as water seepage may cause the fine soil particles to be washed out of the soil matrix for a substantial period and lead to the formation of cavities. Where such site conditions are anticipated, GCO probing and GPR surveys should be considered in order to identify the presence of any underground cavities at an early stage of the excavation project. Chapter 2 provides discussion on these GI techniques.

5.3 Ground Loss Caused by Boring Operations

Soldier pile or pipe pile walls are usually installed by boring, which is a suitable method for penetrating ground where the presence of hard materials (e.g. boulders, dense soils) makes percussive driving difficult. Eccentric drilling systems had been widely adopted previously, but in 2008 there were incidents of ground collapse reported to be related to the use of such systems. The possible cause of these incidents was excessive removal of soil by the compressed air passing through the gap formed between the oversized reamer and the casing (Wong, 2014). As a result, boring operations using an eccentric drilling system are not common nowadays, particularly for sites in urban areas and those with adjoining old and sensitive buildings.

Since then, concentric drilling systems have gained popularity in boring operations. Most concentric drilling systems comprise a pilot bit, a ring bit and a casing shoe. The casing shoe allows the ring bit to rotate freely such that the steel casing does not rotate during boring. All components are aligned concentrically and such a design allows a slightly oversized hole to be formed for pulling down the casing. These boring systems commonly use compressed air flushing to bring the soil and rock cuttings to the ground surface. The steel casing is attached to the concentric ring bit and is pulled down together with the drill bit as it is advanced by the percussive action of the down-the-hole hammer. Figure 5.1 illustrates the key components of a concentric drilling system with idealised air flow paths that aim to return the air and cuttings through the gap between the casing and the inner drill rod.

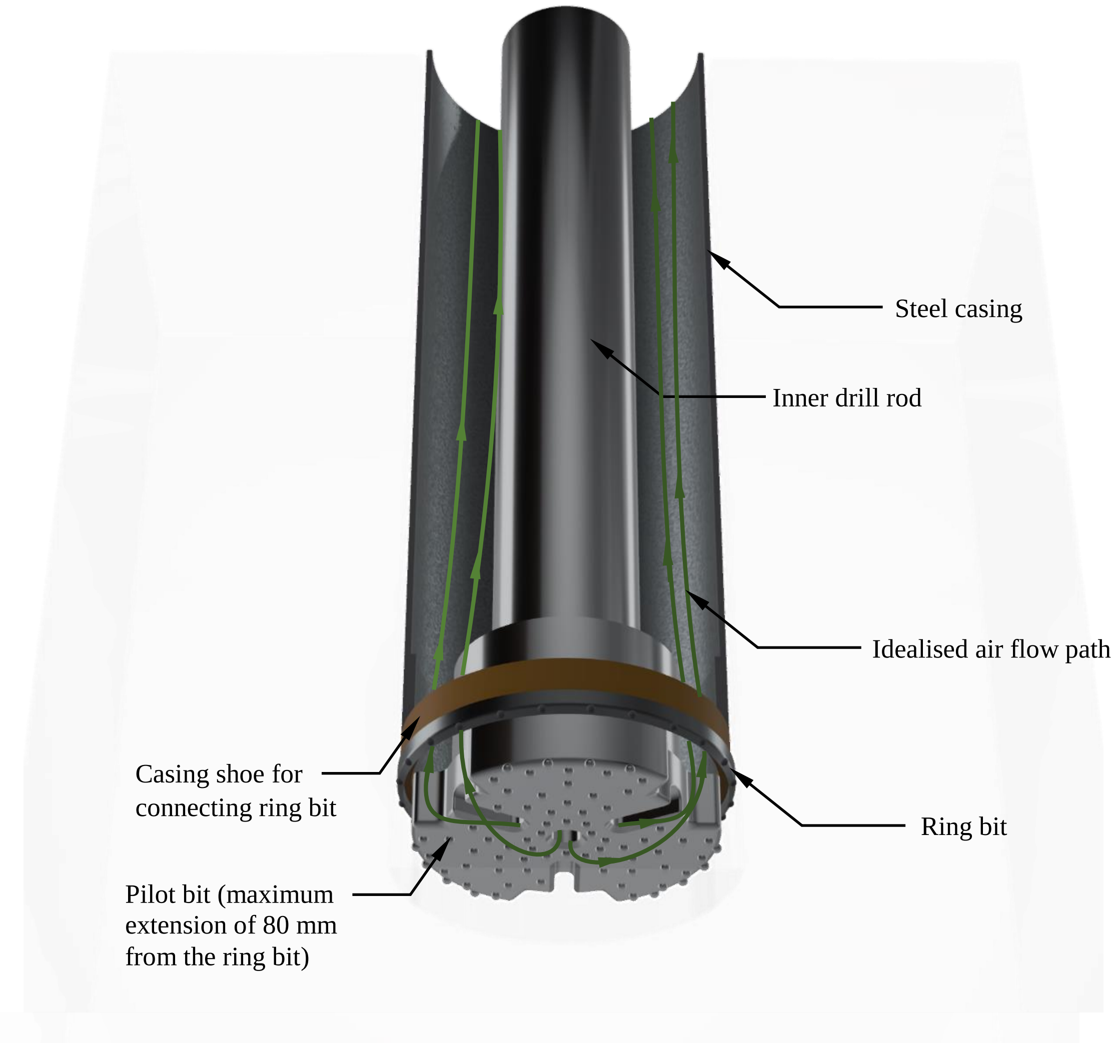


Figure 5.1 Key Components of a Concentric Drilling System with Idealised Air Flow Path

Despite the improvements made in the drilling system, incidents of excessive ground settlement and sudden formation of sinkholes are still reported occasionally. GEO (2023b) documented a few such incidents that were related to excessive disturbance of the ground due to boring operations used for installing piles. It should be noted that sinkholes ultimately formed at the ground surface could be located some distance away from the boring position.

Most concentric drilling systems use compressed air as the flushing medium to remove soil and rock cuttings. However, use of a high air pressure causes a high suction pressure in the system that could also remove excessive soil particles from the soil matrix. Where underground obstructions (e.g. old foundations, buried seawalls, boulders) or mixed soil and rock strata are encountered, more time will be needed to penetrate the hard materials and this may significantly increase the risk of ground loss and excessive settlement. Figure 5.2 shows types of ground disturbance that could result if an unduly high air flushing pressure is used. Excessive soil particles could be extracted from the adjoining ground, leading to the formation of cavities both adjacent to and below the drill bit. On the contrary, if the air flushing pressure is too low, the resulting slow advancement of the drill bit and prolonged air flushing may also increase the risk of ground loss and sinkhole formation.

In order to minimise disturbance to the adjacent ground due to the boring operation, the pressure of the compressed air should be carefully assessed and monitored, especially for sites with a high groundwater table and thick layers of loose fill, bouldery colluvium or rockfill. Trial boring is usually conducted and should be aimed to determine site-specific minimum workable air pressures that could achieve reasonable advancement of the pile in different ground conditions. During trials, drilling should commence with a low air flushing pressure, which is then gradually increased to obtain the minimum workable air pressure that could advance the drill bit. The presence of boulders in fill or colluvium may significantly reduce the advancement rate and cause excessive soil removal due to the prolonged drilling time. Any sudden change of drilling rate should be handled with caution. The established minimum air pressure should be used to install the working piles.

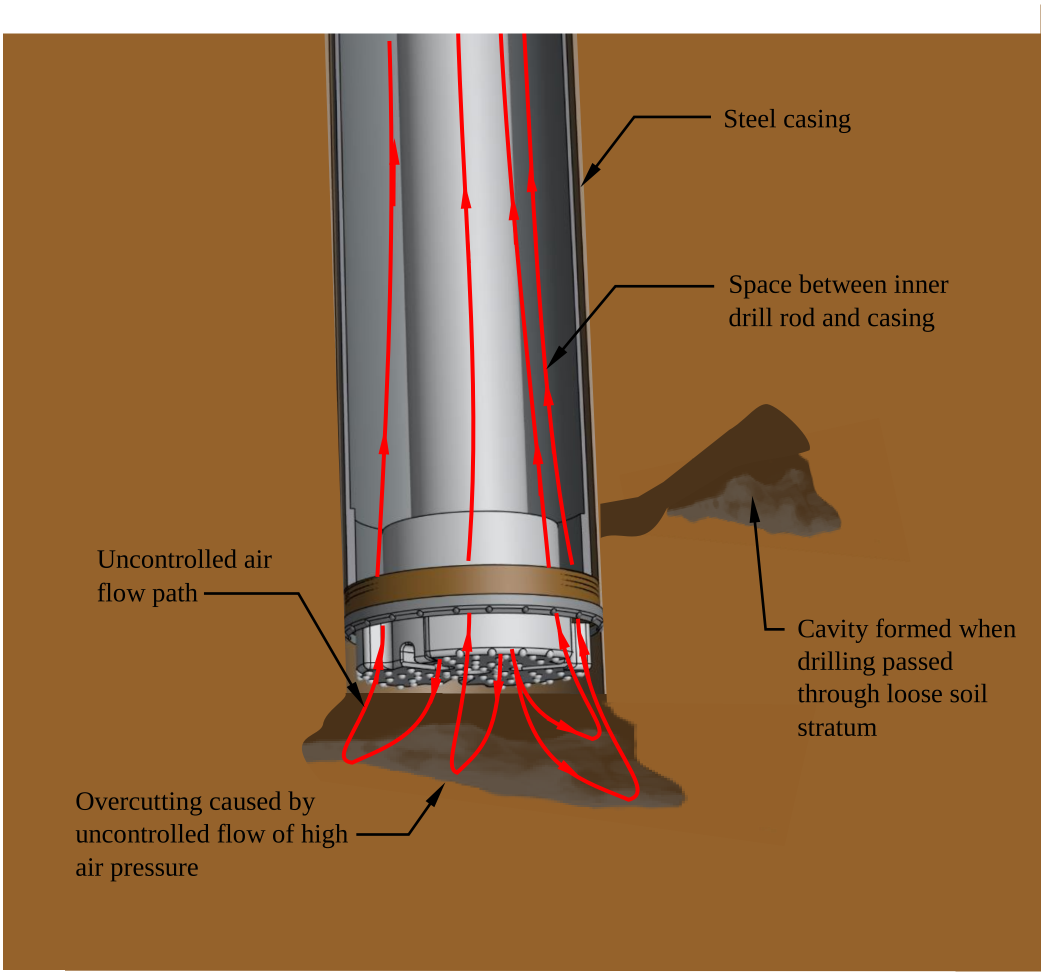


Figure 5.2 Ground Disturbance Caused by Unduly High Air Flushing Pressure during Drilling

It should be cautioned that excessive soil loss and formation of cavities may not be immediately noticeable at the ground surface or revealed from ground settlement monitoring stations. Where the site conditions are particularly vulnerable to ground loss (e.g. loose fill layer), probing tests (e.g. GCO probe or SPT discussed in Geoguide 2) should be conducted before and after the trial boring. Probing tests can help identify the presence of any cavities at depth that may have been formed by the boring operation.

The boring operation parameters, in particular, the applied air flushing pressure, advancement rate and volume of materials removed from the boring, should be closely supervised and monitored by qualified supervision personnel. It is important to properly set up the drilling equipment to facilitate monitoring works. For example, the pressure measurement gauge and the throttle for controlling the air pressure should be housed in the rig operator's chamber, such that the drilling operator can easily read and vary the applied air pressure. Where necessary, an automatic recording system could be implemented to assist with the monitoring. As a minimum, a video recording system should be assembled to record the pressure gauge readings and the material removed from the boring. Such information should be reviewed by the supervision personnel from time to time to ensure that boring operations are conducted according to the site-specific drilling parameters obtained from test borings.

Cavities formed at depth during boring may be temporarily supported by soil arching. Such arching is commonly in a metastable condition and may be easily destroyed due to subsequent changes in soil stresses. It is not uncommon to see sinkholes formed months after the installation of an embedded wall, during the later stage of bulk excavation. Deflection of the embedded wall due to excavation, as well as pushing of the wall by any preloading action, could cause the collapse of metastable soil cavities and subsequently lead to sudden formation of a sinkhole at the ground surface. As such, it is prudent to conduct additional inspection, including GPR survey and CCTV inspection of underground utilities, at critical stage of the works (e.g. after completion of the installation of the embedded wall) and at regular intervals if prolonged dewatering is necessary to facilitate the basement construction (e.g. every three months). The inspection helps to detect any anomalies at an early stage that might be related to cavities formed in association with the ELS works. If an anomaly is identified, probing tests could follow to confirm the presence of any cavities and facilitate the carrying out of timely remedial works.

Other precautionary measures, such as grouting or installation of sheet pile sections prior to the boring operation, have been used in deep excavation projects to minimise the risk of ground loss and damage to adjoining structures. As an alternative, drilling systems employing water as the flushing medium have also been successfully used in Hong Kong for projects in reclaimed land, but in such cases, due consideration should be given to its effect on the groundwater regime and adjacent structures, utilities and facilities.

Any excavation ahead of the toe of temporary casing in bored piling operations may cause excessive ingress of groundwater and soil into the bored hole and should not be allowed. Also, a suitable excess water head should be maintained within the bored hole throughout the installation of a bored pile.

Similar problems may also arise where steep rock head is encountered and it is necessary to advance the bored pile into rock by using RCD. The RCD will grind a mixture of rock and soil when it reaches the rockhead, and prolonged operation may cause excessive removal of the soil. In such circumstances, localised grouting may be carried out at the toe of temporary casing to form a grout plug when the boring has reached the rockhead surface, so as to minimise the soil removed while the RCD is reaming the rock.

5.4 Slurry Trench Instability

Trenches excavated for installing diaphragm wall panels are usually supported by bentonite slurry or a synthetic mud slurry. The stability of the slurry-filled trench is important, as any failure could lead to severe damage to adjacent structures. Detailed discussions on the use of drilling fluids for the support of trench excavation for diaphragm wall and bored piles are given in GEO (2006). This guidance is also applicable to the construction of diaphragm wall panels or bored piles as part of ELS works.

Stability of the slurry trench is maintained by keeping the slurry pressure head in excess of the earth pressure within the trench. It is common to construct guide walls (Figure 5.3) along the alignment of the diaphragm wall panels, which provide support to the trench at shallow depth where the slurry pressure head alone is usually insufficient to support the ground. Guide walls are made of reinforced concrete panels, typically 200 mm to 300 mm thick and embedded 1 m to 1.5 m into the ground. Where necessary, the guide walls can be extended above the ground surface to allow the slurry to be maintained at a higher level inside the trench. The guide walls also help to maintain the positional accuracy of the diaphragm wall panels. Trench stability can also be improved by reducing the panel length.

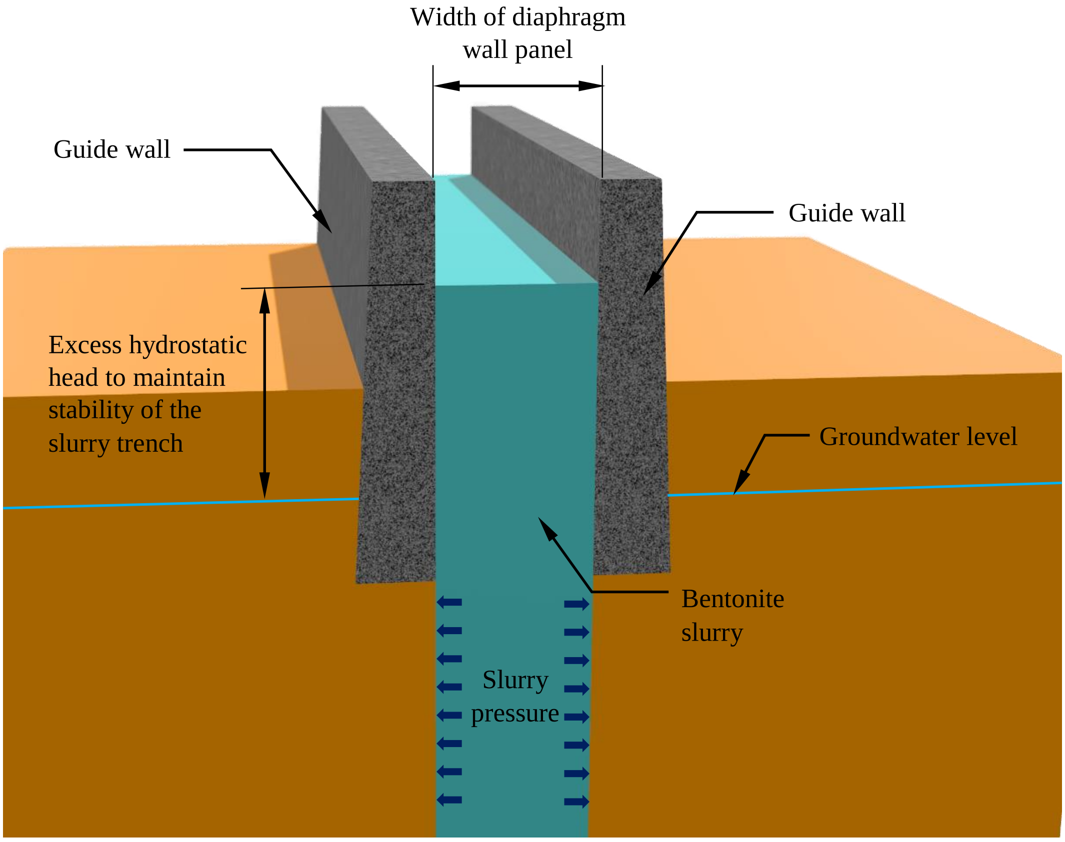


Figure 5.3 Guide Walls for Slurry Trench Excavation

5.5 Grout Curtain

For non-interlocking types of embedded wall (e.g. solider piles, pipe piles and contiguous bored piles), gaps exist between the vertical piles. A grout curtain is usually installed between the vertical piles when it is necessary to form an impermeable barrier for keeping the excavation dry during construction. In Hong Kong, permeation grouting by the Tube-A-Manchette (TAM) method is commonly adopted to form the grout curtain (Figure 5.4). In this method, steel TAM pipes of about 50 mm in diameter are installed in 100 mm diameter drill holes that have been formed to the required depth. The annular space between the pipe and the drill hole is filled with a relatively weak sleeve grout that should harden within a few days. The grout mix of the sleeve grout should be adjusted to avoid being too hard to be cracked by subsequent grouting. The first phase of permeation grouting usually involves injection of bentonite or a similar cement-based grout, which permeates coarse pores and fills major relict fissures. This is then followed by silicate-based chemical grout, which sets as a gel and further reduces soil permeability.

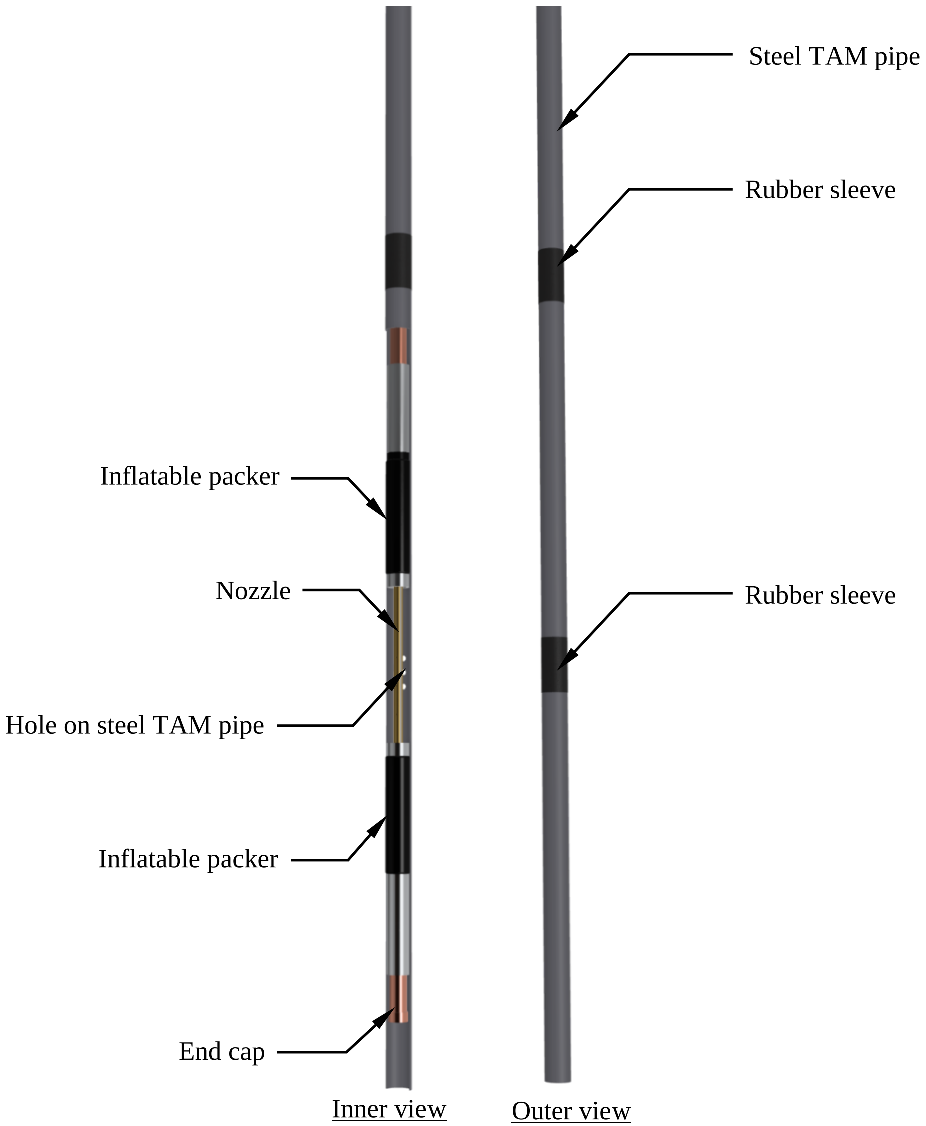


Figure 5.4 Details of TAM Grouting

The effectiveness of permeation grouting depends on factors such as the properties of the grout mix, ground conditions, particle size distribution of the soil to be grouted, and the operational details (e.g. grout hole pattern, grouting sequence, injection rate and pressure). The grout mix is often determined by a specialist grouting contractor. Site trials should be conducted in order to determine the final design of the grout mix. Shirlaw (1987) discussed the bentonite cement grout and chemical grout mixes found to be effective in decomposed granitic soils in Hong Kong. Sometimes, the use of microfine grout may also be considered.

Permeation grouting is generally effective in sandy and gravelly soils with relatively coarse pore sizes. However, permeation grouting in fine-grained soils, such as silts and clays, is ineffective and in many cases such soils do not need to be grouted for further reduction in permeability. For fissure grouting in rock, general guidance is given in Geoguide 4 (GEO, 2018) regarding the grout mix and other design considerations.

The performance of grout curtains as impermeable barriers may deteriorate with time for various reasons, e.g. sodium silicate-based grouts are generally considered non-durable, excavation to make space for installing laggings between vertical piles may damage the grout curtain. As bulk excavation commences, there may be differential movement between the vertical piles and the surrounding soil that could crack the grout curtain, particularly for an excavation support system with large preloading forces (Figure 5.5). Disruptive works in the adjacent areas should be minimised as far as practicable to avoid possible damage to the grout curtain.

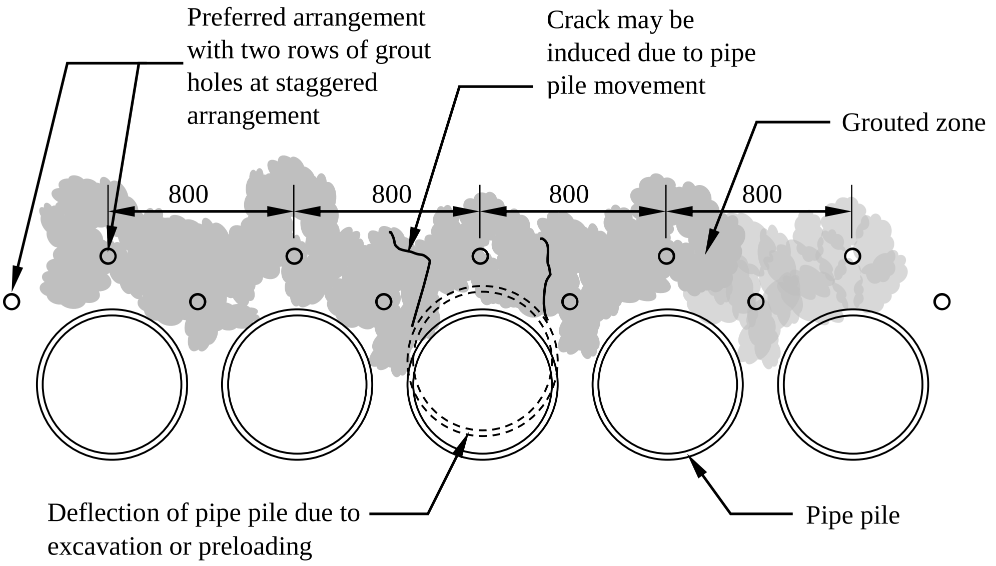


Figure 5.5 Possible Damage to a Grout Curtain

One row of grout holes might give poor water cut-off performance, especially at sites with a high groundwater table and loose fill layer (e.g. reclaimed land) or when the spacing between contiguous piles is large. Except for shallow excavations, the grout curtain should be formed by two rows of grout holes in a staggered alignment where possible, with spacing varying between 0.6 m and 1.0 m, to give satisfactory impermeable performance. If pipe piles are installed close to nearby buildings, structures and services and there is insufficient space for two rows of grout holes, interlocking pipe piles provide an alternative solution. In general, pressurised grouting should not be applied in the top 2 m of the ground cover, so as to ensure a sufficient confining pressure to prevent heaving of adjacent ground and grout spillage at the surface, which may cause damage to adjacent underground utilities.

Leakage of a grout curtain during excavation is particularly problematic, especially when the ground level inside the site has been excavated to a lower level. There may not be adequate space for mobilising drilling rigs and equipment to carry out any remedial grouting. Therefore, suitable precautionary measures should be considered as part of the grouting proposal to facilitate re-grouting if found necessary. After completion of grouting works, grout holes or TAM pipes should be thoroughly flushed so that they can be used for re-grouting if necessary. TAM pipes should be of a durable type (e.g. steel tube) and care should be exercised to prevent damaging the pipes during excavation or other site activities. A system of recharge wells may be installed prior to the bulk excavation works. If there are particular concerns about possible leakage due to defects of a grout curtain (e.g. excessive drawdown of groundwater level in the unexcavated side, leakage of water between vertical piles), these measures should be activated to minimise the risk of excessive ground settlement. GEO (2023b) reported incidents of sudden collapse of the adjoining ground that might have been caused by grout curtain defects.

Grouting works should be carefully controlled (e.g. in relation to grout volumes, injection pressure and rate). The grouting pressure should be limited to avoid ground hydrofracture, in which the soil matrix is broken and grout flows away from the intended treatment zone. The onset of hydrofacturing may be marked by a sudden drop of grouting pressure whilst the inflow rate increases.

Grout is prone to be washed away if there is substantial groundwater seepage flow beneath the site, e.g. due to a buried stream, or a fluctuating tidal current at the coastal front. In such cases, accelerators and other additives could be used to reduce the set time and improve the quality of the grout curtain. During grouting, underground utilities and manholes in the nearby area should be inspected to detect any grout leaking from the treatment zone.

During excavation, the rate of groundwater inflow and the drawdown of groundwater level outside the site should be closely monitored in order to verify the actual performance of the water cut-off measure. Anomalies such as a sudden increase of groundwater inflow or drawdown, ingress of a large amount of soil, and excessive removal of grouted soil may all indicate a defective or damaged water cut-off measure. It is important to conduct site inspections to identify any signs of ground loss or sinkhole formation at the ground surface (e.g. deformed pavement), and carry out necessary measures (e.g. fence off the concerned area, carry out GI to detect any underground voids, and undertake remedial works if required under a safe condition). Re-grouting can also be carried out prior to or during excavation if any anomalies are observed.

5.6 Structural Support to Embedded Wall

Connections between structural members should be constructed strictly according to the designed details. Adequate stiffeners and steel studs should be provided, and properly welded, as these are important to the entire load transfer mechanism of the ELS works. The collapse of the Nicoll Highway in Singapore (COI, 2005) was triggered by the initial bending of flanges and buckling of walings at strut-to-waling connections. Endicott (2020) discussed a few cases in which the failures were caused by poor detailing in the structural connections. It is a good practice to install vertical stiffeners at walings in order to provide adequate capacity against sway buckling.

Li et al (2010) discussed some precautionary measures that were used in a deep excavation project to prevent accidental damage to structural supports. Where possible, the strutting layout should be so arranged such that a sufficiently large and unobstructed space is allowed for the hoisting of materials by crane, so as to prevent installed struts from being accidently hit by the hoist and lifted objects moved in or out of the excavation. All material deliveries should be confined to such a designated area. Struts and walings immediately adjacent to the designated area can be painted with signs to indicate their vulnerability and installed with anti-collision steel frames. Also, it is common nowadays for excavators and other machinery to be equipped with object avoidance detection sensors, such that operators are alerted to any potential collisions with objects and site personnel. These are all good practice measures that will improve overall safety of the excavation during construction.

The sequence of removal of structural support should be properly designed and well planned for. The load transfer mechanism and the loading on individual members can vary substantially when the struts and walings are removed. CIRIA C517 discussed the key considerations in the removal of the structural supports for an excavation. It is important to note that strain energy is stored in the struts and should be released in a safe and controlled manner. The filling between the permanent structure and the embedded wall should be compacted adequately. Any unbalanced loading across the excavation should be considered. It is common to design the waling with struts acting as intermediate support. The effective length and the bending moment induced on the waling will change when the struts are removed. It is necessary to check that the waling will not buckle and the connection at the waling will not deform excessively, particularly when the waling is carrying substantial axial load.

5.7 Site Supervision

A proper supervision and monitoring system is needed to ensure that the excavation support system is constructed in accordance with the design.

Site monitoring is essential to verify the design assumptions (e.g. ground and groundwater conditions) and to evaluate the actual performance of the system as it is being constructed. On the other hand, site supervisory staff should always be alert and take note of any signs of possible ground loss and formation of sinkholes, which typically include the following abnormalities:

(a) Larger than expected groundwater discharge seeping into the excavation (e.g. the need to operate more submersible pumps to maintain a dry condition);

(b) Significant increase in the amount of soil accumulated in sump pits or sedimentation tanks;

(c) Sudden increase in the quantity of cuttings extracted during boring operations (e.g. more truck loads are required to remove the cuttings offsite); and

(d) Excessive movement of adjacent structures or facilities.

It is also important for the site supervisory staff to check and maintain the adequacy and functionality of all monitoring instruments. There have been cases where some monitoring stations, e.g. piezometers, were not properly installed and monitored or were damaged without replacement, and the problems were not identified until severe consequences occurred (e.g. sinkhole incident in 2014 caused by the ELS works at Jardine's Bazaar in Causeway Bay). Measurements at monitoring stations involving settlement of ground and utilities should be periodically conducted and certified by a qualified land surveyor (e.g. at monthly intervals). This will help to ensure the quality of the monitoring system and allow early identification of anomalies on site. Guidance on I&M is given in Chapter 10.

A contingency plan should be carefully devised, with adequate provision for prompt actions to deal with any signs of distress and observed ground loss and excessive groundwater ingress. The contingency provisions should include emergency measures that can be quickly mobilised if required, and the plant and equipment necessary for carrying out emergency works should be maintained in a good and ready condition, e.g. grout pipes should be kept unblocked in case they are needed for re-grouting.

6 Limit State Design

6.1 General

Limit state design is commonly adopted in the design of ELS works, which should satisfy the fundamental requirements of stability and serviceability (i.e. ULS and SLS).

Safety factors against limit states can be applied either by the Global Factor Method (GFM) or the Partial Factor Method (PFM). The GFM is more widely adopted in local practice because it allows simple stability checks based on a single factor of safety to cater for overall uncertainties. On the other hand, the PFM permits uncertainties to be considered for individual loading and material characteristics and provides a more rational basis for design.

In ELS works, groundwater is a key governing factor that affects wall deflection and ground deformation. Different groundwater levels are considered in assessing both ULS and SLS conditions. Sometimes, a conservatively estimated groundwater level is used in both ULS and SLS design. However, using the ULS groundwater level for SLS design may lead to overestimation of predicted wall deflection and corresponding ground deformation. On the other hand, such assumptions may result in the adoption of heavier struts and walings, or even preloading, so as to control ground deformation to within the tolerable limits of nearby sensitive receivers, which may be unnecessary and costly.

There are different methods of analysis for limit state design under the GFM or PFM. Common methods include empirical, limit equilibrium and numerical analyses. Selection of an appropriate method of analysis should consider the complexity and needs for the ELS works. For example, an empirical method is often simple and adequately robust for a shallow trench excavation. Numerical analysis is more suited to deep excavations and sites with complex ground conditions, where there is a need for more accurate estimates of performance of the ELS works.

This Chapter discusses the key design aspects of the GFM and PFM, the recommended safety factors to be adopted, and the considerations pertaining to selection of the DGWL for ULS and SLS design. Different methods of analyses for limit state design are also introduced.

6.2 Stability at Ultimate Limit State

The stability of ELS works should be assessed under the ULS design. Performance of the system should not permit a state at which a failure mechanism can occur in the ground or the ELS works (i.e. design requirement of ULS). The ULS design usually includes failure modes involving loss of overall stability, overturning or toe instability, base heave (in clayey soils), hydraulic failure (i.e. piping and uplifting), and structural failure.

Guidance on design of for ULS and SLS and ground deformation estimation are given in Chapter 7 and Chapter 8, respectively.

6.2.1 Loss of Overall Stability

The overall stability of ELS works can be enhanced by extending the embedded length of the wall (Figure 6.1). For an excavation on a steep slope, the overall stability should be checked against slope failure in accordance with the Geotechnical Manual for Slopes (GEO, 1984). In such cases, it is generally assumed that the potential failure plane will pass underneath the wall toe.

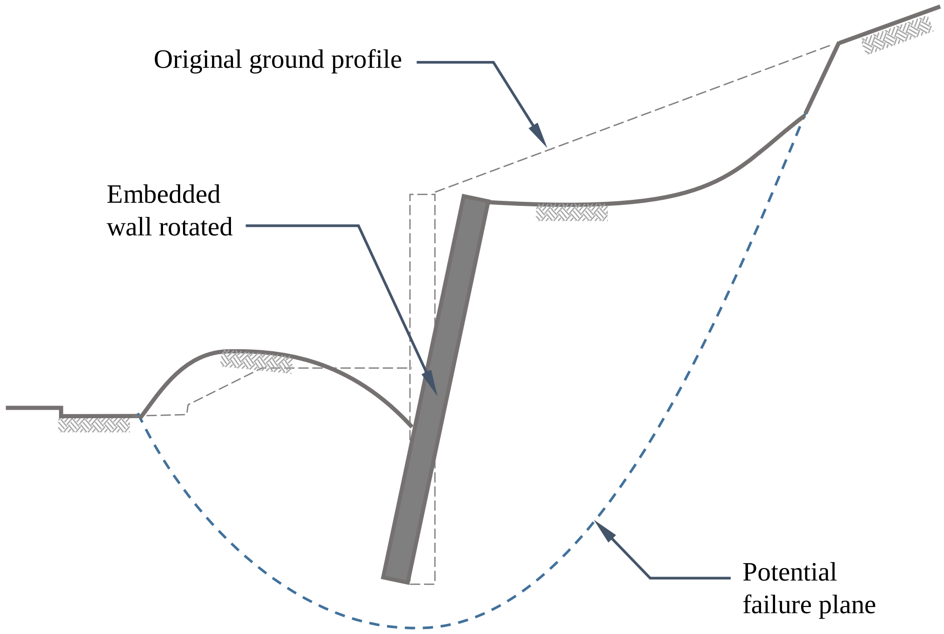


Figure 6.1 Loss of Overall Stability

6.2.2 Failure by Overturning or Toe Instability

Overturning or toe instability failure involves rotation of the wall at some point within the embedded portion for a cantilevered wall (Figure 6.2) or at the prop position for a strutted wall or tied-back wall (Figure 6.3). The embedded length and moment capacity of the embedded wall should be sufficient to prevent overturning or toe instability.

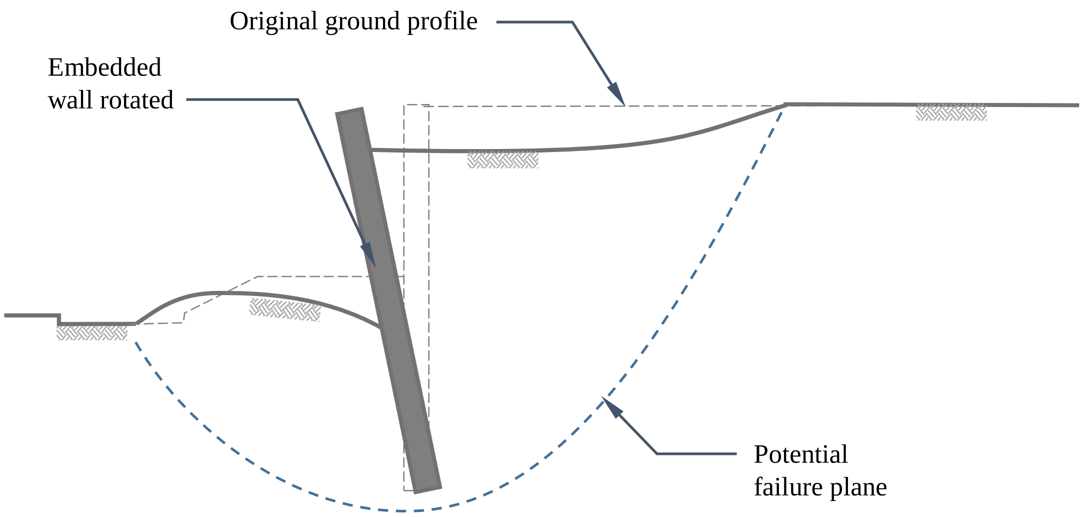


Figure 6.2 Failure by Overturning for a Cantilevered Wall

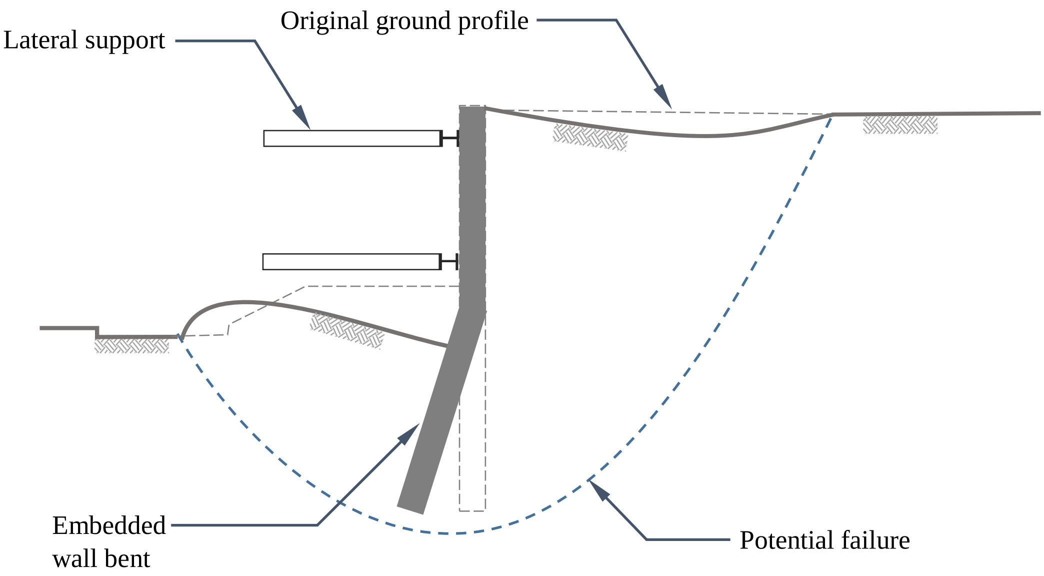


Figure 6.3 Failure by Toe Instability for a Strutted or Tied-back Wall

6.2.3 Failure by Base Heave

Base heave is a failure arising from the weight of soil outside the excavation zone exceeding the bearing capacity of soil at the excavation level, causing the soil to move and the base of the excavation to heave so much that it may cause the ELS works to collapse. Figure 6.4 shows the failure mechanism of base heave. It primarily occurs in ground conditions where soft clay extends to a considerable depth below the excavation base. Sufficiently deep wall embedment below the excavation level, or penetration of the wall base through the soft clay stratum and into firmer ground, should be provided in order to prevent base heave failure.

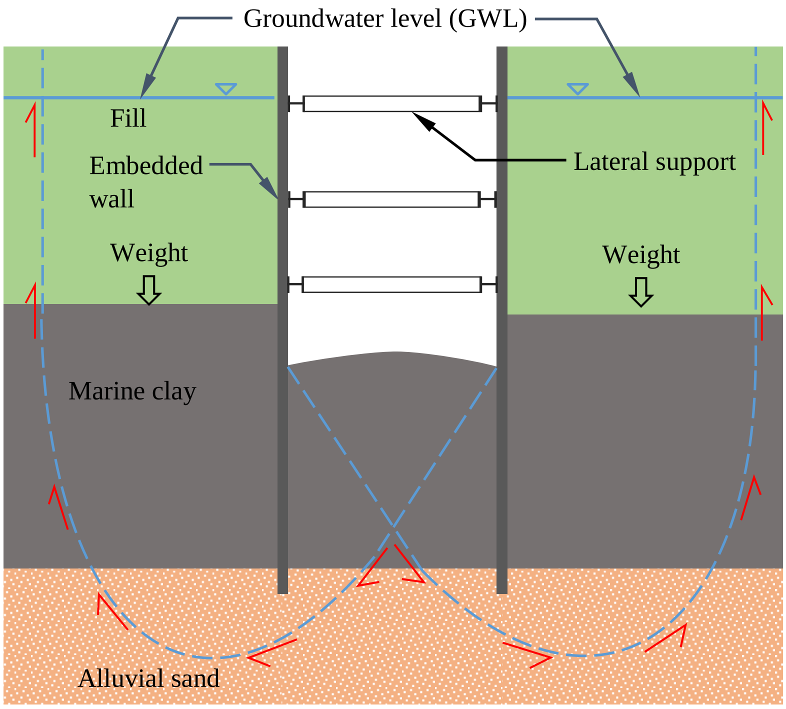


Figure 6.4 Failure by Base Heave

6.2.4 Hydraulic Failure

6.2.4.1 Failure by Piping

Piping failure occurs when upward seepage forces caused by groundwater flow into the base of an excavation reduce the effective stress in the soil to zero. Soil particles are then washed away by the upward seepage flow and the excavation loses the support provided by the passive soil resistance. Figure 6.5 illustrates the general mechanism of piping failure. Piping is more likely to occur when the excavation encounters a loose sandy layer with a high permeability. Where piping occurs, the resulting high seepage pressure may further erode soil material and cause sinkholes. In such cases, deeper embedment of the retaining wall should be provided to increase the length of the seepage flow path, so as to reduce the upward seepage forces and prevent the piping phenomenon.

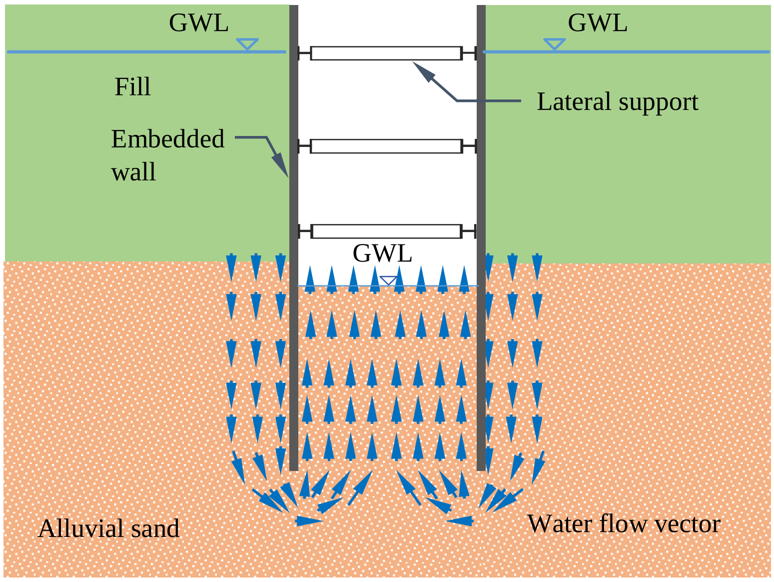


Figure 6.5 Piping Failure Mechanism

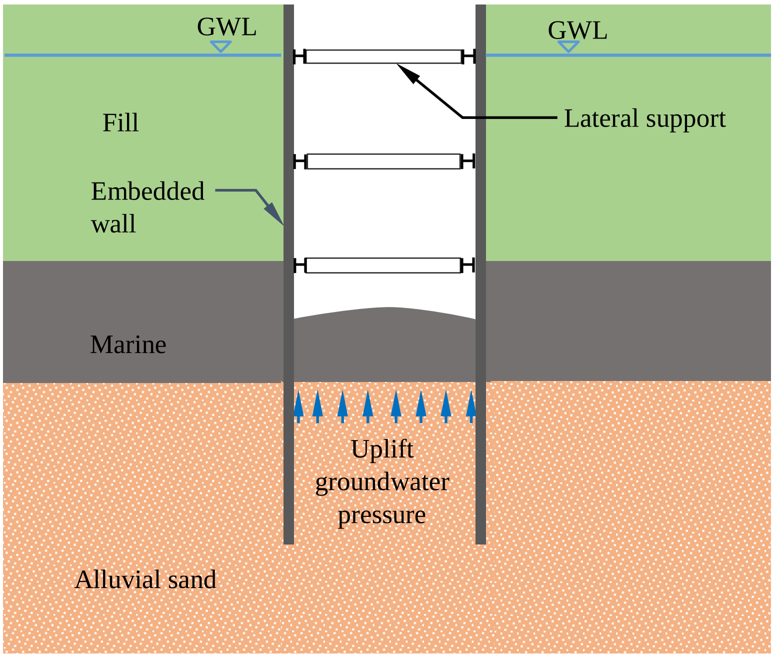


Figure 6.6 Uplifting Failure Mechanism

6.2.4.2 Failure by Uplifting

Failure by uplifting may occur when a layer of low permeability soil overlies a sandy soil within the excavation and when the artesian groundwater pressure under the low permeability layer exceeds the overburden pressure. The failure mechanism is illustrated in Figure 6.6. Such ground conditions are common in reclaimed land where soft marine clay is often underlain by alluvial sandy soils. Significant uplift forces may cause excessive upward movement at the excavated level and failure of ELS works. In such cases, either dewatering beneath the clay layer or forming pressure relief holes through the layer can be carried out to prevent uplifting.

6.2.5 Structural Failure

Structural design of ELS works should be carried out in accordance with the requirements of relevant structural codes and standards. The design of the strut layout and detailing should be sufficiently robust against structural failure, especially when the excavation involves irregular layouts and unbalanced loads.

6.3 Performance at Serviceability Limit State

The performance of ELS works should not permit a state at which ground deformation induced by the ELS works will affect the serviceability of nearby sensitive receivers (i.e. design requirement of SLS). Potential serviceability problems include unacceptable total and differential movement and cracking. The serviceability requirements are specific to sensitive receivers and should normally be agreed with relevant stakeholders.

6.4 Methods of Applying Safety Factors

The GFM has been used for many years due to its simplicity. On the other hand, the PFM usually requires additional design effort and review. However, use of the PFM may enable a more rational design (e.g. shorter wall embedment) to be prepared in some site settings as compared to the GFM, especially for deep excavations.

6.4.1 Global Factor Method

In a ULS design, the GFM adopts a single factor of safety. In a SLS design under GFM, a factor of unity is adopted to assess deflection of the embedded wall and associated ground deformation, and their impact on nearby sensitive receivers. The GFM has been widely adopted in Hong Kong and the recommended minimum global factors of safety are summarised in Table 6.1.

Table 6.1 Recommended Minimum Global Factors of Safety

Limit States		Minimum Global Factors of Safety
Ultimate Limit State	Overall instability	Refer to Geotechnical Manual for Slopes (GEO, 1984)
	Overturning(1)/Toe instability(1)	1.5 for effective stress analysis 2.0 for total stress analysis
	Base heave	1.5
	Hydraulic failure (i.e. piping and uplifting)	1.5
	Structural failure	1.4
Serviceability Limit State		1.0

Note: (1) The factor of safety against loss of moment equilibrium of the embedded wall should be applied on the passive earth pressure. Water pressure should not be factored.

6.4.2 Partial Factor Method

The PFM applies individual factors to loads and material properties commensurate with different types of uncertainty in the design of ELS works. The recommended partial factors are presented in the following sections, which are consistent with the recommendations given in Geoguide 1 for the design of retaining walls.

Factored values of loading and soil shear strength parameters, as defined below, should be used in design:

$$F_f = F \cdot \gamma_l \qquad (6.1)$$

$$X_f = \frac{X}{\gamma_m} \qquad (6.2)$$

where F_f and X_f = factored values of loading F and soil shear strength parameter X, respectively.
γ_l and γ_m = the partial load factor and partial material factor, respectively.

For typical soil shear strength parameters in design, the above general equations become:

$$\tan \phi_f' = \frac{\tan \phi'}{\gamma_m} \qquad (6.3)$$

$$c_f' = \frac{c'}{\gamma_m} \qquad (6.4)$$

$$s_{uf} = \frac{s_u}{\gamma_m} \qquad (6.5)$$

where c' and φ' = apparent cohesion and angle of shear resistance of soil respectively, in terms of effective stress.

s_u = undrained shear strength of soil in terms of total stress.
c_f' and ϕ_f' = factored apparent cohesion and angle of shear resistance of soil respectively in terms of effective stress.
s_{uf} = factored undrained shear strength of soil in terms of total stress.

The recommended minimum partial factors using the PFM are summarised in Table 6.2.

The s_u value for clayey material varies and depends on the quality of testing and method of interpretation. In general, a partial factor of 2.0 on s_u is recommended in Geoguide 1. In recent years, however, the CPT has become popular on sites where clayey materials are encountered. The CPT provides a continuous ground profile and generally gives better estimates of the in-situ shear strength than other discrete methods (e.g. vane shear test). If sufficient site-specific representative field tests (e.g. CPT calibrated with representative laboratory test results) are carried out, the minimum partial factor of s_u may be reduced from 2.0 to 1.5.

The partial factor for surcharge loading is recommended to be 1.3. This is based on the consideration that the surcharge imposed on ELS works of a temporary nature is generally more readily controllable on site to prevent overloading.

Table 6.2 Recommended Minimum Partial Factors

Strength Properties and Load Conditions		Ultimate Limit States	Serviceability Limit States
Partial material factor (γ_m)	Unit weight	1.0[(1)]	1.0
	Drained shear strength[(2)]	1.2	1.0
	Undrained shear strength	2.0[(3)]	1.0
	Shear strength of rock joint	1.2	1.0
	Soil and rock stiffness parameters	1.0	1.0
Partial load factor (γ_l)	Dead load	1.0	1.0
	Surcharge[(4)]	1.3	1.0
	Water pressure	1.0	1.0

Notes: (1) γ_m = 0.67 should be applied to the effective vertical stress which provides a stability effect for the hydraulic failure checks (i.e. piping and uplifting).
(2) γ_m should be applied to soil shear strength parameters of c' and tan ϕ'.
(3) γ_m may be reduced to 1.5 where sufficient site-specific representative field tests are carried out (e.g. CPT calibrated with representative laboratory test results).
(4) γ_l should be set to zero for surcharge which provides a stabilising effect.

6.5 Design Groundwater Level

Given the temporary nature of ELS works, DGWL should be related to possible scenarios that could occur within the duration of the works for different limit states. It is not necessary to consider the effects of long-term and extreme events (e.g. due to climate change). Based on local experience and practice, the following considerations are usually adopted in the estimation of DGWL for ULS and SLS design.

6.5.1 Design Groundwater Level for Ultimate Limit State

The DGWL for a ULS design should represent the highest groundwater level anticipated during the ELS works. The DGWL for ULS should be based on site-specific field measurement of groundwater levels and its assessment should consider factors such as the topography and hydrogeological conditions of the surrounding environment, possible presence of a perched water table and confined aquifer, and potential damming effects of the ELS works. The presence of soil layers with low permeability (e.g. clayey deposits) may result in a confined aquifer and the presence of perched water table. Hence, standpipes and piezometers should be carefully planned and installed at suitable depths to identify variations of piezometric pressures, which could differ from the hydrostatic condition.

Many projects in Hong Kong are executed under a fast-track programme and it is not uncommon for groundwater levels to be monitored only for a limited time. In Hong Kong, copies of all GI records are provided to the Civil Engineering Library for public inspection and a digital platform is available for disseminating the GI records, including GI logs, laboratory test results and monitoring data. Reference should be made to the groundwater monitoring records of previous GI carried out in the vicinity of the ELS works.

For ELS works with excavation depths greater than 4.5 m, it has become normal practice to adopt a DGWL for ULS by adding a rise of 1 - 2 m to the monitored highest groundwater level, and in circumstances where the monitored groundwater level is found to be lower than the excavation level, the DGWL for ULS is assumed to be at one-third of the excavated depth. These assumptions are found to be satisfactory for most excavation works taking into consideration their limited duration, unless the works will affect particularly sensitive structures. If warranted, a sensitivity check of the design using a range of DGWLs may be carried out to demonstrate the robustness of the ELS works, including damming effects. If it is anticipated that the groundwater level may be strongly influenced by seasonal variation or specific hydrogeological condition, e.g. excavation adjoining sloping ground with a large catchment area, the period of monitoring of groundwater levels should be suitably lengthened to cover such variation.

For shallower excavation, the DGWL for ULS can be assumed based on a similar approach, but usually with a smaller additional rise commensurate with the shorter construction time and the available depth in which the groundwater level can fluctuate beneath the ground.

In reclaimed land, the groundwater level is strongly influenced by tidal variations, with an attenuation and lag that depends upon the permeability of the filling material, storage capacity and horizontal distance from the shoreline. Laver (2021) observed that the influence of tidal fluctuation became unnoticeable in groundwater level measurements at about 100 m from the seafront of the West Kowloon reclamation. On another reclaimed site where rock fill was the predominant filling material, the attenuated tidal phenomenon was observable up to 300 m from the seafront. For sites in reclaimed land, the DGWL for ULS can generally be taken as the high tide level, with suitable allowances to cater for the site characteristics, e.g. the effects of storm surges and possible additional water infiltration into the ground. Some reclamations were formed to a level much higher than the mean seawater level (e.g. at +7 mPD). In such cases, it is not essential to assume the DGWL for ULS to be at the ground level.

6.5.2 Design Groundwater Level for Serviceability Limit State

The DGWL for a SLS design should represent a realistic estimate of the groundwater level under normal circumstances during the ELS works. The selected levels are usually based on site-specific monitoring data. If there is no site-specific groundwater monitoring, the DGWL may be based on recorded groundwater levels from available GI data in the vicinity, or on design experience in similar ground conditions, with suitable adjustments to cater for site-specific characteristics.

More than one DGWL should be considered in SLS design to cater for different scenarios. The design high groundwater level (DHGWL) for SLS should be based on realistic estimation of the highest groundwater level and used for assessing wall deflection caused by the excavation.

On the contrary, the design low groundwater level (DLGWL) for SLS should be based on the lowest recorded groundwater level. The lowest allowable groundwater level should be determined in the design to assess the acceptable ground settlement caused by groundwater level drawdown outside the excavation, which will exclude settlement that would have already occurred due to natural variation of the groundwater level over time.

Illustrative guidance for the determination of DGWL for ULS and SLS designs is shown in Figure 6.7. In addition, trigger values for response actions should be specified for monitoring the assumed DGWL, e.g. a high trigger value at 0.5 m below the highest DGWL for ULS, and a low trigger value at 0.5 m above the lowest DGWL for SLS as shown in Figure 6.7. If groundwater reaches these trigger values, the cause and any adverse effect to the nearby sensitive receivers should be investigated and a design review carried out with remedial works proposals if necessary, such as grouting and recharging. More details of the control mechanism for monitoring groundwater levels are presented in Chapter 9.

It should be cautioned that the DHGWL for SLS should not be overly conservative. For excavation works that involve preloading of struts to control wall deflection, reaction to the preloading force is provided by combined action of the soil and water on the retained side of the excavation. Obviously, a lower-than-expected groundwater level in reality will affect the wall deflection profile, particularly when preloading is taking place. The movement of struts at higher levels may be reversed due to preloading of struts at a lower level. Such relative movement should be properly allowed for in the design, including forces induced on the struts as well as the connections between structural elements.

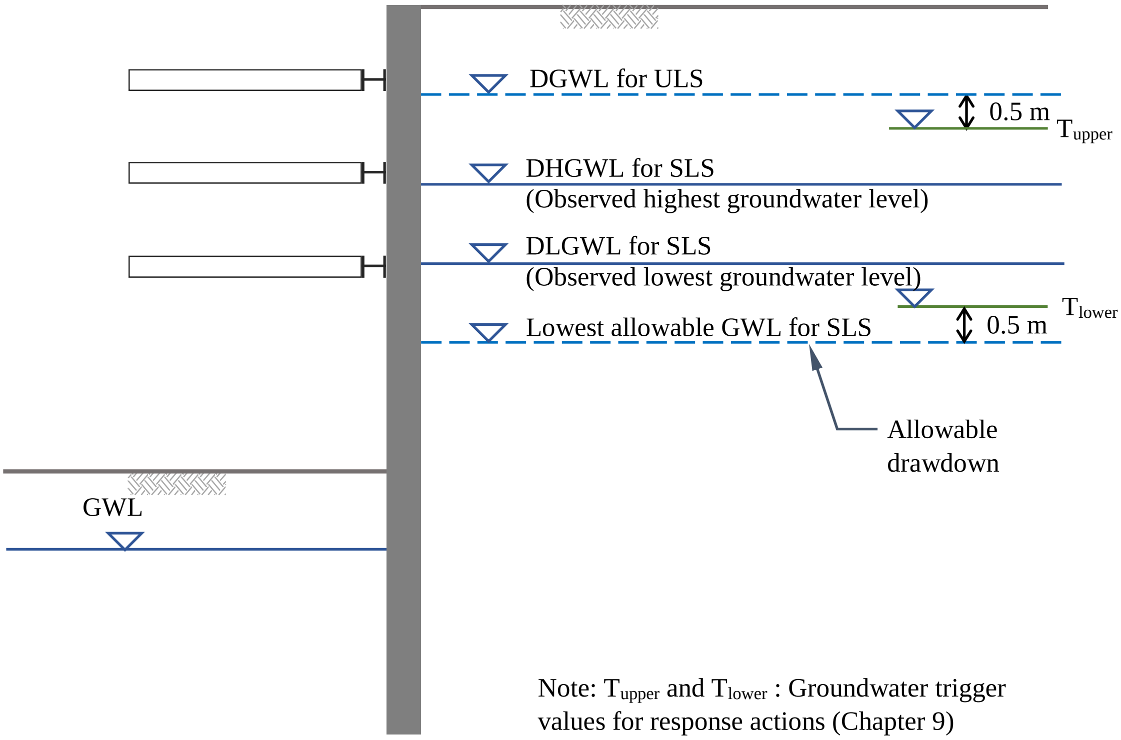


Figure 6.7 Illustrative Guidance for the Determination of DGWL

6.6 Methods of Analysis for Limit State Design

6.6.1 Empirical Method

Empirical methods for the design of ELS works are experience-based methods which have been successfully applied in previous cases and have withstood the test of time. They offer a quick and simple way to determine the adequacy of excavation support systems that can satisfy the limit states. However, they should only be applied within their areas of applications and known limitations such as scale of excavation, ground conditions and type of excavation support provided.

6.6.2 Limit Equilibrium Method

The limit equilibrium method is based on the conditions at collapse when the soil strength is fully mobilised in relation to a presumed failure mechanism. Design of the embedded wall is typically based on lateral earth pressure profiles, usually assumed to increase linearly with depth, and checks against moment and force equilibrium carried out for the assumed failure mechanism. Constant active or passive earth pressure coefficients for each soil stratum is usually applied and the design carried out either by hand calculations or a simple computer program.

The limit equilibrium method is simple and straightforward, and the calculations are often coded in computer spreadsheet programs. The input and output data are easy to check. The method can be used with confidence to calculate depths of wall embedment and the ULS effects where stress redistribution due to SSI is not significant. It can also provide an approximate check on the results of SSI analyses for more complex situations (Gaba et al, 2017).

However, the limit equilibrium method does not consider SSI and therefore cannot provide predictions of wall deflection, which is an essential part of the SLS design. It should be noted that wall deflection and rotation can lead to non-linearity of the lateral earth pressure, particularly in multi-level strutted excavations. In such cases, empirical or numerical methods are normally used to assess the ground deformation induced by the excavation and dewatering works. Where an SSI analysis is carried out, there may be scope for optimising the embedment depth and the bending stiffness of the wall.

6.6.3 Numerical Analysis

Numerical analysis has the advantage that it considers the SSI and the excavation sequence, including the soil conditions and behaviour (e.g. in-situ earth pressures, soil characteristics), changes of porewater pressure, stiffness of the embedded wall and the lateral support system. The deflection of the embedded wall and the deformation of the adjoining ground can be derived directly from the analysis. Commonly-used numerical methods include the beam on elastic foundation method, boundary element method and the finite element or finite difference method.

6.6.3.1 Beam on Elastic Foundation Method

In this method, the soil mass is modelled as a series of elasto-plastic springs in which the reactive pressure generated in each spring is assumed to be proportional to the wall deflection. Typically, the ground is discretised into a series of springs fixed at nodes and attached to beam or plate elements representing the embedded wall. In most computer programs, the springs are considered as independent and there is no interaction between adjoining springs. General guidance on the evaluation of spring constants (also called coefficients of subgrade reaction) is given in Geoguide 1. Use of this method does not provide direct prediction of ground settlement caused by the excavation and empirical correlations are then adopted to derive the ground settlement profile.

The beam on elastic foundation method can account for structural flexibility and soil stiffness. Thus, the effects of stress redistribution in the soil as a result of differential structural deflections are accommodated. However, it is not easy to select an appropriate spring stiffness and to simulate some support features, e.g. the initial stresses in the ground cannot be accounted for. It is also difficult to model the spring stiffness to reflect the effects of soil berms, raking struts and ground anchors, which rely on soil resistance remote from the embedded wall. The mobilisation of wall friction has a significant effect on lateral earth pressure and deflection of the wall, and such behaviour cannot be easily modelled in the spring model.

6.6.3.2 Boundary Element Method

The boundary element method assumes the wall to consist of a series of discrete beam elements attached to the soil at common nodes. The soil on each side of the wall is modelled as an elastic solid, and the wall deflection is generated either by integrals of the partial differential equations (e.g. the Mindlin solution), or by a pseudo finite element method (Pappin et al, 1985) (as illustrated in Figure 6.8).

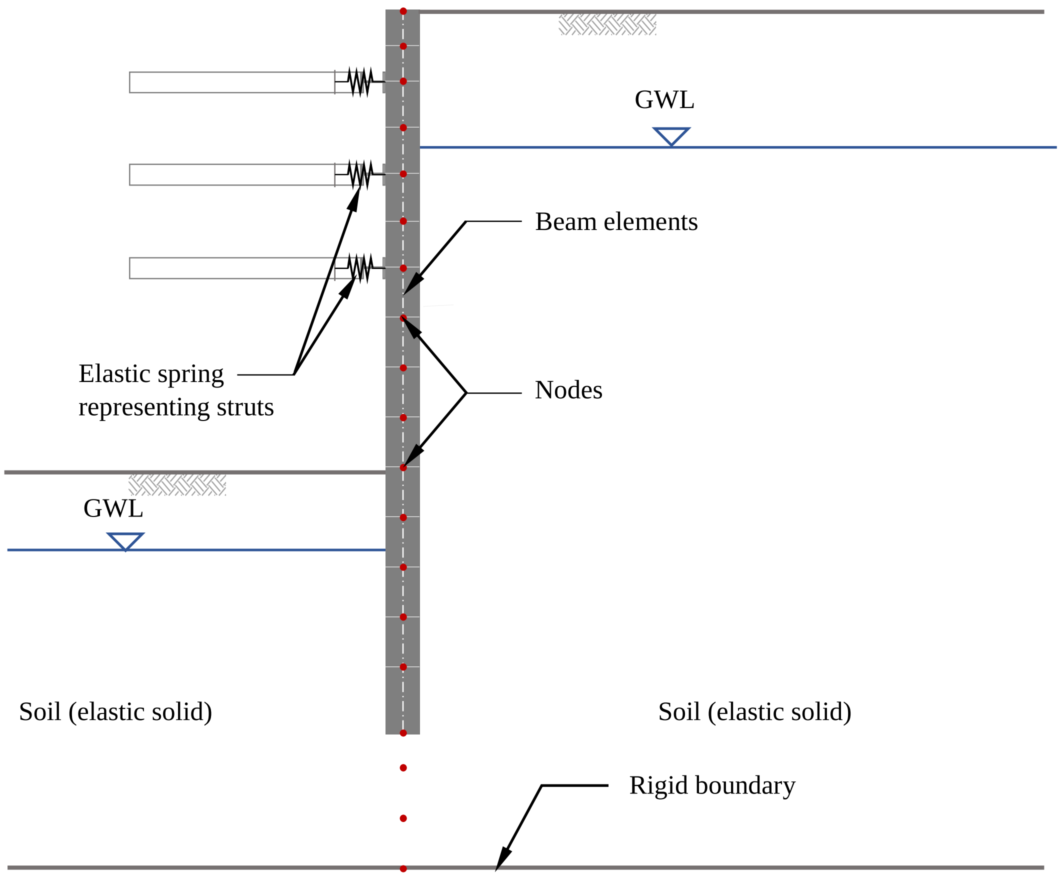


Figure 6.8 Pseudo Finite Element Method (modified from Pappin et al, 1985)

An iterative procedure is adopted to calculate soil reactions, which should always remain within the predefined active and passive soil pressure limits. At each excavation stage, the incremental movement is computed and summed up to give the overall wall deflection profile. In addition, soil arching and redistribution of lateral earth pressure can be considered in this method of analysis. In general, the boundary element method requires less computation time compared with the beam on elastic foundation method, although this consideration is less important nowadays with the advance of computing power. Modelling of excavations by this method is relatively simple and is often used in parametric studies. However, the method does not provide any prediction on ground deformation caused by an excavation.

6.6.3.3 Finite Element and Finite Difference Methods

Both the finite element method (FEM) and finite difference method (FDM) are numerical techniques used for solving problems in geotechnical engineering. They are popular in geotechnical modelling, as commercially-available computer programs for numerical analyses are becoming more versatile and user-friendly. No prior postulation of the failure mechanism or mode of failure is required, as the numerical model can predict them. Both methods are able to consider the soil behaviour (e.g. different soil constitutive models), stiffness and flexibility of the embedded wall and the lateral strutting system, as well as sequences of construction activities. The accuracy of both methods depends largely on using an appropriate constitutive model to represent the real soil behaviour, and on imposing the correct boundary conditions.

If numerical analysis is anticipated at the design stage, planning of the GI and laboratory tests should ensure that appropriate soil information is obtained, especially where advanced soil constitutive models are likely to be proposed (e.g. strain hardening model, small-strain stiffness model and high stiffness soil model).

The FEM is suitable for modelling complex geometries and boundary conditions. It involves discretising the ground into a large number of smaller, interconnected elements in the form of a continuum (Figure 6.9). The effects of loads and displacements are imposed on the boundary conditions, which are then solved to derive the stress and strain variables at the nodes of each element. The FEM is particularly well suited for solving problems involving nonlinearity, such as deformation, plasticity and creep.

The FDM, on the other hand, divides the ground into a grid of cells and solves for the unknown variables at the grid points. In terms of computing resources, the FDM is simpler and faster than the FEM, but it is less flexible and has limitations in terms of the types of problems it can solve.

Numerical modelling techniques, including modelling of compatibility, material constitutive behaviour and boundary conditions, have been discussed by Potts & Zdravkovic (1999; 2001), Lees (2016) and O'Brien & Higgins (2020). Shiu et al (1997) and Yau & Sum (2010) presented some useful applications of the FEM and FDM in local large-scale excavation projects. With the rapid advancement in computing power, the techniques have been extended to 3D numerical analysis (e.g. Orazalin & Whittle, 2016) for the modelling of more complex excavation and construction sequences. In any case, numerical analyses are often complex and it is important that the results are carefully scrutinised and examined, especially regarding the input parameters, and soil mechanics principles and assumptions that have been incorporated in the particular computer program. Non-convergence of solutions may be caused by errors in the element discretisation or numerical instability of the integration.

Details of the methods of analyses commonly used in current design practice are presented in Chapter 7 and Chapter 8, for design of ULS and SLS respectively.

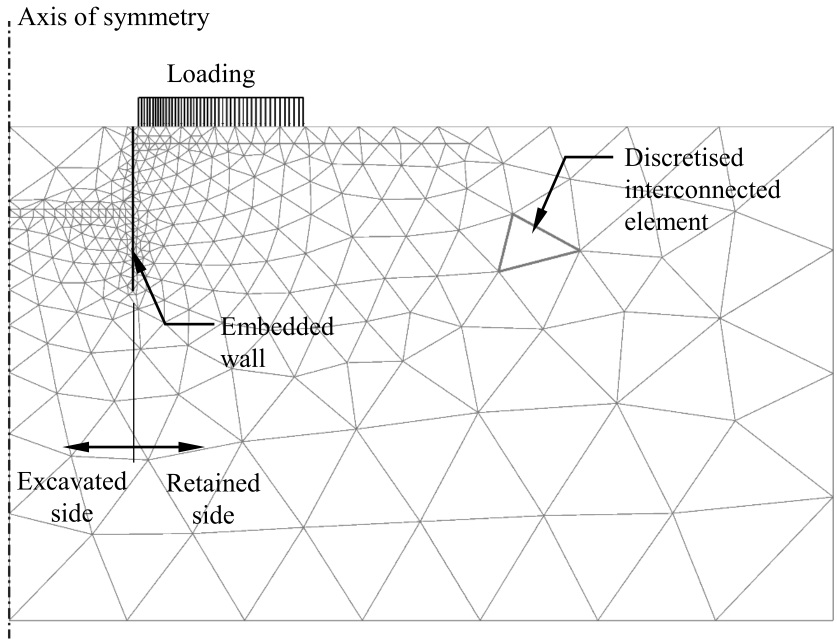


Figure 6.9 Illustration of Finite Element Modelling of Shaft Excavation (modified from Les et al, 2016)

7 Ultimate Limit State Design

7.1 General

Limit equilibrium and numerical analyses are the most common methods used in ULS design of ELS works. The use of empirical methods in ULS design is usually limited to the checking of base heave and hydraulic failure. Empirical methods are also applied in simple design of shoring support and drainages measures for trench excavations (e.g. UTLC, 2003). This Chapter provides guidance on ULS design for various modes of failure and support systems as described in Chapter 6.

7.2 Overall Stability

Limit equilibrium analysis using the method of slices is commonly used for the checking of overall stability of ELS works. This method usually assumes a potential slip surface which passes underneath the embedded wall toe. Loss of overall stability is likely to occur in excavations near a steeply-sloping site with a high groundwater table, or where a weak subsoil layer (e.g. loose sand or soft clay) is present below the embedded wall.

The methods of slices developed by Janbu (1972) and Morgenstern & Price (1965) are the most common methods used to check overall stability. However, experience has shown that for excavations involving abrupt changes in the ground profiles, the results of the analysis could be very sensitive to assumptions concerning the lines of action of the interslice forces and their inclinations to the horizontal. Therefore, the interslice force inclinations should be chosen conservatively, especially in the passive zone of the trial failure surfaces. Detailed guidance on the use of such methods is given in the Geotechnical Manual for Slopes. The safety factors as presented in Chapter 6 should be applied in the overall stability check.

The stability of slopes above and below the embedded wall should be considered in design if they are likely to be affected by the ELS works. Checking of slopes below the wall is particularly important since any loss of slope stability can lead to instability of the excavation. For a tied-back wall, interaction between the ground, wall and anchors should be considered as a complete system in the assessment of overall stability. In addition, sliding failure involving outward movement of the entire wall due to shearing along its base, or along a weak soil layer underneath the base, should also be checked, especially for a cantilevered wall.

Numerical analysis is seldom adopted in local practice for overall stability assessment due to the relatively large design effort required, unless more sophisticated soil constitutive behaviour needs to be modelled in the design of ELS works.

7.3 Overturning or Toe Instability

7.3.1 Cantilevered Wall

Similar to the case of overall stability assessment, limit equilibrium analysis is usually performed to calculate the factor of safety of a cantilevered wall against overturning or toe instability along assumed potential slip surfaces. The deflected shape of a cantilevered wall is illustrated in Figure 7.1, which shows the wall would typically rotate about a point O near its base in order to satisfy the force and moment equilibrium. The theoretical pressure distribution for this case is shown in Figure 7.2, when the wall is at a limiting condition. The pressure distribution shown is considerably idealised, particularly at the point of rotation, O, where it is assumed there is an instantaneous change from full passive pressure in front of the wall to full passive pressure behind the wall. Calculation of the depth of embedment corresponding to this pressure distribution involves equating the horizontal forces and taking moments about O in order to obtain two equations with two unknown depths, d and z (Figure 7.2), which are rather complicated, one being a quadratic and one a cubic expression in both d and z. The solutions for d and z are usually obtained by a process of iteration.

In view of the considerable algebraic complexity of the full method, the simplification illustrated in Figure 7.3 is widely used in local practice. It assumes that the difference between the passive resistance at the back of the wall and the active pressure in front acts as a concentrated force, R, at the toe. By taking moments about the toe (thereby eliminating R from the equation), the depth of embedment in the simplified model, d_o, is easily found. Because of this simplification, the value of d_o is slightly less than the value of d obtained from the full method and is more likely to be nearer to d - z/2. To account for this, it is common practice to increase d_o by up to 20%. A simple check of force equilibrium is then usually made to ensure that the additional embedment is sufficient to provide a force at least as large as the assumed force, R. This can be achieved from a simple consideration of force equilibrium. In most cases, a 20% increase in embedment (d_o) is applicable in common practice. This small additional increase in the value of d_o is specifically to account for the simplification, rather than to provide an additional factor of safety. Both the GFM and PFM as set out in Chapter 6 should still be applied in design.

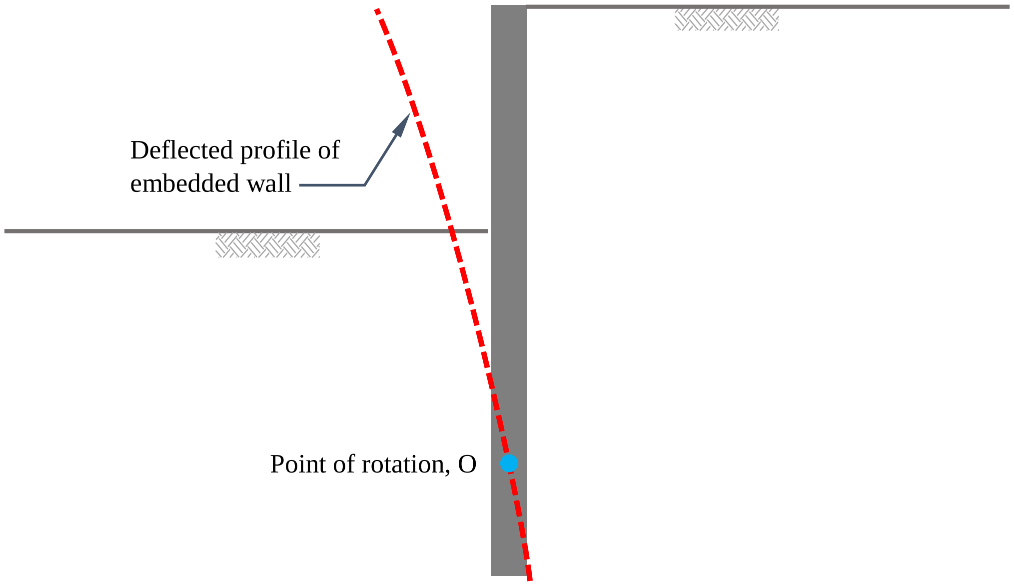
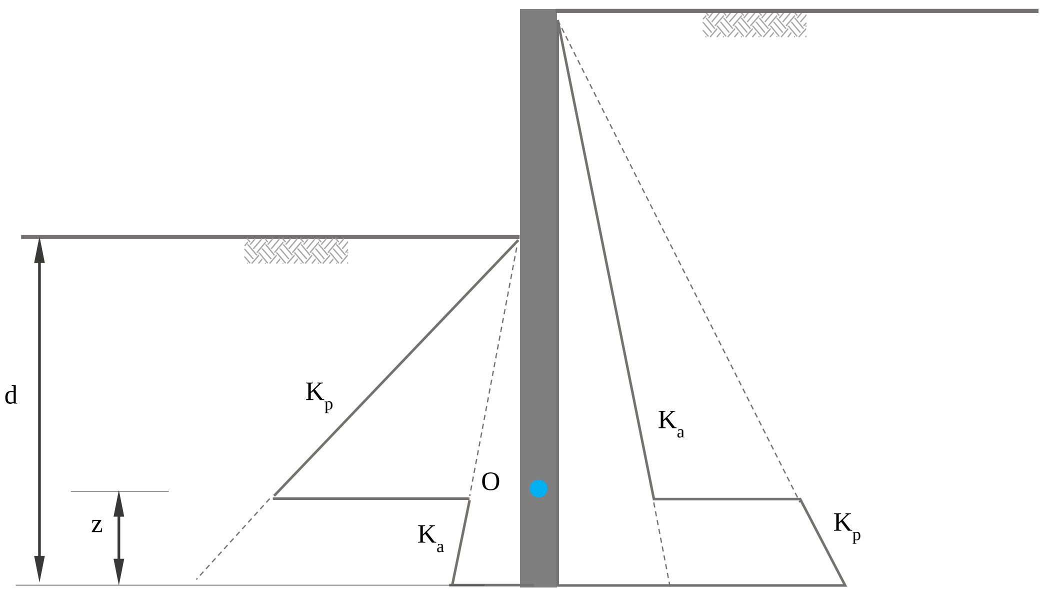


Figure 7.1 Deflected Shape of a Cantilevered Wall Based on a Fixed-earth Support Condition (modified from Padfield & Mair, 1984)



where K_a = Active earth pressure
K_p = Passive earth pressure

Figure 7.2 Soil Pressure Distribution on a Cantilevered Wall Based on a Fixed-earth Support Condition (modified from Padfield & Mair, 1984)

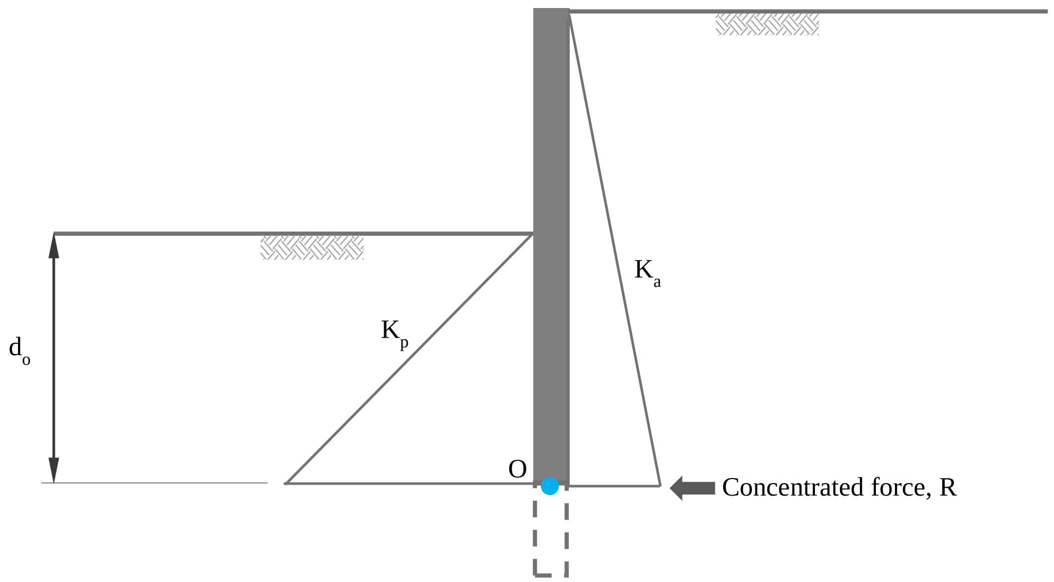


Figure 7.3 Simplified Soil Pressure Distribution on a Cantilevered Wall Based on a Fixed-earth Support Condition (modified from Padfield & Mair, 1984)

Stability checking of a cantilevered wall is usually simple and therefore it is common practice to perform limit equilibrium analysis in ULS design. Numerical analysis may be employed if significant SSI is envisaged in the checking of overturning or toe instability.

For a cantilevered wall socketed into rock, overall stability depends on the lateral load capacity of the rock socket. The method used to assess this load capacity was reviewed by in the rock mass should be considered, in addition to the bearing capacity of the intact rock. The latter is seldom the governing factor, unless the rock mass is highly fractured with closely spaced joints. The design approach and considerations in using the PFM for a rock-socketed cantilevered wall as given in Geoguide 1 are generally applicable. For the GFM, the resistances at the top of the rock socket (i.e. bending moment, M and shear force, V), should be sufficient to prevent overturning or toe instability under a ULS check. When applying the GFM in the design of a rock socket against planar discontinuity-controlled failure, global factors of safety against overturning and toe instability in Table 6.1 should be applied on the reactions at the top and bottom portions of the rock socket. In any cases, the rock socket should have a minimum embedment length of 1 m, so as to cater for the variation of rock head profile and possible disturbance due to construction of rock socket.

Inspection personnel are now generally prohibited from descending into bored piles to inspect rock mass conditions at the base. Therefore, when the use of a rock socket is anticipated a rock discontinuity survey should be conducted during the GI stage. Sufficient discontinuity surveys at different borehole locations should be carried out to establish the presence of any adversely-oriented planar discontinuities, together with their persistence, orientation and spacing. If there is no evidence of persistent adverse joint sets, the checking of planar discontinuity-controlled failure for rock socket design is not warranted. Simply assuming the presence of a planar discontinuity at the worst possible orientation, without evidence and justification, will lead to excessively over-conservative design (Cheung et al, 2023). The corresponding construction risk and associated movement caused by installing the wall to a greater depth should also be considered.

7.3.2 Single-level Strutted Wall

The minimum wall penetration required to safeguard against the loss of moment equilibrium of a single-level strutted wall, as for a cantilevered wall, is commonly assessed using either limit equilibrium analysis or a numerical analysis such as SSI using the FEM.

For limit equilibrium analysis, there is a choice between adopting a free-earth or a fixed-earth procedure for analysing the loss of moment equilibrium (Gaba et al, 2017). For a strutted wall, consideration of loss of moment equilibrium about the prop position is only applicable to free-earth support conditions, that is where there is insufficient embedment to prevent rotation of the toe of the wall (Padfield & Mair, 1984). Where a fixed-earth support condition applies, i.e. the embedment length is sufficiently long such that rotation of the wall toe becomes negligible, and provided that the wall is adequately propped and designed to resist the shear forces and bending moments, there is no failure mechanism relevant to a loss of moment equilibrium (Padfield & Mair, 1984). A multi-level strutted wall designed on the assumption of a fixed-earth condition is discussed in Section 7.3.3. For a free-earth support condition with no fixity developed at the wall toe, the depth of embedment is usually determined by taking moments about the position of the strut (Figure 7.4) to check the toe stability.

When using SSI analysis, the convergence of numerical analyses depends upon, among other factors, the number of iterations specified and the magnitude of any convergence tolerance specified in a particular numerical program. If excessive wall deflection has occurred in order to reach convergence, it may be considered that the wall has actually failed, despite the analysis reaching a point of convergence (Pickles, 2012). Therefore, caution should be exercised when the analysis shows a significantly deflected shape, signalling the system is on the verge of failure under the ULS condition. In practice, this often manifests as an increasing lateral wall deflection towards the toe of the wall. Therefore, in using numerical analysis for determining the required depth of wall embedment, the maximum lateral deflection of the wall should be designed so as not to occur at the wall toe.

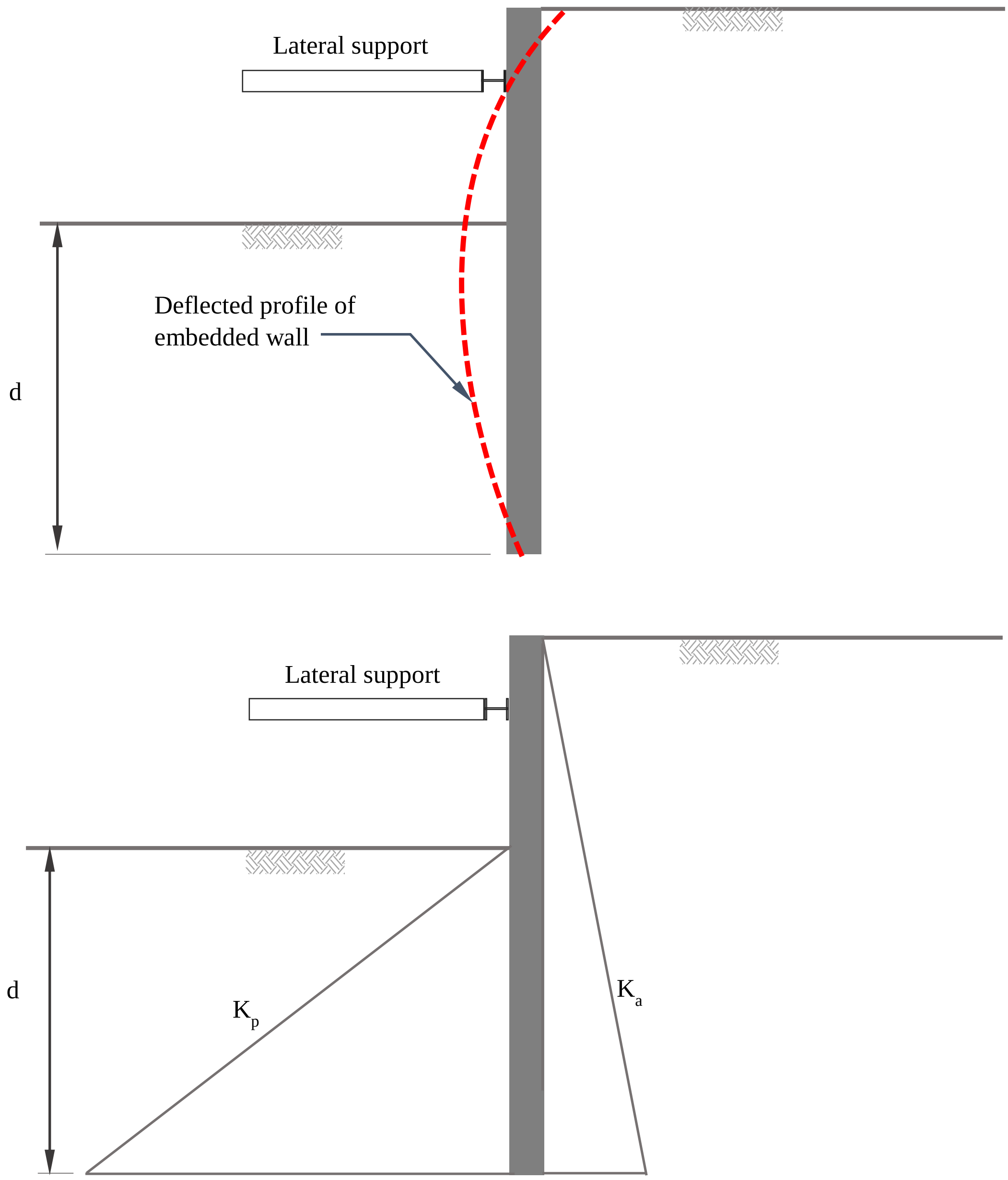
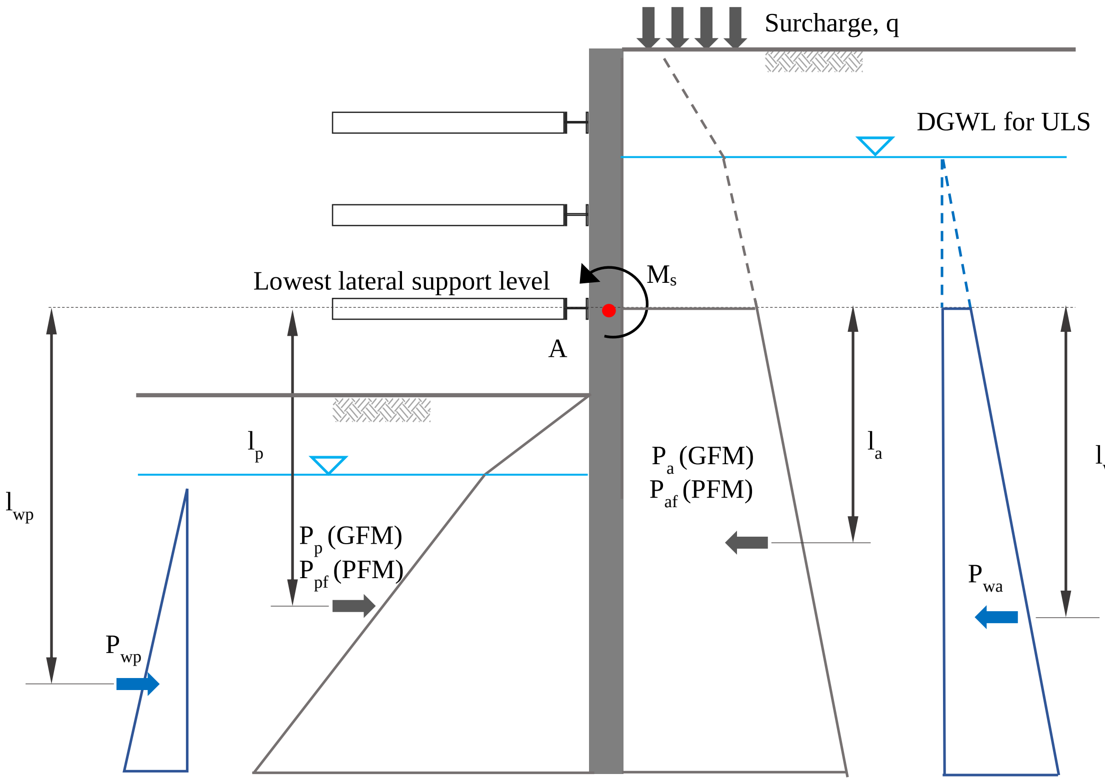


Figure 7.4 Free-earth Support Condition for a Strutted Wall (modified from Padfield & Mair, 1984)

7.3.3 Multi-level Strutted Wall

The minimum required wall penetration of a multi-level strutted wall can be determined either by limit equilibrium analysis or SSI analysis, similar to the checking for a single-level strutted wall. In a limit equilibrium analysis, a check should be carried out by considering the equilibrium of the free-ended span below the lowest strut, assuming fixity at that strut (Point A in Figure 7.5). When using limit equilibrium analysis with the GFM to determine the required embedment depth, the safety factor should be applied on the passive resistance of the excavated side. This is called the gross pressure method, with details as described by NAVFAC (1986b) and Ou (2006).



Notes: The penetration required is determined from the inequalities given in Equations 7.1 and 7.2, for the GFM and PFM respectively, by considering the equilibrium of the free-ended span below Point A, assuming fixity at A:

For the GFM in Equation 7.1, F_s is the global factor of safety against loss of moment equilibrium of the wall in Table 6.1.

For the PFM in Equation 7.2, the factored resultant forces due to active earth pressure (P_{af}) and passive earth pressure (P_{pf}) should be based on the recommended minimum partial factors in Table 6.2.

$$P_a l_a - \frac{P_p}{F_s} l_p + P_{wa} l_{wa} - P_{wp} l_{wp} - M_s \le 0 \quad \ldots\ldots\ldots\ldots \quad (7.1)$$

$$P_{af} l_a - P_{pf} l_p + P_{wa} l_{wa} - P_{wp} l_{wp} - M_s \le 0 \quad \ldots\ldots\ldots\ldots \quad (7.2)$$

where
- P_a = Resultant force (unfactored) due to active earth pressure below Point A
- P_{af} = Resultant force (with partial factors) due to active earth pressure below Point A
- P_p = Resultant force (unfactored) due to passive earth pressure
- P_{pf} = Resultant force (with partial factors) due to passive earth pressure
- l_a = Moment arm of resultant force P_a or P_{af} about Point A
- l_p = Moment arm of resultant force P_p or P_{pf} about Point A
- P_{wa} = Resultant force due to groundwater pressure on the retained side below Point A
- P_{wp} = Resultant force due to groundwater pressure on the excavated side
- l_{wa} = Moment arm of resultant force P_{wa}
- l_{wp} = Moment arm of resultant force P_{wp}
- M_s = Allowable bending moment of the embedded wall

Figure 7.5 Calculation of Embedment Depth of an Embedded Wall Below the Excavation Level (modified from NAVFAC, 1986b)

Numerical analysis using a finite element or finite difference method is more commonly adopted than limit equilibrium analysis for the ULS design of a multi-level strutted wall. Numerical analysis enables modelling of the entire construction sequence including installation and removal of lateral support at each stage. The bending moment profile of the embedded wall can be also calculated directly from an SSI analysis for ULS design on overturning failure and toe instability. Guidance on design considerations for the use of numerical analysis is given in Section 6.6.3 of Chapter 6.

7.3.4 Circular Shaft

The stability of a circular shaft mainly relies on development of the hoop action in compression acting between structural panels aligned in a circular profile, which is different to the stability of a strutted or tied-back wall provided by lateral support. Overturning or toe instability can only occur at individual panels within a circular shaft if they are not connected or aligned properly, which are aspects that should normally be assured during construction. The stability check is commonly carried out by numerical analysis to assess the tolerable deflection of the panels forming the circular shaft. Sometimes, ring beams may be required to minimise the panel deflection.

7.3.5 Tied-back Wall

The overturning stability of a tied-back wall supported by single level of tie-back is often assessed using limit equilibrium analysis, similar to the case of a single-level strutted wall as discussed in Section 7.3.2. For a wall supported by multiple tie-backs, numerical analysis is normally used to consider the significant SSI. In the case of a wall tied back by soil nails, design guidance is given in Geoguide 7. For the use of pre-stressed ground anchors, Geospec 1 presents the technical design standard used in local practice.

A tied-back wall should be designed to adequately resist the vertical component of the anchor forces induced by the inclined loads acting on the anchors (Figure 7.6). For a multi-level anchored wall, the vertical anchor forces could be significant, and therefore vertical stability of the wall becomes an important consideration. For this reason, it is common for a multi-level anchored wall to be founded on a hard soil stratum or rock to avoid excessive downward movement.

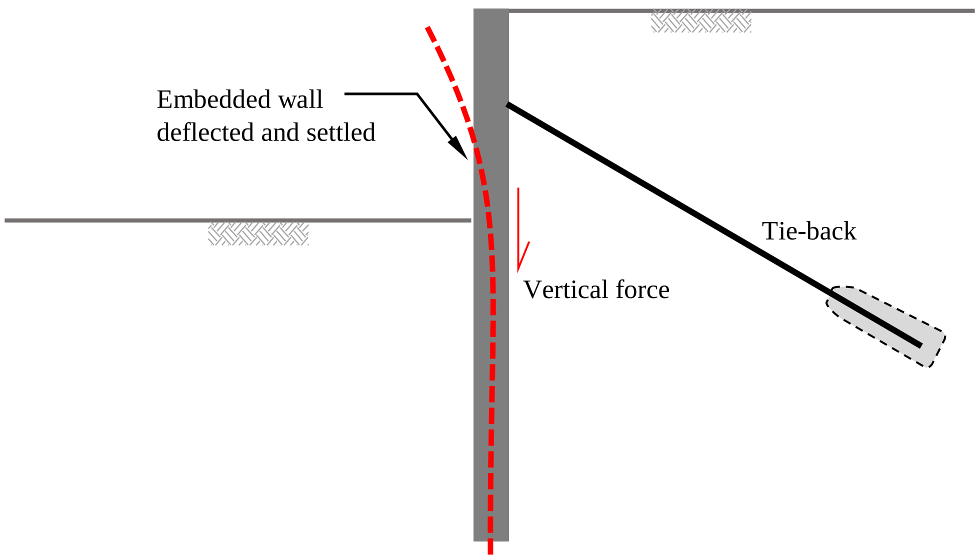
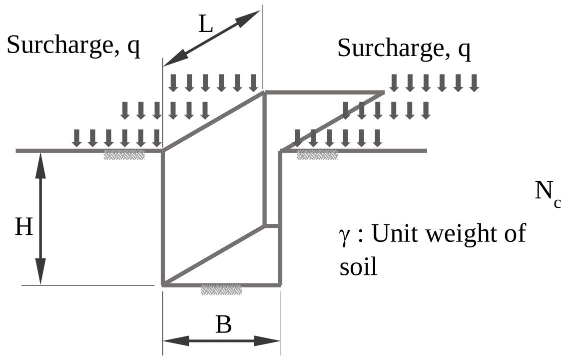
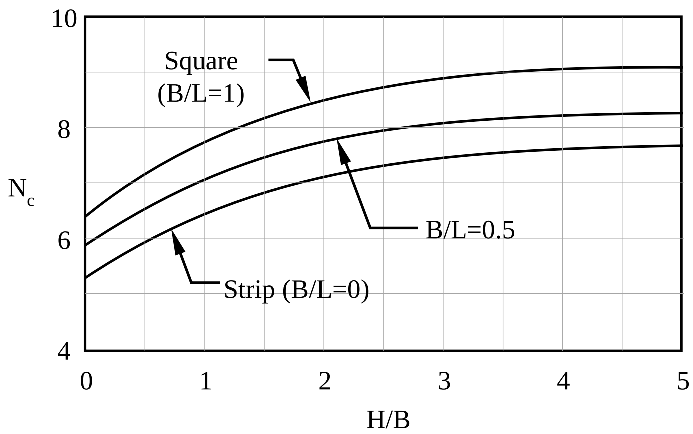


Figure 7.6 Vertical Stability Check for Tied-back Wall

7.4 Base Heave

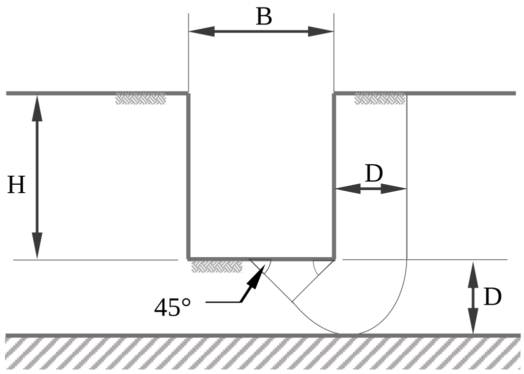
Two types of empirical analyses (Clough et al, 1979) are commonly used to check against base heave for the undrained condition, as shown in Figure 7.7. The method proposed by Terzaghi (1943) is applicable to shallow or wide excavations, where the excavation width, B, is larger than the excavation depth, H. For deep or narrow excavations where the excavation depth exceeds the excavation width, Terzaghi’s method may not yield reasonable results because it assumes that the failure surface extends up to the ground surface and that the soil shear strength is fully mobilised all the way to the surface. Neither of these assumptions are applicable to deep excavations, and thus the method of Bjerrum & Eide (1956) is more suitable (Ou, 2006). However, a disadvantage of all these empirical methods is that the internal friction between the soil and the embedded wall is ignored.

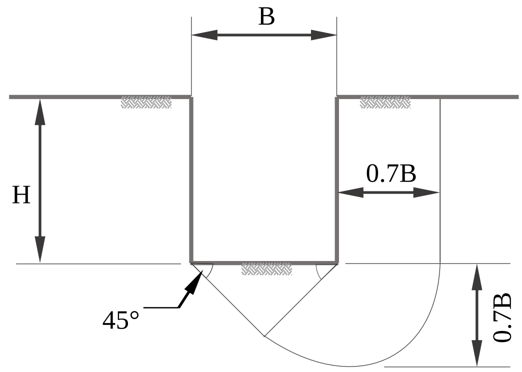




GFM: Factor of safety = $\dfrac{s_u N_c}{\gamma H + q}$ **PFM**: $s_{uf} N_c \geq \gamma H + q_f$

(a) For Deep Excavations with $\dfrac{H}{B} > 1$ (Bjerrum & Eide, 1956)





GFM: Factor of safety = $\dfrac{s_u N_c}{H\left(\gamma - \dfrac{s_u}{D}\right)}$ **GFM**: Factor of safety = $\dfrac{s_u N_c}{H\left(\gamma - \dfrac{s_u}{0.7B}\right)}$

PFM: $s_{uf} N_c \geq H\left(\gamma - \dfrac{s_{uf}}{D}\right)$ **PFM**: $s_{uf} N_c \geq H\left(\gamma - \dfrac{s_{uf}}{0.7B}\right)$

(b) For Shallow or Wide Excavations with $\dfrac{H}{B} < 1$ (Terzaghi, 1943)

where N_c = Bearing capcity factor
q_f = Factored surcharge

Figure 7.7 Methods of Base Heave Analysis in Fine-Grained Soils (modified from Clough et al, 1979)

Deeper wall embedment with higher stiffness will enlarge the potential failure surface and restrain base heave failures (Wong & Goh, 2002; Ou, 2006). The critical failure surface of the above two methods has a radius of about 0.7B. When the depth of the wall is not larger than 0.7B, these two methods can provide a reliable assessment of base heave stability. Otherwise, the methods might underestimate the factor of safety and yield conservative results if the part of wall beyond a depth of 0.7B is stiff enough to restrain wall lateral deflection.

In recent years, numerical analyses are more commonly used to assess base heave stability and the factor of safety, as well as to estimate ground deformation with consideration of SSI. Limit equilibrium analysis is less commonly used, as it does not consider the beneficial effects of the wall and soil stiffness.

7.5 Hydraulic Failure

7.5.1 Piping

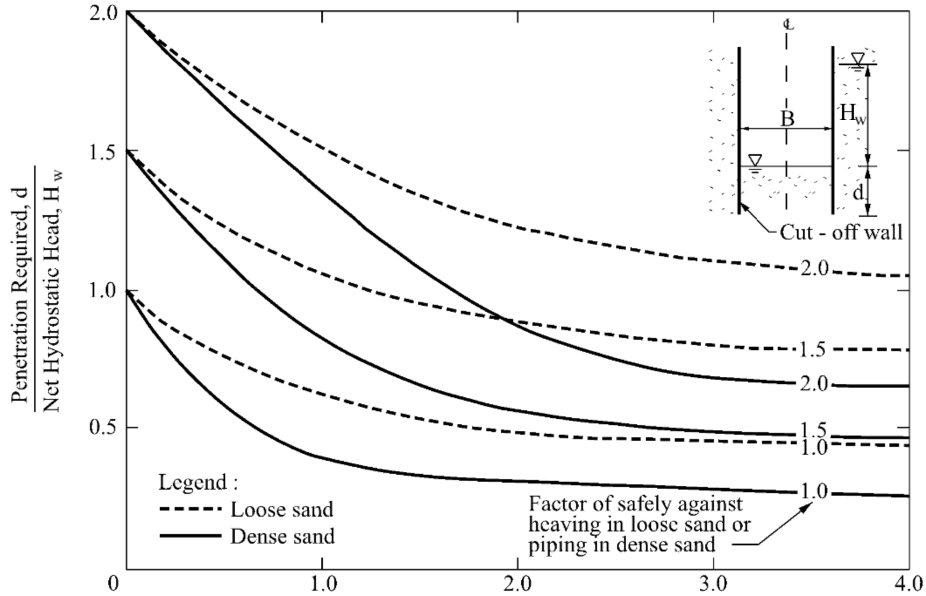
Empirical analysis using design charts is commonly adopted in local practice for checking the likelihood of piping failure. The wall penetration required for various safety factors against piping in homogeneous sands is given in Figure 7.8 (NAVFAC, 1986a). In the case of stratified subsoils, design charts involving empirical and analytical methods are given in Figure 7.9. The applicability of the charts in Figures 7.8 and 7.9 and the uncertainties related to the seepage analysis results should be assessed with due consideration of the ground conditions (e.g. soil layering and heterogeneous permeability), the site conditions and the geometry of the excavation. Furthermore, some design manuals also suggest that the empirical seepage exit gradient for a circular excavation, at the mid-section of the sides of a square excavation, and in the corners of a square excavation, is 1.3, 1.3 and 1.7 times that for a strip excavation respectively (e.g. Canadian Geotechnical Society, 2006). The shape effects of the seepage conditions become more significant in deep excavations with a large hydrostatic water pressure.

Apart from the use of design charts, an analytical method can also be used to assess the vertical equilibrium between the overburden and the hydraulic uplift force. An analytical method for checking piping failure is given in Figure 7.10. The vertical seepage exit gradient and the uplift force can also be assessed by performing seepage analysis using numerical tools. Guidance on seepage analysis is given in Section 7.5.3.

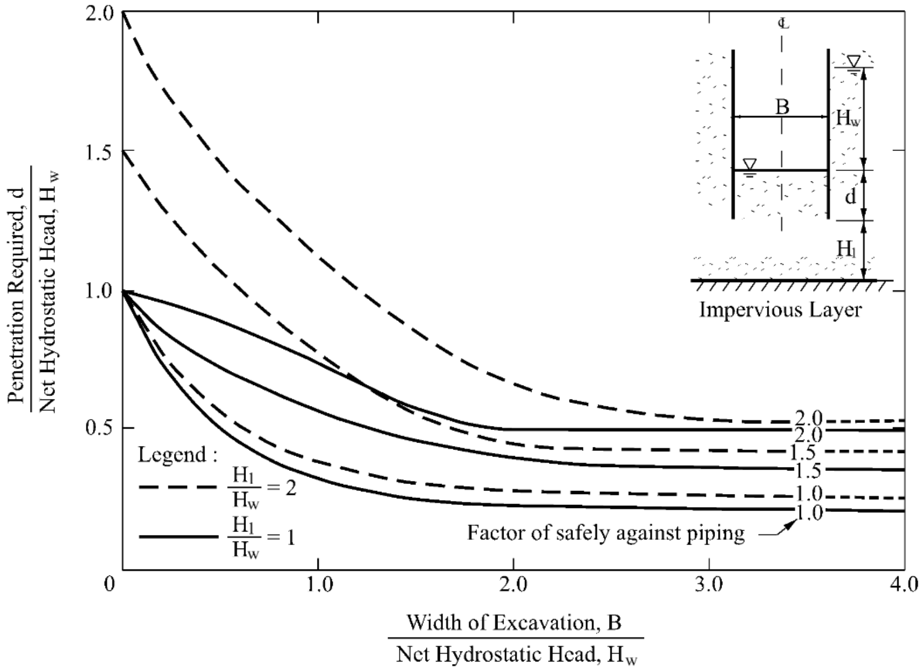
7.5.2 Uplifting

An analytical method is commonly used to check the likelihood of uplifting failure. However, in the absence of seepage analysis, the artesian pressure in the confined aquifer is unknown. For this case a simplified analysis could be undertaken following condition (c) in Figure 7.9 by conservatively assuming the groundwater pressure, which tends to lift the impermeable layer, to be the same as the hydrostatic pressure prior to dewatering.

If the artesian pressure is determined using seepage analysis, the analytical method given in Figure 7.11 may be followed to check against uplifting failure. Guidance on seepage analysis is given in Section 7.5.3.

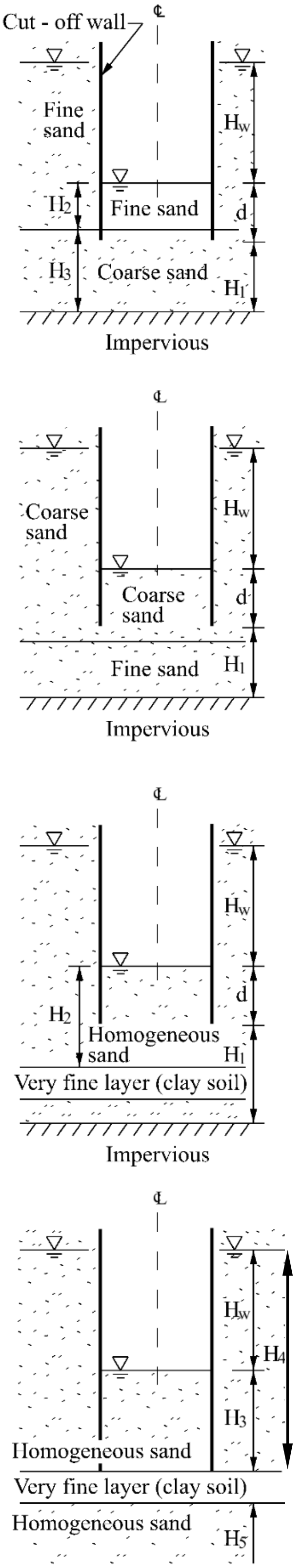


(a) Penetration Required for Cut-off Wall in Sands of Infinite Depth



(b) Penetration Required for Cut-off Wall in Dense Sands of Limited Depth

Figure 7.8 Penetration of Cut-off Wall to Prevent Hydraulic Failure in Homogeneous Sand (modified from NAVFAC, 1986a)



(a) Coarse Sand Underlying Fine Sand

- Presence of coarse layer makes flow in the fine material nearly vertical and generally increases seepage gradient in the fine material compared to the homogeneous cross-sections of Figure 7.8.

- If top of coarse layer is below toe of cut-off wall at a depth greater than width of excavation, safety factors of Figure 7.8(a) for infinite depth apply.

- If top of coarse layer is below toe of cut-off wall at a depth less than width of excavation, then uplift pressures are greater than that for the homogeneous cross-sections. If permeability of coarse layer is more than ten times that of fine layer, failure head (H_w) = thickness of fine layer (H_2).

(b) Fine Sand Underlying Coarse Sand

- Presence of fine layer constricts flow beneath cut off wall and generally decreases seepage gradients in the coarse layer. If top of fine layer lies below toe of cut-off wall, safety factors are intermediate between those derived from Figure 7.8 for the case of an impermeable boundary at (i) the top of fine layer, and (ii) the bottom of the fine layer assuming coarse sand above the impermeable boundary throughout.

- If top of fine layer lies above toe of cut-off wall, safety factors of Figure 7.8 are somewhat conservative for penetration required.

(c) Very Fine Layer in Homogeneous Sand

- If top of very fine layer is below toe of cut-off wall at a depth greater than width of excavation, safety factors of Figure 7.8 assuming impermeable boundary at top of fine layer apply.

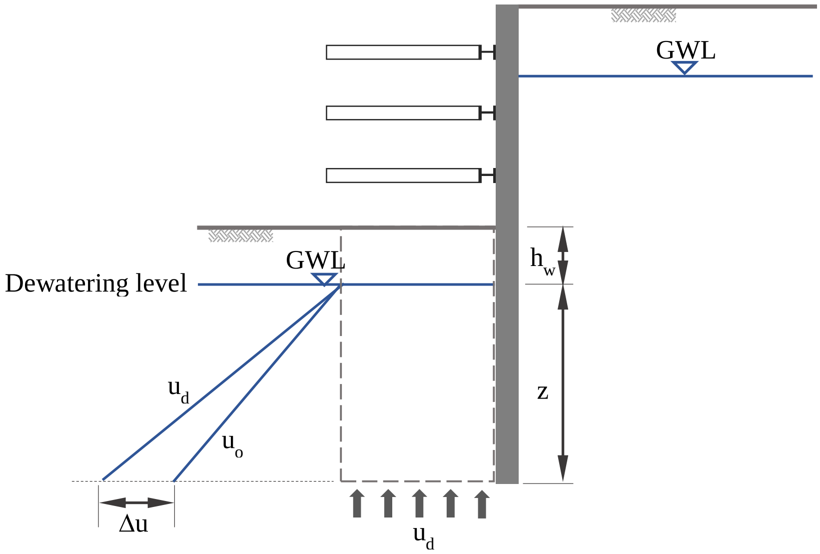
- If top of very fine layer is below toe of cut-off wall at a depth less than width of excavation, pressure relief is required so that unbalanced head below fine layer does not exceed height of soil above base of layer.

- To avoid bottom heave when toe of cut-off wall is in or through the very fine layer, ($\gamma_s H_3 + \gamma_c H_5$) should be greater than $\gamma_w H_4$.

  γ_s = Saturated unit weight of the sand
  γ_c = Saturated unit weight of the clay
  γ_w = Unit weight of water

- If fine layer lies above subgrade of excavation, final condition is safer than homogeneous case, but dangerous condition may arise during excavation above fine layer and pressure relief is required as in the preceding case.

Figure 7.9 Penetration of Cut-off Wall to Prevent Hydraulic Failure in Stratified Soil (modified form NAVFAC, 1986a)

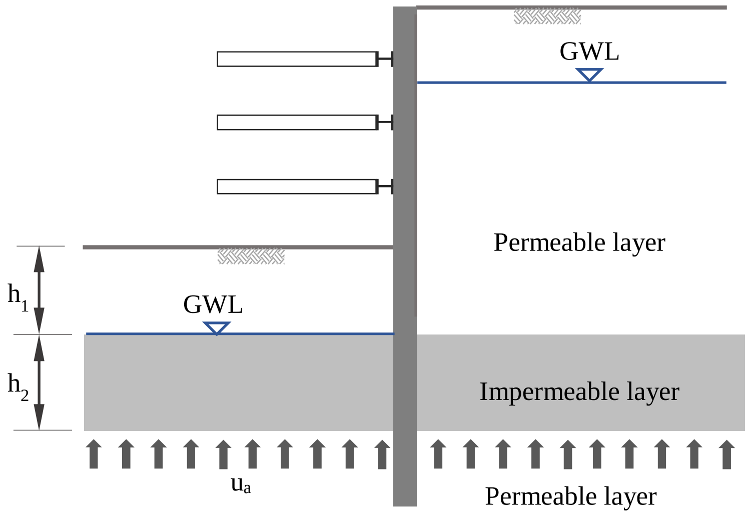


GFM: $\dfrac{\gamma h_w + (\gamma - \gamma_w)z}{\Delta u}$ ≥ Required global factor of safety in Table 6.1

PFM: $\Delta u \le \gamma_m [\gamma h_w + (\gamma - \gamma_w) z]$

where u_o = Groundwater pressure in the absence of flow
u_d = Design groundwater pressure in the presence of flow
Δu = Pressure difference
γ = Bulk unit weight of soil
γ_w = Unit weight of water
γ_m = Partial material factor in Table 6.2

Figure 7.10 Method of Piping Analysis (modified from BSI, 2022)



GFM: $\dfrac{\gamma_1 h_1 + \gamma_2 h_2}{u_a}$ ≥ Required global factor of safety in Table 6.1

PFM: $u_a \le \gamma_m (\gamma_1 h_1 + \gamma_2 h_2)$

where $\gamma_1 h_1 + \gamma_2 h_2$ = Total stress at the depth of $(h_1 + h_2)$ on excavated side
u_a = Groundwater pressure determined from seepage analysis

Figure 7.11 Method of Uplifting Analysis (modified from Ou, 2006)

7.5.3 Seepage Analysis

For seepage analysis, the groundwater pressure profile assessed in ULS should be used in an effective stress analysis. In addition, the hydraulic gradient and inflow rate can be assessed for checking against hydraulic failure and for the design of a dewatering proposal within an excavation. Numerical seepage analysis is normally applied for these assessments. A thorough understanding of the hydrogeological regime of the site, including groundwater conditions, groundwater recharge sources and boundaries, in-situ soil mass permeabilities, and the associated uncertainties is crucial when conducting a seepage analysis.

Seepage analysis may not be required if a hydrostatic water pressure distribution can be assumed for a wall embedded in soil with very low permeability (e.g. highly or moderately decomposed rock). Guidance on hydrostatic conditions relating to soil and rock characteristics, types of embedded wall and provision of a cut-off system is given in Chapter 4.

While a simplified method is available for a wall embedded in homogenous soil, such simplified groundwater pressure distribution may not be sufficiently accurate in other conditions, and a proper flow net analysis should be carried out by using established techniques described in treatises on groundwater flow (e.g. Cedergren, 1989).

7.5.3.1 Flow Net Construction

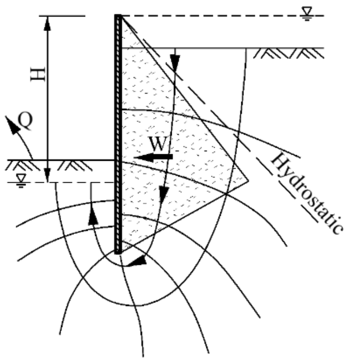
A flow net is a graphical representation of the continuity equation for steady-state seepage and is constructed for estimating groundwater flow and evaluating pressure head in the soil mass. The continuity equation in an isotropic medium is represented by two orthogonal families of curves, i.e. the flow lines and the equipotential lines. A combination of a number of flow lines and equipotential lines forms the flow net and are drawn in such a way that:

(a) the equipotential lines intersect the flow lines at right angles; and

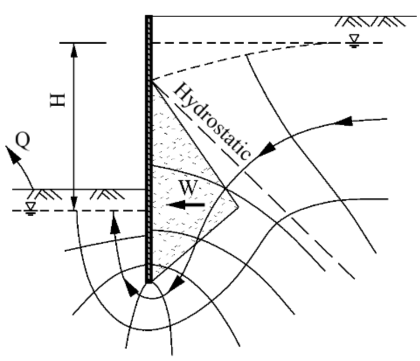
(b) the flow elements formed are approximate squares for soil with isotropic permeability.

Following these rules, the head drop from the retained side to the excavated side can be assessed, hence the groundwater pressure acting on the embedded wall and the seepage uplift pressure within the excavation can be calculated. The inflow rate can also be estimated by counting the numbers of potential drops and flow channels. The flow net can also be modified to take into account anisotropic soil permeability and multiple soil layers with different permeabilities. Further guidance on flow-net construction and the theory of groundwater flow is given in Cedergren (1989).

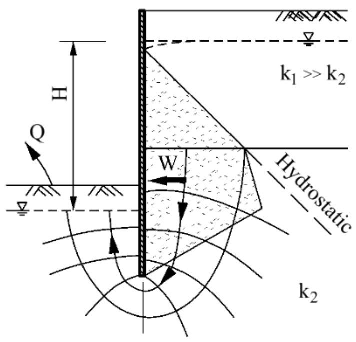
Kaiser & Hewitt (1982) discussed the factors influencing the flow pattern, including boundary conditions, anisotropy, relative permeabilities, impermeable layers, and the groundwater flow pattern, and some of the resultant groundwater pressures are illustrated in Figure 7.12. A flow-net around an embedded wall in homogeneous soil with a constant phreatic surface under steady-state seepage condition is shown in Figure 7.12(a).



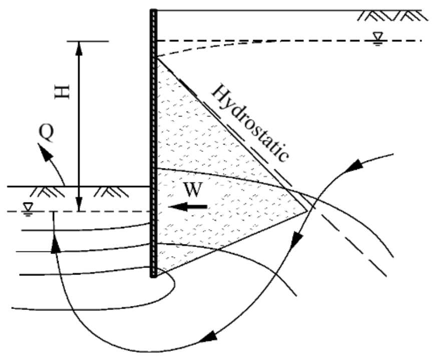
(a) Open Water



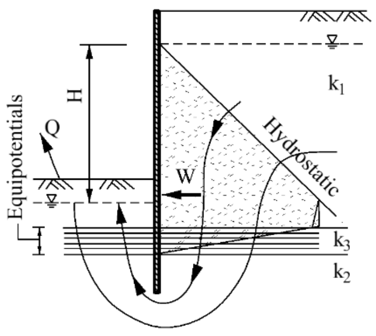
(b) Excavation below Groundwater Table



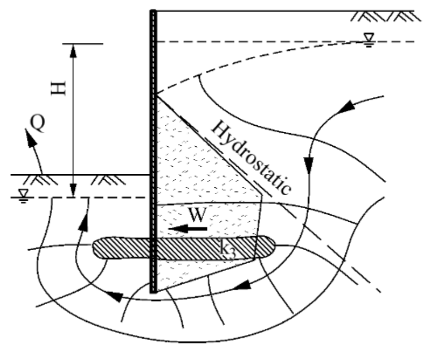
(c) High Permeability over Low Permeability



(d) Anisotropy $k_h > k_v$



(e) Low Permeability Lens $k_3 << k_1$, $k_3 << k_2$



(f) Discontinuous Lens of Low Permeability Material

where Q = Water flow
W = Groundwater pressure
k = Permeability

Figure 7.12 Groundwater Flow Patterns and Resultant Groundwater Pressures behind Excavations (modified from Kaiser & Hewitt, 1982)

7.5.3.2 Simplified Method

For a homogeneous isotropic soil under steady-state groundwater seepage, the simplified flow net shown in Figure 7.13 may be adopted for determining the groundwater pressure across a wall for design purposes. This simplified distribution assumes that the hydraulic head varies linearly along the flow path, i.e. down the back and up the front of the wall, and is sometimes denoted as the linear seepage method. In routine designs, it is often assumed there is no drawdown of the phreatic surface on the retained side of the excavation, and therefore this method is not used for estimating ground settlement associated with dewatering.

For sites with marked variation in soil hydraulic properties, the resultant groundwater pressures can exceed those developed in the homogeneous isotropic soil condition. The presence of pervious silt or sand partings within a clay stratum may also convey water at hydrostatic pressure to the toe of the wall (Padfield & Mair, 1984). In such cases, the simplified groundwater pressure distribution shown in Figure 7.14 may not be sufficiently accurate.

This simplified method may underestimate groundwater pressures below narrow excavations that have a width less than four times the differential head across the wall (Gaba et al, 2017).

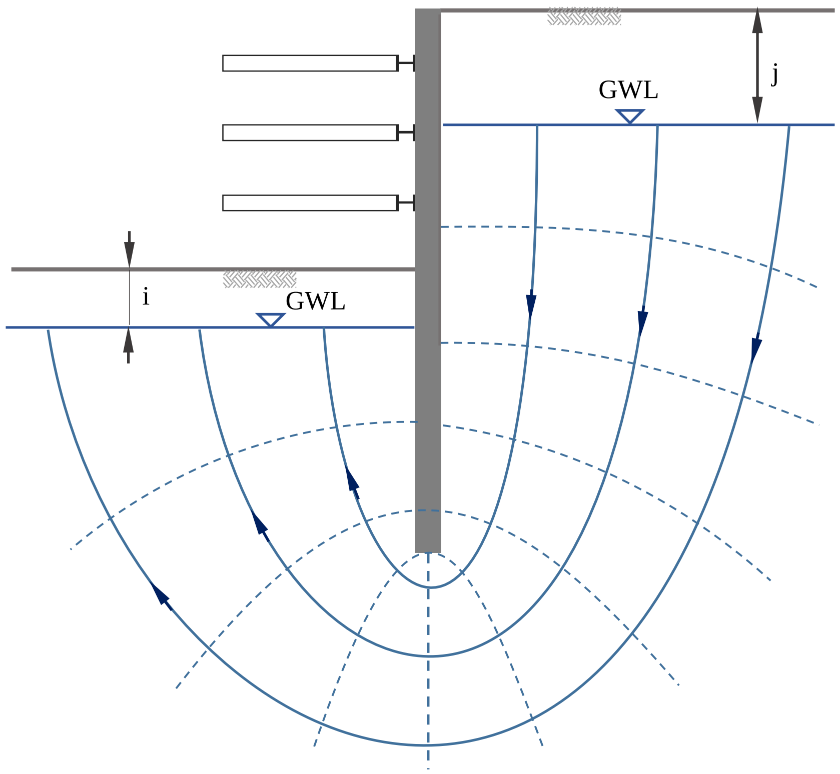
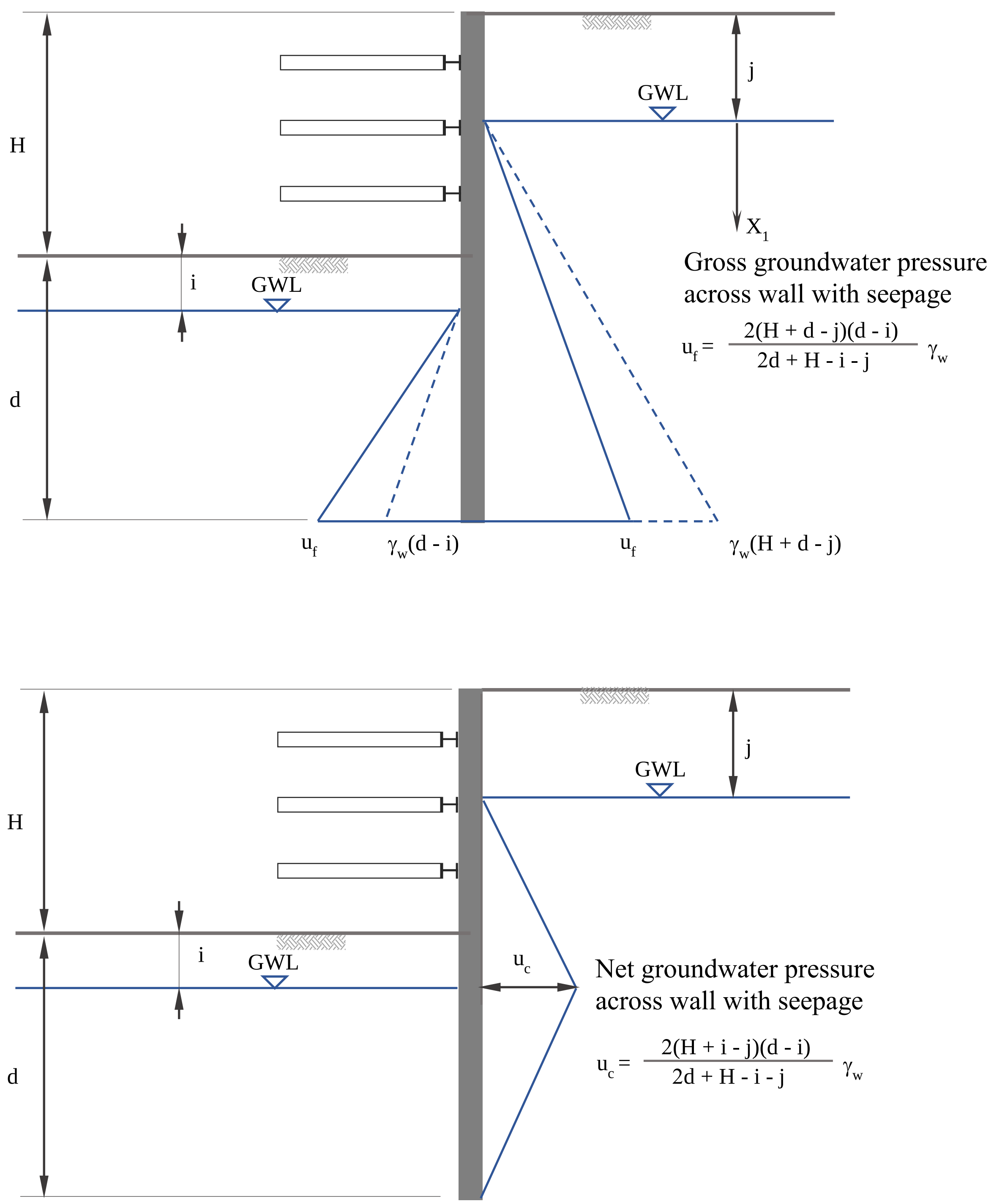


Figure 7.13 Flow Net and Pressure Distribution Across an Embedded Wall under a Steady-state Seepage Condition (modified from Padfield & Mair, 1984)



where u_f = Groundwater pressure at the toe of wall
u_c = Net groundwater pressure

Figure 7.14 Simplified Groundwater Pressure Distribution Across an Embedded Wall Under a Steady-state Seepage Condition (modified from Padfield & Mair, 1984)

7.5.3.3 Numerical Analysis

Numerical methods are available for seepage analysis and offer the possibility to assess groundwater flow and piezometric heads in more complex geological conditions that are difficult to deal with using a flow net. The analysis requires selection of permeabilities for various elements in the model space that will potentially affect the results, such as different soil/rock types as well as the cut-off system.

Soil and rock mass permeabilities assessed through in-situ permeability tests usually vary considerably. The choice of design permeabilities for the seepage analysis could be different for predicting groundwater pressures acting on the wall, inflow rate and groundwater drawdown. The hydraulic gradient across the excavation is also influenced by the ratio of permeabilities across different soil layers. Sensitivity checks on how the variation of design permeabilities affects the analytical results may be required.

The permeability of a grout curtain can vary with the ground conditions, grout mix and workmanship. The possibility of water flow or seepage through an embedded wall (e.g. a pipe pile wall with a grout curtain) should be considered, if only limited space is available for forming the grout curtain. An equivalent permeability value may be adopted for the embedded wall in the seepage analysis. Actual performance of the water cut-off ability of a grout curtain can be verified by regular monitoring during the construction stage.

Apart from permeability, another key input parameter in seepage analysis is the boundary distance, i.e. the distance from the excavation boundary where the groundwater regime will not be influenced, also sometimes referred to as the distance of influence. The site hydrogeological regime should be considered and engineering judgement should be exercised when defining the boundary distance. For example, if there is a source of groundwater recharge nearby, the distance of influence may be close to the embedded wall. CIRIA C113 (Sommerville & Large, 1986) provides guidance on the estimation of the distance of influence based on soil permeability, type of flow and water drawdown. However, it should be recognised that the formulae in CIRIA C113 were derived for sites of open underground pumping and the associated assumptions (e.g. homogenous soil condition) should be duly considered for the design of ELS works.

Typical boundary conditions for a water cut-off wall are shown in Figure 7.15. Where water inflow or seepage is possible through an embedded wall (e.g. a contiguous pile wall without a grout curtain), appropriate boundary conditions (e.g. zero groundwater pressure or a phreatic surface at the line of wall) should be assumed in the analysis (Gaba et al, 2017).

7.6 Structural Failure

Structural design of embedded walls, lateral support systems and details of connections should be carried out in accordance with the requirements of the relevant codes of practices on the use of structural concrete and steel. It should be noted that previous cases of under-design of connections in the strutting system has led to catastrophic failures (e.g. COI, 2005). For structural design of tied-back walls, reference should be made to Geospec 1 regarding the use of pre-stressed ground anchors. Guidance on structural design of soil nails used as tie-backs is given in Geoguide 7.

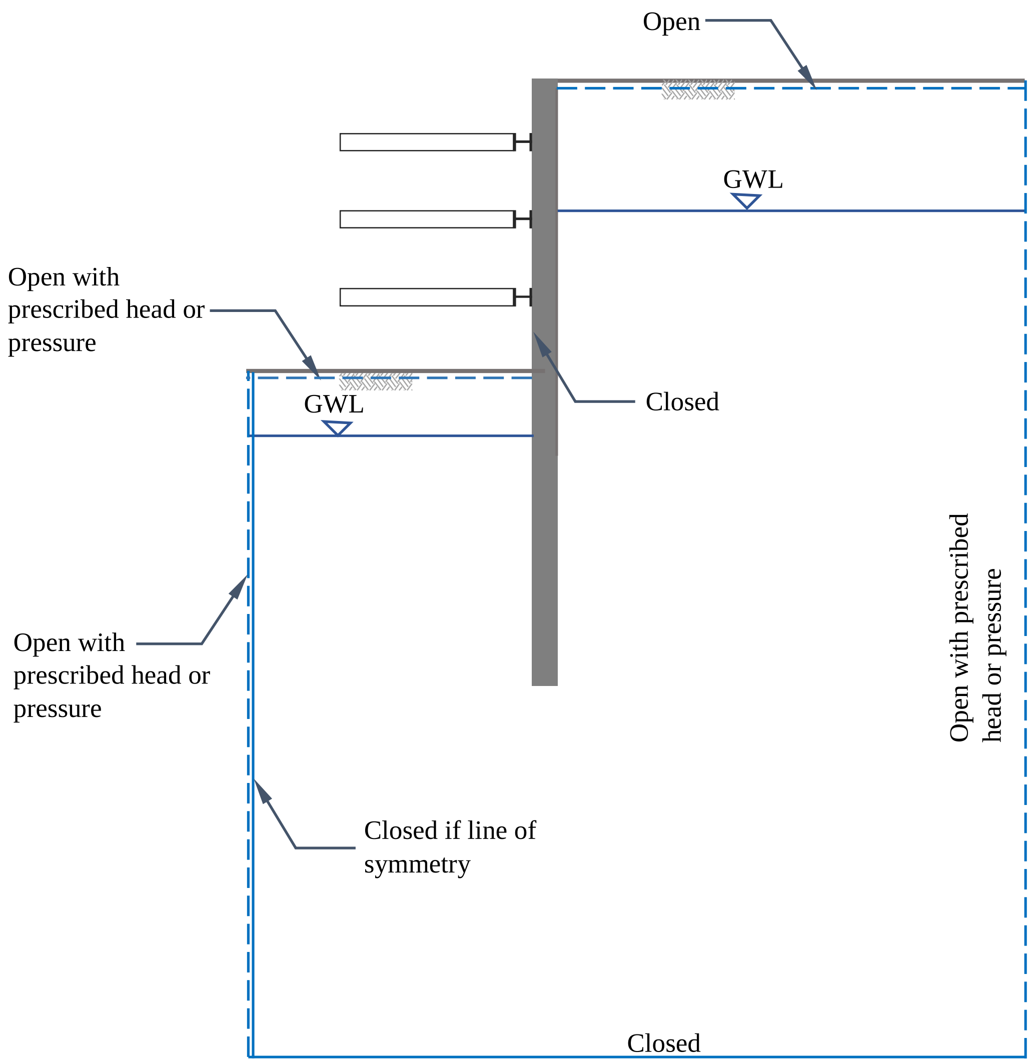
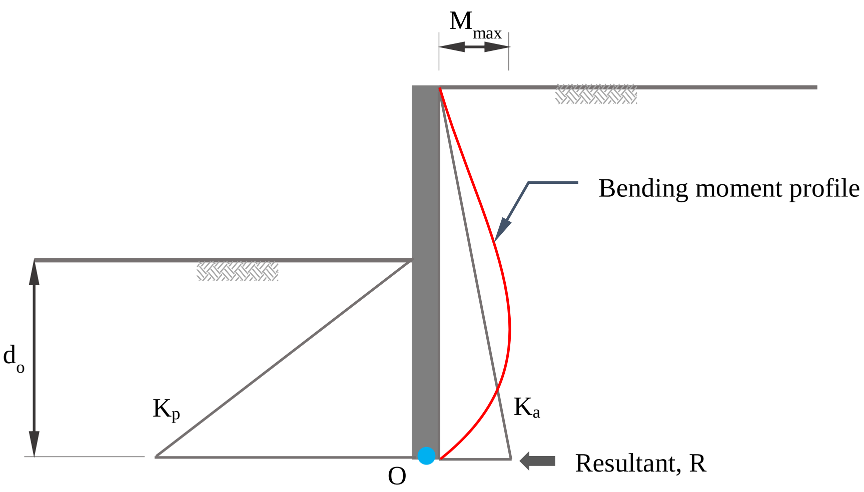


Figure 7.15 Typical Hydraulic Boundary Conditions (modified form Lees, 2016)

7.6.1 Cantilevered Wall

When limit equilibrium analysis is applied for the structural design of a cantilevered wall, the bending moment profile under working conditions can be simplified by using an assumed linear lateral pressure distribution as shown in Figure 7.16. However, the reinforcement should not be curtailed at the point where the calculated bending moment is zero using this simplified method. The reinforcement bars should be provided down to the bottom of the cantilevered wall, and on both faces, to allow for small reverse bending moments which may occur near the toe as shown in Figure 7.17.



where M_{max} = Maximum bending moment

Figure 7.16 Assumed Linear Soil Pressure Distribution for a Cantilevered Wall
(modified from Padfield & Mair, 1984)

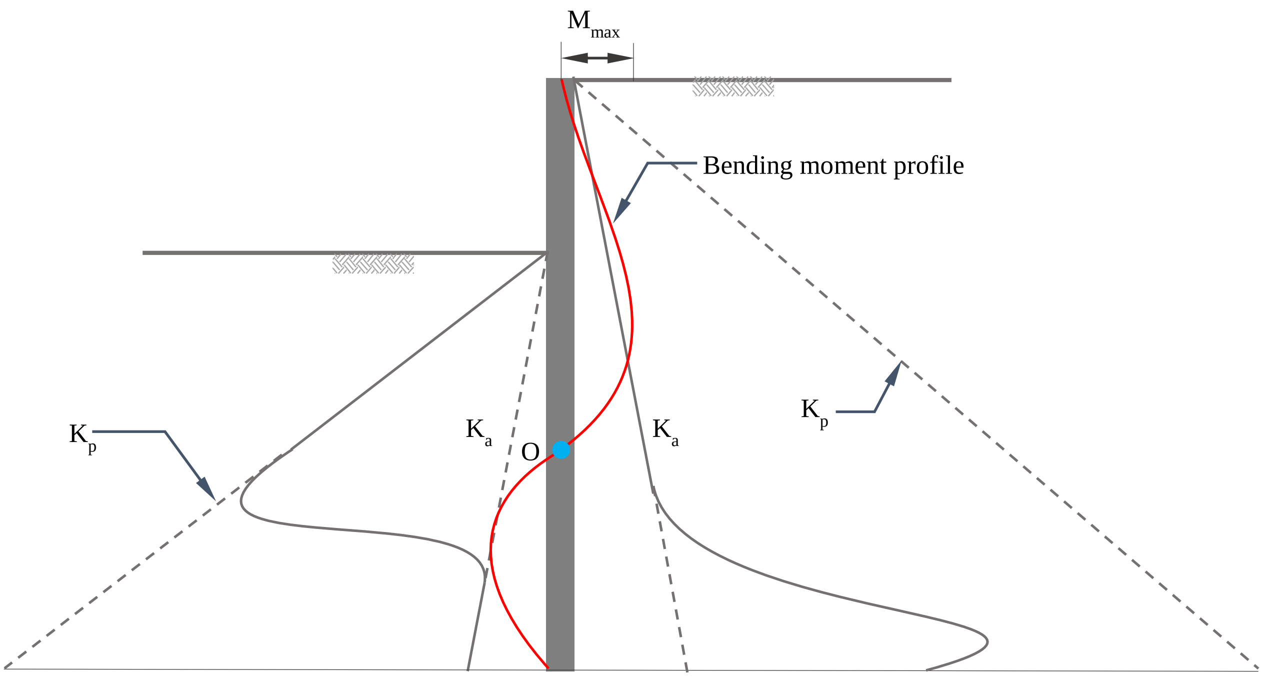


Figure 7.17 Calculation of Maximum Bending Moment in a Cantilevered Wall
(modified from Padfield & Mair, 1984)

7.6.2 Single-level Strutted or Tied-back Wall

Similar to the case of a cantilevered wall, the assumed linear lateral pressure distribution of a strutted or tied-back wall is shown in Figure 7.18. Similarly, bending moment and shear force profiles for a single-level strutted wall can also be solved by simple limit equilibrium analysis (Figure 7.19). However, if limit equilibrium analysis is used for the structural design, the lateral force is usually increased by 25% to allow for the possibility of arching and stress redistribution behind the wall (Padfield & Mair, 1984). Similarly, for a single-level tied-back wall, any load of tied-back should be also increased by 25% behind the wall.

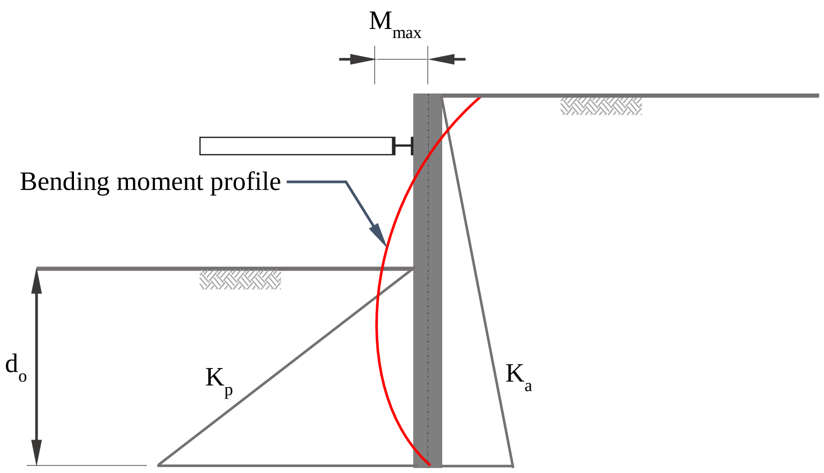


Figure 7.18 Assumed Linear Soil Pressure Distribution for a Strutted or Tied-back Wall (modified from Padfield & Mair, 1984)

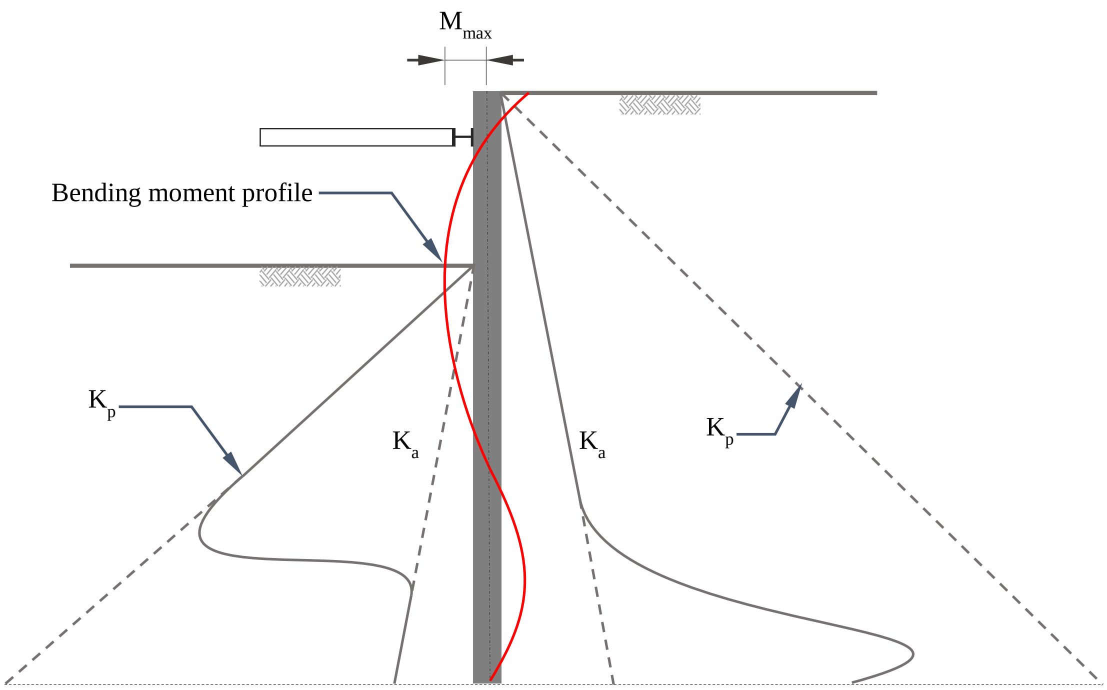


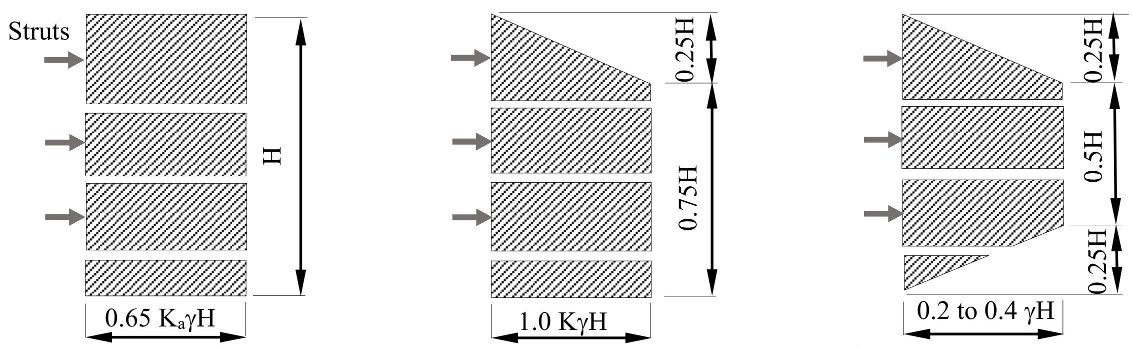
Figure 7.19 Calculation of Maximum Bending Moment in a Strutted or Tied-back Wall (modified from Padfield & Mair, 1984)

7.6.3 Multi-level Strutted or Tied-back Wall

The loads resisted by the struts in a multi-level strutted or tied-back excavation depend on two governing factors. The first factor is the magnitude of the total active earth pressure exerted by the soil behind and beneath the wall, down to the depth where soil deformation affecting the wall is no longer significant. This depends exclusively on the shear strength and the unit weight of the soil behind the wall. The second factor is the distribution of the earth pressure, which determines how much of the total active earth pressure will be carried by the struts. This distribution depends on the amount of arching and is controlled by the magnitude of deformations in the soil beneath the excavation relative to those of the struts.

A multi-level strutted wall performs differently at each stage of construction (e.g. wall deflection profiles vary during installation and removal of struts). It is necessary to consider each excavation stage in order to determine the maximum structural load. Also, the effect of any unbalanced horizontal loading across the excavation should be considered. In this regard, limit equilibrium analysis is not recommended for the structural design of a multi-level strutted wall as it is statically indeterminate and the earth pressure distribution cannot be determined by classical theories (e.g. Rankine and Coulomb theories). Numerical analysis (e.g. use of a boundary element method or FEM with simulation of the staged construction sequence) is more suitable as it considers both SSI and stress redistribution. It is also capable of evaluating the cumulative effect of incremental changes in stresses and strains that occur during each stage of construction.

For shallow excavations that do not affect sensitive utilities and buildings, a semi-empirical method such as the apparent pressure method can also be used to estimate strut loads. The method is derived from measured strut loads. Envelopes of maximum design pressure have been developed for different soil types by distributing the measured strut loads over a ‘tributary area’. Apparent earth pressures incorporate many factors, including soil type, the support system used and the construction sequence (Canadian Geotechnical Society, 2006). Semi-empirical envelopes were developed by Terzaghi & Peck (1967), which were later summarised by Peck (1969) (Figure 7.20). Zhang & Liu (2021) also reported the use of the apparent pressure method to provide conservative estimates of earth pressures and estimates of the maximum strut loads for design purposes. For multi-level tied-back wall, prestressing loads applied to tie-backs might be higher than the upper limit values designed for strut loads by apparent pressure diagrams as reported by Clough (1975), and the diagrams as shown in Figure 7.20 may not be applicable.



$K_a = \tan^2 (45 - \frac{\phi'}{2})$

$K = 1 - m \frac{4s_u}{\gamma H}$

m = 1.0 except where the excavation is under-lain by deep soft normally consolidated clay, then m = 0.4

(a) Sand (b) Soft to Medium Clay (c) Stiff-fissuerd Clay

where γ = Bulk unit weight of soil

Figure 7.20 Apparent Pressure Diagrams for Computing Strut Loads in Strutted Excavations (modified form Peck, 1969)

7.6.4 Circular Shaft

The stability of a circular shaft relies on the development of hoop stress or circumferential stiffness to resist external earth and groundwater pressures. Documented case histories of circular shafts in Hong Kong and overseas reported that the lateral displacements were small, which might restrict the development of full active or passive earth pressure. In the absence of a detailed assessment, the earth pressures should be assumed to be at the at-rest state (K_o) for limit equilibrium analysis of a circular shaft. When available, numerical analysis in 2D axi-symmetry or 3D is preferred for the design of circular shafts.

The bending moments and shear forces developed in the wall panels of a circular shaft are generally small because the excavated side is continuously supported by the hoop stress. However, it is necessary to carry out structural checking of the hoop stress against the compressive strength of the wall material.

8 Serviceability Limit State Design

8.1 General

In SLS design, the ground deformation induced by ELS works is usually assessed by either empirical correlations or numerical analyses. The assessment is aimed at demonstrating that the estimated ground deformation is within tolerable limits with respect to nearby sensitive receivers and that the excavation works could be safely executed.

Empirical correlations have been developed based on field observations from overseas projects, supplemented with local experience over the years, and cover a wide range of support systems and ground conditions. Some correlations give the ratio between the maximum ground settlement and wall deflection that are applicable to ground deformation caused by dewatering and bulk excavation. Numerical analyses are more commonly used nowadays, especially for deep excavations, and provide estimates of both vertical and horizontal ground deformation, taking into account the SSI and construction sequence of the ELS works.

Limit equilibrium analysis is seldom used in SLS design because it does not consider the soil and wall stiffnesses, nor their interaction. Therefore, the induced ground deformation cannot be estimated by using this method. This Chapter provides guidance on SLS design for various support systems in the checking of serviceability limits.

8.2 Sources of Ground Deformation

The following are the common sources of ground deformations caused by ELS works and should be considered in SLS design:

(a) Wall installation;

(b) Bulk excavation;

(c) Dewatering;

(d) Preloading of struts; and

(e) Removal of lateral support.

Other construction activities may also induce ground deformation, such as ground improvement works (e.g. excessive grouting works) and removal of temporary sheet piles or pipe pile wall, all of which should be considered in SLS design if their effects are judged to be potentially significant in causing ground deformation. However, prescribed values based on past experience, rather than an analytical approach, are usually adopted to allow for these additional ground deformation.

Some local experiences have shown that although the design assumptions of ELS works were justified at the design stage, the observed maximum ground deformation during excavation works still sometimes exceeded the design estimations. These problems were attributed to various construction issues, such as late installation of lateral support, over-excavation, over-breaking during wall installation, ingress of soil or groundwater due to excessive dewatering and inadequate penetration of wall embedment (e.g. Malone, 1982;

GEO, 1992; Lee, 2019; Endicott, 2020). Design and construction aspects of control measures to avoid the occurrence of these problems are discussed in Chapters 4 and 5 respectively.

8.3 Estimation of Ground Deformation

8.3.1 Wall Installation

Types of wall installation are broadly classified as either displacement or replacement methods. Both methods induce ground settlements and the magnitude of settlement depends on the ground conditions, construction plant adopted, construction duration and workmanship. The induced ground settlement is usually assumed to be the same as the lateral ground deformation caused by wall installation. The construction considerations for wall installation methods and quality assurance measures are discussed in Chapter 5.

8.3.1.1 Displacement Method

It is rather difficult to simulate the installation process of a ground displacement method (e.g. driving of sheet piles) with reasonable accuracy by numerical analysis, and there is a general lack of empirical correlations between pile impacts and ground deformations. Thus, a prescribed value is usually adopted based on monitoring records from local projects in similar ground and groundwater conditions. Induced ground deformation by displacement piling in competent ground conditions is usually minimal and localised. However, if the ground conditions comprise a large extent of thick loose fill with a high groundwater table, the induced ground deformation due to vibration associated with pile driving could be significant. In such cases, the effect should be considered using a prescriptive approach and verified by site trials prior to construction.

8.3.1.2 Replacement Method

Wall installation by a ground replacement method usually involves the removal of soil by boring, with the empty bore supported by either slurry or casing. Inevitably, the circumferential stress around the bored hole is reduced to some extent and causes lateral deformation of the soil around the empty bore, hence inducing ground deformation. Overburden drilling with casing is the method usually adopted for installing small diameter replacement piles. The installation process should allow the casing to be advanced together with the drill bit such that only a small section of unsupported bore is permitted ahead of the casing. In such cases, the induced ground deformation is generally minimal and localised. However, good workmanship and careful control of the air flushing pressure and advancement rate are essential for successful pile installation with minimal ground disturbance. Chapter 5 gives a detailed account of the site control and supervision that should be implemented. Site trials are necessary to observe and verify any ground disturbance caused by the boring works.

Large-diameter bored piles in urban setting are usually installed with temporary steel casing provided down to the competent soil stratum or rock layer. Therefore, the induced ground deformation is usually minimal. However, where slurry is used to support the excavation of bored piles or diaphragm wall panels, significant ground settlements associated with the wall installation were observed in some local cases. The magnitude of the induced ground deformation depends on the ground conditions, geometry of the excavation, the time elapsed between excavation and concreting, and the effective slurry pressure. Collapse of the arching effect in slurry-filled trenches excavated for diaphragm wall panels have also contributed to significant ground settlement in some cases.

There are many reports on the magnitude and extent of ground settlement associated with diaphragm walls formed by slurry trench excavation. Davies & Henkel (1980) reported that the measured ground settlements depend on the effective slurry pressure supporting the trench, which is the difference between the slurry pressure inside the trench and the external groundwater pressure. Clough & O'Rourke (1990) summarised the ground settlements observed from the construction of diaphragm wall panels in different ground conditions. The maximum ground settlement, δ_v, was about 0.15% of the excavation depth of the diaphragm wall, H_t, as illustrated in Figure 8.1. However, most of these measurements were taken behind the diaphragm wall and within the influence zone of the excavation. The average magnitude is generally less than 0.05%H_t. It should also be noted that the case studies in Clough & O'Rourke (1990) mainly included sites comprising clayey materials, except for the excavations in Hong Kong which were predominately in granular soils.

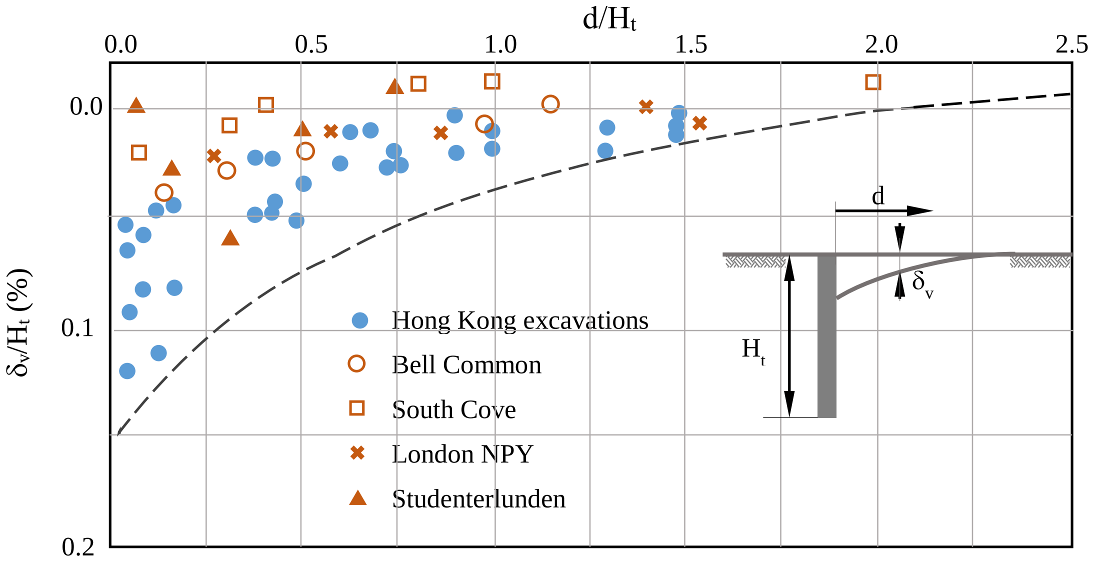


Figure 8.1 Measured Ground Settlement Caused by the Installation of Diaphragm Wall (modified from Clough & O'Rourke, 1990)

Pickles et al (2003) reported a local excavation project in reclaimed land where the maximum ground settlements in the order of 10 mm to 50 mm were observed during pre-trenching and installation of diaphragm wall panels with depths varying from 15 m to 45 m down to bedrock. In this case, the ratio of the induced maximum settlement was about 0.1%H_t. It was also observed that relatively larger settlements occurred when excavating in weak saprolite with corestones and in a buried old seawall structure. As discussed in Chapter 3, the installation of diaphragm wall panels should be closely monitored for quality control, and an adequate slurry head should be always maintained during the trench excavation.

The maximum recorded ground settlements due to diaphragm wall installation in eight local ELS projects in reclaimed land are presented in Figure 8.2 and key information about the projects is summarised in Appendix A. Most of the induced maximum settlements are less than 0.1%H_t, except for the projects at the Chater Station and the Tsuen Wan West Station. At Chater Station, which was constructed in 1979, Davies & Henkel (1980) reasoned that the larger ground deformation was caused by a rise in the groundwater table as a result of the construction of preceding wall panels, which reduced the effective slurry pressure supporting the subsequent trench excavations. Pickles et al (2003) reported that the large ground settlement observed during the diaphragm wall construction for the Tsuen Wan West Station was mainly associated with the cobbles and boulders encountered in the buried old seawall structures. In a case where ground improvement works were carried out prior to trenching at the roadworks at Kwun Tong, the induced ground settlement was only 0.02%H_t, which was about 10 mm (Figure 8.2).

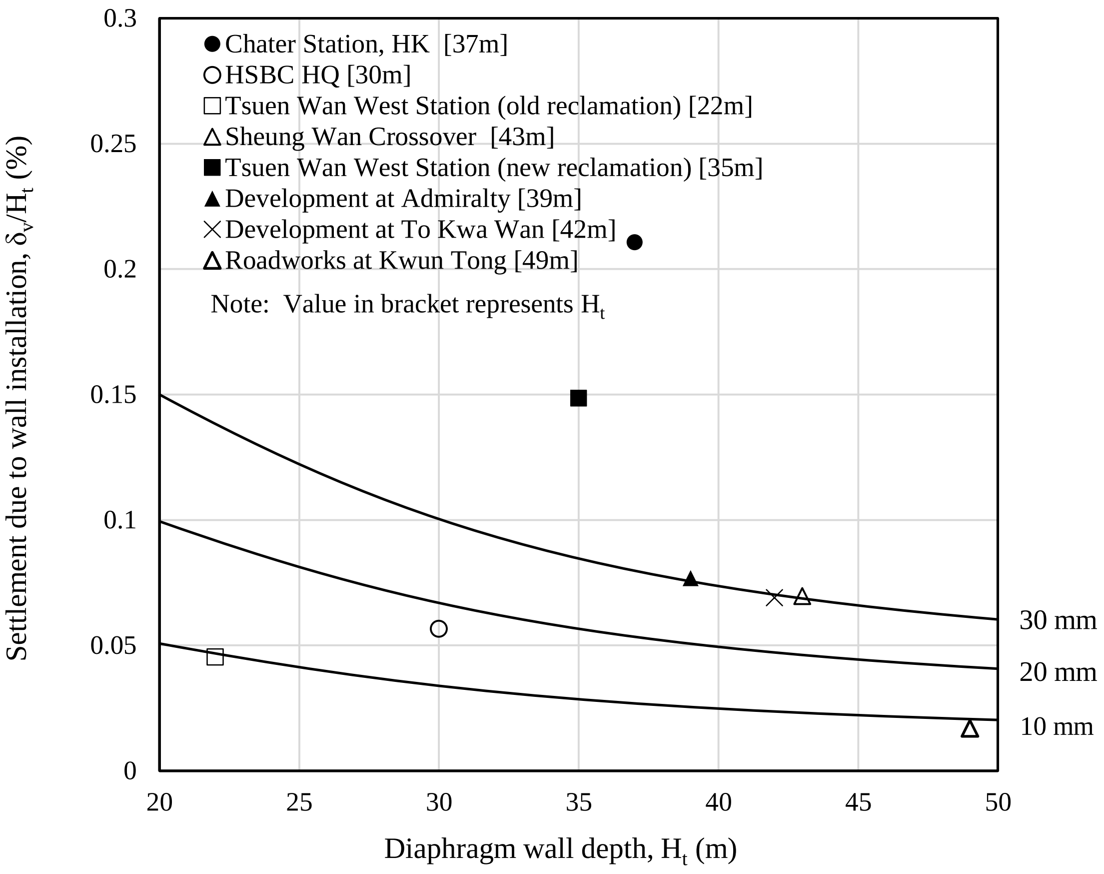


Figure 8.2 Measured Ground Settlement due to Diaphragm Wall Installation from Case Study Data

It is recommended that for estimating the effects of diaphragm wall panel construction, a minimum ground settlement of about 0.05%H_t should be allowed for when assessing the impact on nearby sensitive receivers. However, where the trench excavation involves removal of corestones in saprolite or buried man-made ground features, a higher prescribed value may be more appropriate. In competent ground conditions (e.g. dense saprolite), or if ground improvement works have been implemented prior to trenching, the induced ground settlement due to diaphragm wall installation might be less than the nominal value of 0.05%H_t. On the other hand, ground settlement can be also assessed using either an empirical approach (e.g. by referring to previous projects with similar ground conditions and wall panel geometry) or by a numerical method. Site-specific trials should be undertaken to confirm the design slurry pressure and the induced ground deformation.

Advanced numerical models have been used by some researchers to estimate the ground deformation caused by the installation of driven or bored piles. However, applications of such models in practice are still limited because of the simplifications and assumptions that have to be made and the absence of sufficient site verification for wider adoption. A pragmatic approach using prescribed values and field verification is usually preferred in SLS design.

8.3.2 Bulk Excavation and Dewatering

At the bulk excavation stage, lateral deformation of an embedded wall is caused by the release of horizontal stresses due to removal of soil and the resulting difference in soil and groundwater pressure between the excavated and unexcavated side of the ELS works. The magnitude of induced ground deformation is influenced by many factors, including the type of wall, geological and hydrogeological conditions and the strutting system.

Clough & O'Rourke (1990) discussed typical profiles of wall deflection and the adjacent ground deformation based on case histories (Figure 8.3). During the initial stage, dewatering and soil excavation are carried out before installation of the first lateral support. As such, the wall deflects as a cantilever element. The adjacent ground settles in a parabolic shape where ground settlement decreases in inverse proportion to the distance from the edge of the excavation (Figure 8.3(a)). When the excavation and dewatering advances to a greater depth, the upper wall deflection is restrained by the installed struts, and deep inward movement of the wall occurs (Figure 8.3(b)). The cumulative deflection of the wall and the ground is the combination of the cantilever and deep inward components as shown in Figure 8.3(c).

Where a numerical method is used in the SLS design, it should be noted that the model always requires simplification of the ground conditions and makes assumptions on the constitutive behaviour of the material on both sides of the wall. The Mohr-Coulomb constitutive soil model is commonly adopted in local practice. This model is simple to apply and gives reasonable estimates of wall deflection, as concluded from back-analysis case studies reported by Chan (2003). However, the simple model does not account for non-linearity of soil stress-strain behaviour and soil stiffness under the unloading condition, and usually predicts ground heave at the initial stage of soil excavation. More advanced soil constitutive models representing non-linear stress-strain behaviour and small strain stiffness can better simulate the ground response. Where the use of a more advanced soil model is anticipated, corresponding site-specific field and laboratory tests should be conducted to obtain the required parameters for setting up the model.

A tied-back wall offers an advantage in providing a larger working space within the excavation when compared to a strutted excavation. Modelling of a tied-back bored pile wall using a 2D FEM in a local project was reported by Lam (2018). In this case, it was considered that the Mohr-Coulomb constitutive soil model might have overestimated the ground deformation under the small-strain condition for a tied-back wall, and instead a hardening soil small strain model was used and gave a better match with the actual field performance.

The layout of the excavation can affect the magnitude and distribution of ground deformation around it. The corners of ELS works with diagonal struts often restrict wall deflection. Ou et al (1996) reported that the measured maximum wall deflections at the corners of ELS works could be less than 50% when compared with the results of 2D plane strain from 3D analysis. More sophisticated 3D numerical analysis may be adopted to examine such corner effects, in case there are nearby highly sensitive buildings/structures/services that require careful and more accurate assessment of ground deformation. Pappin et al (2005) presented a case history of a Tsim Sha Tsui Station concourse extension excavation using a 3D finite difference method. The predicted ground deformation based on the 3D model was found to be more representative when compared with the results measured during construction.

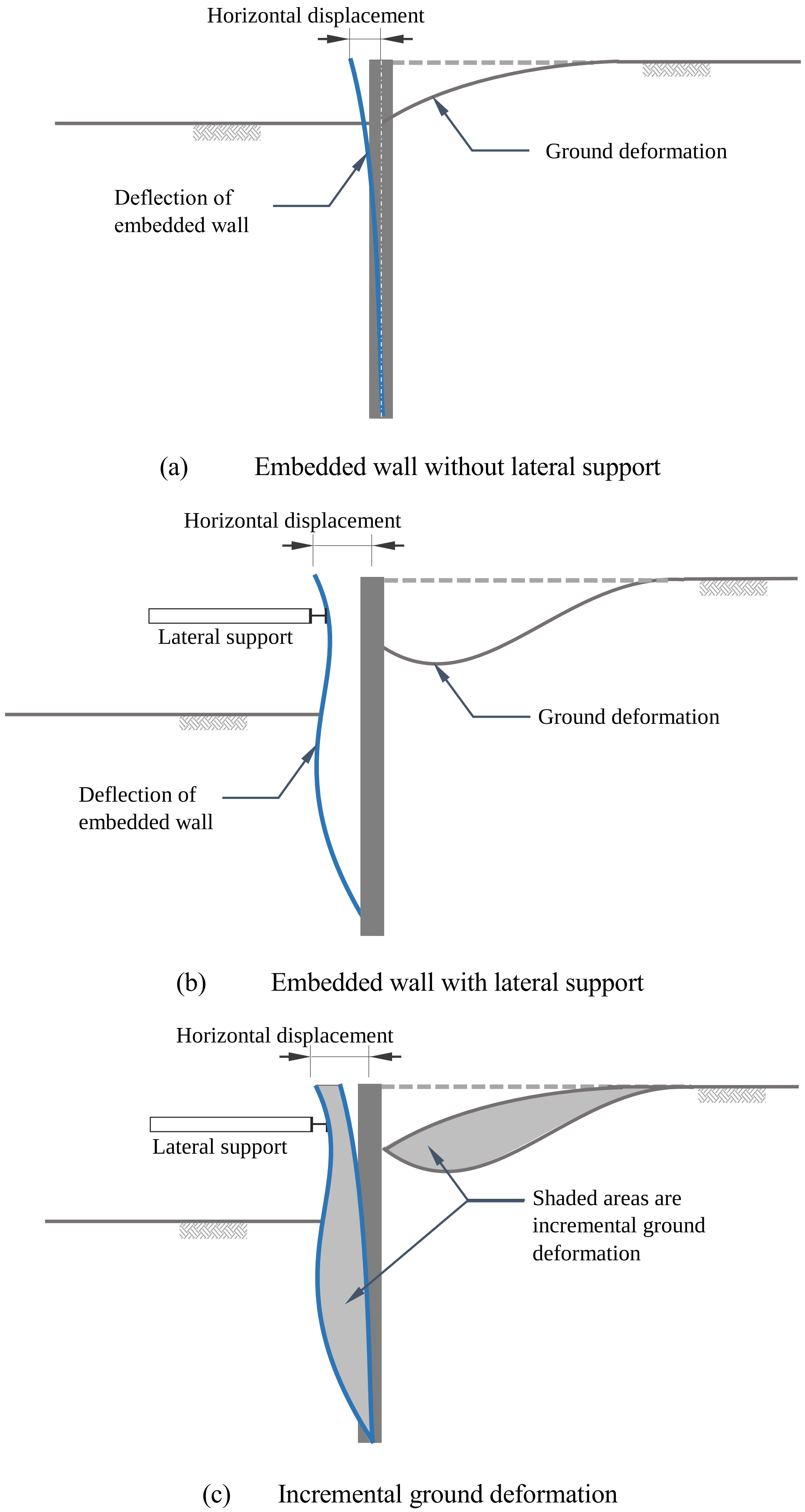


Figure 8.3 Typical Profiles of Wall Deflection and Adjacent Ground Deformation
(modified from Clough & O'Rourke, 1990)

The boundary element method is also popular for modelling staged excavations. In this method the wall is assumed to consist of a series of discrete beam elements attached to the soil at common nodes and an iterative procedure is adopted to calculate the soil reaction. Incremental ground deformation at each excavation stage is computed and summed up to give the wall deflection profile. However, this method only calculates the lateral deflection of the embedded wall and estimation of ground settlement is usually based on empirical correlations. Lui & Yau (1995) reported the back analysis of the basement excavation at the Dragon Centre, Kowloon. The maximum excavated depth was about 26 m and a diaphragm wall was used for support. The maximum ground settlement caused by the bulk excavation was equal to about 50% of the maximum wall lateral deflection. In contrast, Li et al (2004) and Pickles et al (2006) reported a larger ratio in the excavation for the Tsuen Wan West Station, where a diaphragm wall was also used to support the deep excavation in reclaimed land. The observed maximum ground settlement in this case was about 0.75 to 1.0 times the maximum wall lateral deflection and the authors considered that it was attributed to two-stage full-scale pumping tests being carried out concurrently with the bulk excavation works.

Figure 8.4 presents observed ground settlement against lateral wall deflection from twenty-three deep excavation projects which provided good-quality monitoring data. This figure only shows the ground settlement associated with dewatering and bulk excavation. The ground settlement caused by wall installation is not included. These ELS works were constructed in various ground conditions with maximum excavation depths (H_e) varying from 14 m to 45 m. In each case the observed maximum ground settlement and lateral wall deflection is based on a pair of ground monitoring stations and available nearby inclinometers. Figure 8.4 also shows ranges of the ratio between maximum ground settlement (δ_v) and maximum lateral wall deflection (δ_h). For projects with small ground deformations (i.e. δ_h / H_e and $\delta_v / H_e < 0.1\%$), there is no apparent correlation between the maximum ground settlement and lateral wall deflection. For other cases (i.e. δ_h / H_e and $\delta_v / H_e > 0.1\%$), the maximum ground settlement is usually about 50% of the maximum lateral wall deflection, except for the Tsuen Wan West Station case that is mainly associated with the cobbles and boulders encountered in the buried old seawall structures. For the circular shaft excavation cases, inward wall deflection is usually insignificant.

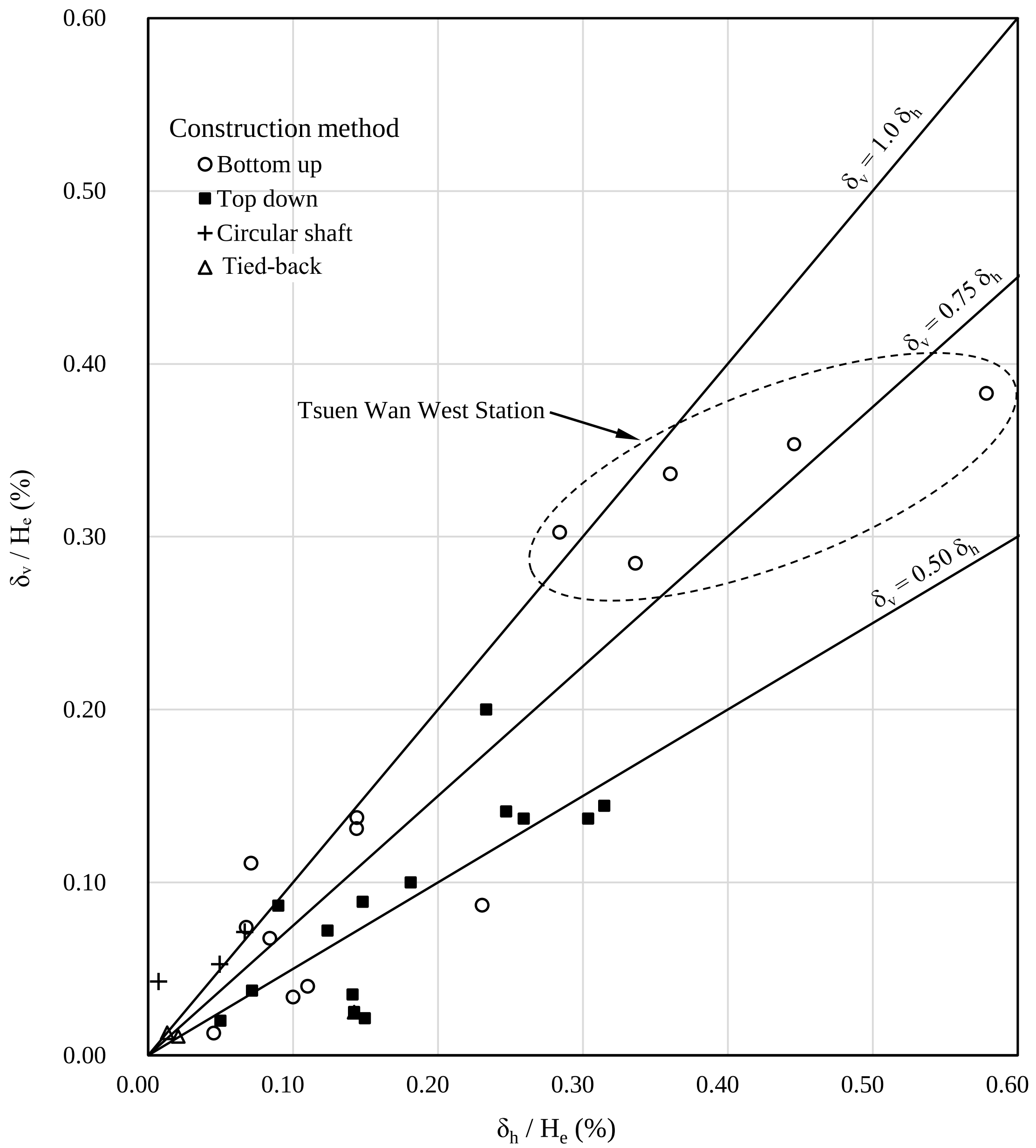
8.3.3 Ground Settlement due to Groundwater Drawdown

8.3.3.1 Elastic Settlement

When ELS works do not include an impermeable barrier installed to a stratum with very low permeability or bedrock, groundwater drawdown outside the excavation may occur. This drawdown will lead to an increase in the effective stress in the soil matrix and consequential settlement. However, it is recognised that the groundwater level varies naturally, and only settlement caused by groundwater drawdown below the lowest level that the soil has experienced needs to be considered in the SLS design. In calculating settlement due to an increase in vertical effective stress, the soil stiffness in 1D compression (i.e. the constrained modulus, E_o') can be used. The relationship between Young's modulus in terms of effective stress, E_s' and E_o' is discussed in Section 5.6 of Geoguide 1.

8.3.3.2 Consolidation

Consolidation settlement is also caused by an increase in effective stress due to change in porewater pressure within the soil matrix. This type of settlement is especially important in fine-grained soils (e.g. marine clay) of low permeability and dissipation of the porewater pressure takes a much longer time than in more granular soils. The delayed effects of such ground settlement should be properly addressed, particularly for excavation projects with a long construction period. It is preferable to complete the excavation and terminate the groundwater drawdown as soon as possible. As discussed in Section 2.1.2, the differential piezometric pressures between marine clay and alluvial sand layers could lead to porewater pressure dissipation and associated consolidation settlement in the clay layer even when the groundwater level in an overlying fill layer remains stable.



where δ_h = Maximum lateral wall deflection
H_e = Maximum excavation depth

Figure 8.4 Relationship between Ground Settlement and Lateral Wall Deflection from Case Study Data

Consolidation settlements are seldom estimated using empirical methods. Instead, they are commonly assessed by carrying out one-dimensional consolidation analysis assuming zero lateral strain. The coefficient of volume compressibility or compression index of the fine-grained soil is required for such estimation. Guidelines on one-dimensional consolidation analysis are given in Geoguide 1.

The effects of consolidation on nearby sensitive receivers should be properly assessed in SLS design of ELS works, especially in reclaimed land. It may cause significant long-term settlement of ground and structures if there are continuous dewatering works, even during the construction of basements and superstructures.

8.3.4 Preloading of Struts

Preloading of struts serves to reduce excessive wall deflections and hence ground deformation. However, it should be properly designed so as not to damage any underground utilities and structures, especially near the ground surface with a relatively small soil overburden. Therefore, the maximum allowable preloading at each layer of strutting should be duly considered with respect to the estimated movement of struts and the embedded wall in relation to the tolerable limits of sensitive receivers.

8.3.5 Removal of Lateral Support

The sequence of removal of temporary support should be considered in SLS design, especially for ELS works with preloaded struts. Rebound of the embedded wall due to removal of the struts can induce significant wall deflection and hence ground settlement.

8.4 Case Histories of Observed Ground Settlement

Long (2001) studied some 300 cases of deep excavation projects worldwide and concluded that the observed maximum ground settlements were usually less than 0.2%H_e. Zhang & Liu (2021) extended the study by Long (2001), and included more excavation projects in ground conditions comprising strata with clayey soils. Zhang & Liu (2021) found the largest ground settlement to be as high as 10.25%H_e, with an average value of about 0.55%H_e.

If the sites comprising mainly clayey soils were excluded, Zhang & Liu (2021) reported better performance of the ELS works, with the largest ground settlement being 0.31%H_e and the average settlement about 0.14%H_e.

Leung & Ng (2007) reported that the maximum ground settlement observed in fourteen deep excavation projects in Hong Kong varied significantly from 0.01%H_e to 0.22%H_e. The CIRIA C760 indicated that maximum ground settlement due to excavation in loose to medium dense sand was about 0.3%H_e immediately adjacent to the wall, decreasing to zero at a lateral distance of about 2H_e, as shown in Figure 8.5.

Figure 8.6 shows the compiled data relating δ_v to H_e for twenty-seven excavation projects in Hong Kong. The total settlement comprises all settlement experienced since commencement of the ELS works, including wall installation, and key information about the projects is summarised in Appendix A. The range of δ_v varies from about 0.05%H_e to 0.45%H_e but most of the measurements are less than 0.25%H_e. Davies & Henkel (1980) and

Pickles et al (2003; 2006) discussed the reasons for the larger ground settlement observed at the Chater Station and Tsuen Wan West Station sites. The observed large ratio of 0.45%H_e for the ELS works at Tsuen Wan West Station is mainly due to the maximum settlement of 52 mm induced by pre-trenching carried out in the former seawall layer (cobbles and boulders) at this new reclamation area (Pickles et al, 2003; 2006). The large total settlement of about 0.67%H_e at Chater Station occurred along the side of the adjacent Courts of Justice, where the diaphragm wall foundations were not taken down to bedrock and therefore large settlement of up to 80 mm was observed during dewatering, even though recharge wells had been adopted at the site (Davies & Henkel, 1980).

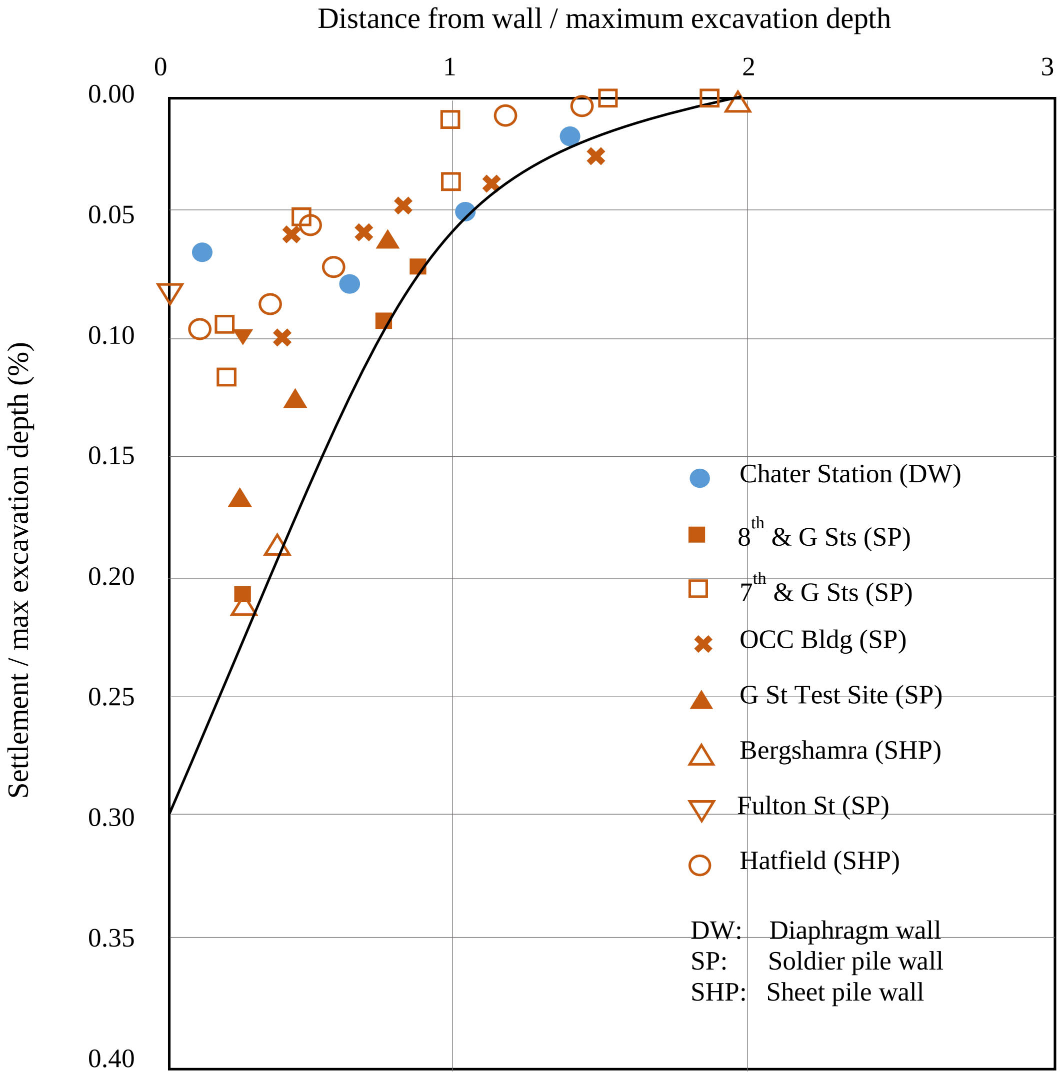


Figure 8.5 Ground Settlement due to Excavation in Coarse Grained Soils (modified from Gaba et al, 2017)

The large data scatter in Figure 8.6 is due to different ground conditions (e.g. reclaimed land with varying ground conditions), wall types (e.g. circular shaft, diaphragm wall and pipe pile wall) and construction methods (e.g. top down or bottom up construction sequence). It can be seen that relatively small ground settlements of less than 0.1%H_e were achieved at the sites employing circular shafts and tied-back walls.

Although the data gathered suggests there is no strong indication that the total observed ground settlement increases with H_e, 0.3%H_e to 0.5%H_e appears to be a reasonable range of settlement to be expected using common and appropriate systems of ELS works, unless particularly sensitive receivers are present and the works have included enhanced measures (e.g. large preloading of struts, pre-grouting, installation of additional sheet piles) to limit the ground settlement. Such a range can be adopted as the targeted ground settlement when devising a suitable ELS system. If the estimated settlement is higher than this range, design of the ELS works should be critically reviewed. Precautionary measures (e.g. pre-excavation grouting) may be necessary so as to reduce the estimated ground settlement below the maximum range, instead of deferring the problem to the construction stage. On the contrary, over-conservative design may result in the adoption of an unnecessary strutting system, or a stiffer and deeper embedded wall. The ensuing construction risk associated with an over-conservative design may not render the ELS system any safer.

Estimation of the maximum lateral ground deformation can be assumed to be equal to the maximum lateral wall deflection at the wall top if there is a particular concern over the impact of lateral ground deformation on nearby sensitive receivers.

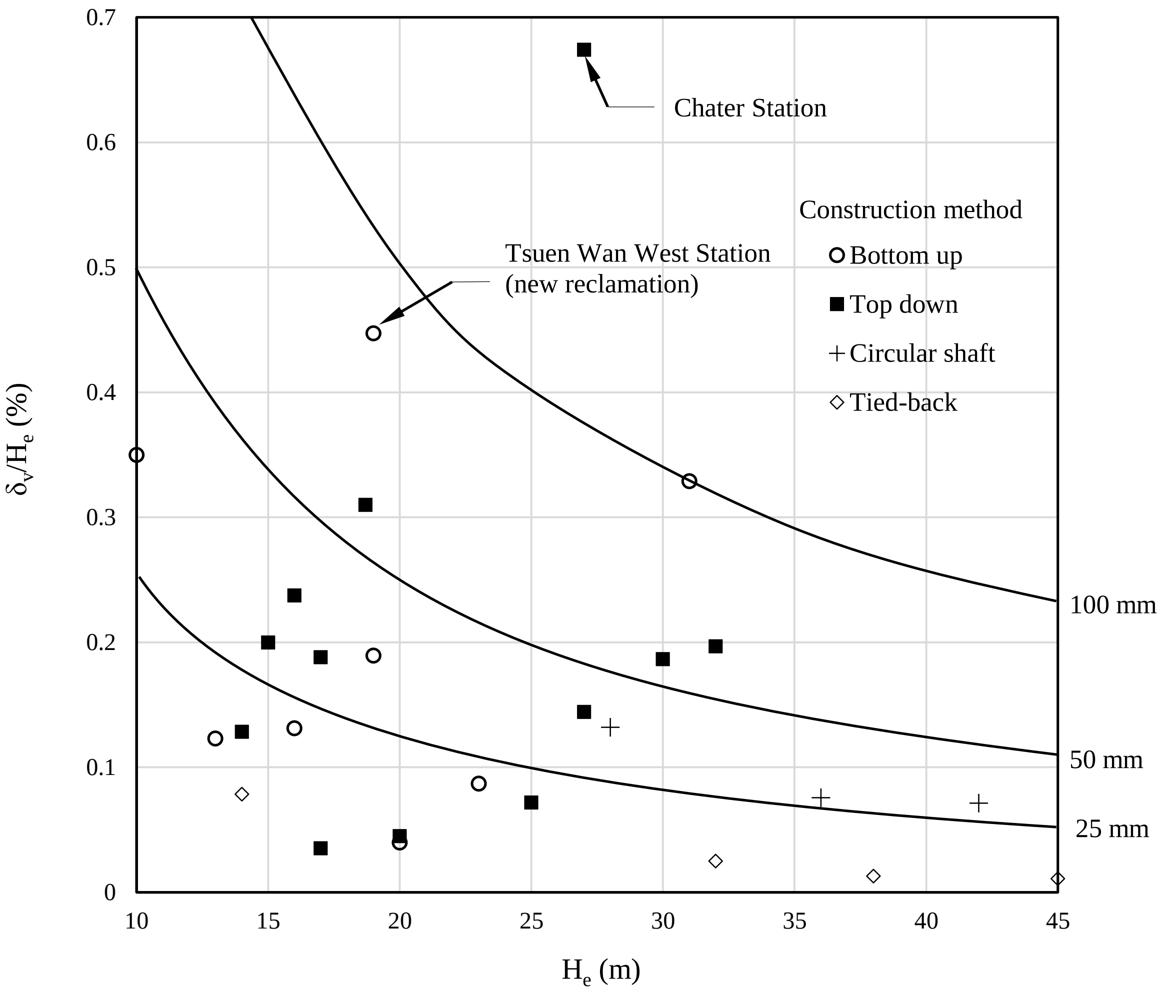


Figure 8.6 Total Ground Settlement against Maximum Excavation Depth from Case Study Data for Projects in Hong Kong

8.5 Tolerable Serviceability Limits of Sensitive Receivers

Estimating ground deformation in the SLS design is an important step in assessing the effect of ELS works on sensitive receivers, which may have different tolerable limits depending on their type and nature. A suitable control mechanism with corresponding response actions should be derived as part of the design, in order to safeguard against any unforeseen and excessive movement of nearby sensitive receivers during construction. Guidance on devising a ground deformation control mechanism and determination of the trigger values of sensitive receivers for response actions is given in Chapter 9.

9 Control of Ground Deformation

9.1 Control Mechanism

ELS works need to be carried out cautiously and implemented together with a prudent mechanism for controlling ground deformation. This is to ensure that the impact of the ELS works on nearby sensitive receivers is kept within an acceptable level. Geotechnical design is often carried out with simplifications and assumptions. The performance of ELS works should be regularly checked and based on monitoring data obtained during construction. An adequate I&M plan should be included in design. Design reviews should be also undertaken by the project team at suitable stages of the works, and where necessary, precautionary measures implemented to minimise any adverse impacts on nearby sensitive receivers. It should not rest solely on the control mechanism to trigger design reviews.

In the course of SLS design, the ground deformation induced by ELS works is assessed either by empirical correlation or numerical analysis. The assessment is used to demonstrate that the estimated maximum ground settlement is within the tolerable limits of the nearby sensitive receivers. Guidance on the estimation of ground settlement is given in Chapter 8. In addition, an I&M scheme and a control mechanism are usually implemented during construction to demonstrate satisfactory performance of the ELS works and to confirm that the induced ground settlement is within the design estimates.

The control mechanism serves as a complementary forewarning to safeguard nearby sensitive receivers and the public. It should trigger appropriate response actions to prevent the situation from deteriorating to the point where it causes adverse impact on sensitive receivers, or poses a hazard to the public. A three-tier triggering control mechanism, i.e. Alert-Alarm-Action (AAA) Levels (BD, 2018), each with corresponding response actions, is commonly adopted for excavation projects. The AAA Levels are specified with due consideration given to the existing conditions and serviceability of nearby sensitive receivers. In general, the AAA Levels are set at 50% and 75% of the Action Level respectively. For sensitive sites or sites with a ground settlement limit (Action Level) estimated based on an engineering approach, the Alert, Alarm and Action levels should be set by referring to Appendix C of PNAP APP-24 for the ground settlement limits, or as specified by the relevant stakeholders (e.g. Antiquities and Monuments Office (AMO), MTR Corporation Limited), to provide more stringent control on the ground settlement, whichever is applicable. For declared monuments and graded historic structures, AMO may have different sets of AAA Levels requirement.

The AAA trigger levels and their response actions under the control mechanism should be determined sensibly and practically. The trigger values of each level should be specified in the I&M plan at the design stage. Relevant stakeholders (e.g. owners, maintenance parties, utility undertakers) should be consulted on the adequacy of monitoring stations for assessing possible impacts on nearby sensitive receivers, the proposed AAA trigger values and the corresponding response actions. The influence zone of the ELS works should be properly assessed in the engineering analysis. Otherwise, a minimum horizontal distance of 1.5 times H_e should be adopted. If the ground in the area surrounding the site includes a thick layer of marine deposits, a larger influence zone with a horizontal distance greater than 1.5 times the H_e should be duly considered. Potential impacts to all affected sensitive receivers within the influence zone should be assessed. It is important to install suitable instrumentation where necessary and to conduct monitoring at timely occasions and intervals.

9.2 Determination of Trigger Values

9.2.1 Empirical Approach

The magnitude of ground settlement that would trigger response actions could be determined using either an empirical or an engineering approach, as stipulated in PNAP APP-24 (BD, 2022). The empirical approach is generally suitable for sites affecting nearby buildings, structures and services that are not particularly sensitive to settlement, in which case the AAA trigger values are usually specified by referring to the empirical limits given in PNAP APP-137 (BD, 2018). In addition to setting trigger values for sensitive receivers, trigger values for groundwater drawdown are also sometimes included in the control mechanism, as a pretext to guard against any adverse effects caused by variation of groundwater levels beyond the assumed ranges.

The prevailing control mechanism of adopting an empirical ground settlement limit of 25 mm as the Action Level has been successfully applied for many simple excavation projects in Hong Kong. However, as excavations become deeper and more complicated, the adoption of empirical values has resulted in some distorted solutions in the ELS design. In many cases, extensive and heavy strutting systems (e.g. high preloading forces, closely-spaced strut layers) are proposed, in order to limit ground settlement within the empirical limit of 25 mm. Preloading is an attempt to push back the embedded wall. The connections between struts, walings and the embedded wall are seldom designed to allow for the reverse movement caused by preloading a strut at a lower level. The integrity of the grout curtain installed behind the embedded wall may also be adversely affected by excessive preloading. Moreover, highly congested struts and cross postings will obstruct the excavation and subsequent basement construction, which may lead to the malpractice of not strictly following the approved design and construction sequence of the ELS works. Hence, it is important to consider the safety and buildability of ELS works in their entirety when determining practical solutions.

In other situations, despite the estimated ground settlement being greater than 25 mm, the Action Level is still set at the empirical limit of 25 mm under the guise of having an early warning (THB, 2020). However, in such cases, the Action Level is almost bound to be triggered, which would then lead to a call for suspension of all site works. Given that the original estimate has already exceeded the 25 mm limit, the AAA trigger values then have to be relaxed subsequently to cater for the actual performance, and sometimes even without any genuine need for additional precautionary or remedial works.

In general, the current prevailing practice for the assessment of ground settlement is rational and reasonably representative of the actual situation. Common and appropriate ELS systems usually result in ground settlements within a typical range of between $0.3\%H_e$ and $0.5\%H_e$, and the impacts on nearby sensitive receivers induced by such settlement should be duly assessed.

To take a specific example, a maximum ground settlement of about 100 mm was recorded at the Exhibition Station during construction of the Shatin-to-Central Link Project, which involved deep excavation of about 30 m (THB, 2020). Hence, adoption of the empirical limit of 25 mm as the trigger value of the Action Level (i.e. works suspension) was obviously questionable in this case, as it had a high chance of exceedance with the known design maximum estimate of about four times larger than 25 mm. Where the estimated ground settlement significantly deviates from the typical range (i.e. between $0.3\%H_e$ and $0.5\%H_e$), the excavation scheme should be critically examined. Overly-conservative design may result in the adoption of unnecessary and unreasonable strutting systems, or an unusually stiff and deep embedded wall, and the ensuing construction risk may not render the ELS works any safer. On the contrary, ground improvement measures (e.g. pre-excavation grouting to improve soil strength, or jet grouting in soft strata) may be necessary to bring the estimated ground settlements within the typical range, or additional strengthening or supporting measures may be required after consultation with stakeholders. Hence, there is a genuine need for rational determination of AAA trigger values and corresponding response actions.

9.2.2 Enhanced Empirical Approach

9.2.2.1 General Considerations

Adoption of an empirical ground settlement limit of 25 mm may not be reasonable and practical for all deep excavations. The typical response actions set at the Alert Level and Alarm Level are too generic and execution of the actions is entirely reliant on the project team and the contractor. The promptness, adequacy and appropriateness of the actions may vary significantly between projects and their importance may sometimes be played down. This may result in a lack of control and may eventually defeat the purpose of forewarning if response actions are not properly implemented. Also, stakeholders are often not notified promptly or consulted on the need for remedial works to affected sensitive receivers until the Action Level is reached (i.e. works suspension).

On the other hand, response actions at the Action Level are sometimes set too broadly by requesting suspension of all works, which might include those works that are essential for maintaining stability (e.g. strut installation, strut preloading). Besides, it usually takes time to obtain consent from the relevant stakeholders and authorities on the appropriate remedial works after the Action Level is triggered and works suspended. Such arrangements may result in prolonged delays to the works programme. Moreover, there is often no clear provision in the response actions regarding revision of the AAA trigger values after exceedance, leading to uncertainty in the control mechanism after works resumption in some circumstances (THB, 2020).

9.2.2.2 Response Actions for Serviceability and Stability

In fact, certain response actions (e.g. repaving cracked road pavements, repairing damaged subgrade, reinstating deformed pipelines) can be readily arranged for effective mitigation measures, with prior agreements obtained from the relevant stakeholders and authorities. Given that exceedances of AAA trigger values are mostly related to ground settlement affecting roads and services, a set of more specific response actions (e.g. GI such as GCO probe and GPR survey) at different levels of ground settlement to cater for the serviceability issues of these sensitive receivers, as well as stability considerations for the ELS works, can further streamline the procedures and enhance the effectiveness and efficiency of the control mechanism.

An alternative control mechanism has been devised to enhance the response actions so that it can address the serviceability of the sensitive receivers and take precautionary measures where necessary, as well as ensure overall safety of the ELS system by checking its performance against measured settlements if the performance deviates from the accepted design. This enhanced control mechanism expands the 3-tier system into a 5-tier system by sub-dividing the third tier response actions into 3 levels (i.e. Alert-Alarm-Action Levels 1-3).

Under the enhanced control mechanism, called the 5A Approach, each Action Level has a set of well-defined responses to address the serviceability of different sensitive receivers and stability of the ELS works. When the respective trigger values of the Action Levels are reached, it is important that the relevant stakeholders and maintenance parties of the sensitive receivers are promptly notified and consulted on the need and timeframe for carrying out any necessary precautionary and remedial works, as well as any subsequent actions required (e.g. revised trigger values of the Action Level after repair works, regular liaison meetings). The trigger values should be determined reasonably and practically with respect to serviceability limit of the sensitive receivers and overall stability of the ELS system. On the other hand, a set of clear-cut response actions should be specified in the I&M plan and should be executed when the trigger values of the Action Level are reached. The I&M plan should state clearly that if the contractor fails to carry out the agreed response actions to the satisfaction of the affected stakeholders, maintenance parties and authorities, the relevant authority and the Engineer or Project Manager can, when necessary, instruct works suspension at any level until the agreed response actions are completed.

The total and differential settlements occurring in the ground do not necessarily equate to the settlement of services and buildings, which may be supported by deep foundations or rest on a different subgrade. Therefore, it is sensible to assess and control the effect of ELS works separately on the ground, services and buildings. Thus, the empirical limits for ground settlement and settlement of services and buildings should be considered and specified separately.

9.2.2.3 Empirical Limits for Ground Settlement

Ground settlement induced by ELS works that caused serviceability issues (e.g. cracks or uneven surfaces) in nearby pavements (e.g. footpaths or carriageways) and surface areas (e.g. playgrounds or carparks) can be readily repaired. The Guidance Notes for Road Inspection Manual (Report No. RD/GN/016C) issued by the Highways Department (HyD, 2016) recommends that depressions larger than 20 mm may pose a hazard to pedestrians and repair works should be carried out if necessary. On the other hand, there may be concern over the integrity of the paving material when there is significant ground settlement. Weng & Wang (2011) reported that differential ground settlement greater than 60 mm might cause considerable tensile strain in the pavement structure. In addition, the Specifications for Design of Highway Subgrades (JTG D30-2015) published as the National Standard of the People's Republic of China (MOT, 2015) recommends allowing differential settlement of up to 100 mm between bridges and road abutments and total settlement of up to 300 mm for general road pavements.

Under the 5A Approach, cumulative settlement should be used for triggering the response actions. Action Level 1 mainly deals with serviceability concerns relating to road pavements. It is recommended to set an empirical value of 20 mm for this Action Level for ground settlement affecting road pavements, including on-grade carriageways and footpaths. Exceedance of this level should trigger consultation with the relevant stakeholders, with repair or repaving works carried out to restore serviceability where considered necessary. The project team should also formulate an Action Plan to lay down the actions to be taken before Action Level 2 is triggered, including additional serviceability checks and remedial works requirements (e.g. when another 20 mm settlement occurs after the repair works). The Action Plan should also include the action to be taken when Action Level 2 is triggered such that preparation work can be done in advance.

When ground settlement continues, Action Level 2 is then used to trigger a review of the performance of the ELS works and assess whether further significant ground settlement may occur. The trigger value of Action Level 2 should be tied to the lower bound value of the typical range of ground settlement, i.e. 0.3%H_e. It should also be subject to a maximum cumulative value of 60 mm, as this magnitude of settlement may raise concern over the integrity of pavement structures. When Action Level 2 is reached, further investigation (e.g. GPR surveys, CCTV or open pit/trench inspection of underground services, and probing tests) should be carried out to investigate any underlying problems with the subgrade and services. The design should be reviewed to assess whether the cumulative settlement at subsequent stages of the excavation would likely exceed the overall estimated value, and whether the performance of the ELS is still acceptable. The project team should also formulate an Emergency Plan on reaching Action Level 3, including intermediate stage serviceability checks and a full-scale investigation plan and Works Suspension Plan where necessary.

Action Level 3 is intended to flag up the situation where the ELS works are under-performing when compared with other ELS works in similar ground conditions and site settings. The trigger value of Action Level 3 should be set to 0.5%H_e, and subject to a maximum empirical value of 100 mm. This upper limit takes into consideration the settlement limit of road pavements and subsequent reinstatement works needed to re-level the pavement. When this Action Level is reached, it is prudent to suspend the works as this will obligate the project team to critically re-examine the performance of the ELS works showing out-of-range movement before the works are resumed. Ground settlement monitoring should be continued in areas where repair or repaving works are being carried out such that cumulative ground settlement can be monitored. Where necessary, additional precautionary measures (e.g. increasing the number of struts, additional grouting), should be carried out to minimise any further ground settlement.

The trigger values of Action Levels 2 and 3 are correlated with the typical range of ground settlement with respect to the depth of excavation. This will ensure that appropriate ELS systems are adopted. For example, the trigger values of Action Levels 1 to 3 for a 10 m deep excavation will be set to empirical values of 20 mm, 30 mm and 50 mm respectively (i.e. limited to the range of 0.3%H_e and 0.5%H_e). Similarly, the trigger values of Action Levels 1 to 3 for a 20 m excavation or deeper will be set to empirical values of 20 mm, 60 mm and 100 mm (i.e. limited by the maximum values).

Given a trigger value of 20 mm under Action Level 1 for all excavation depths, it is also recommended to set the minimum trigger value for both Action Levels 2 and 3 to make the control mechanism more meaningful and practical. Taking into consideration the range of allowable settlements for a shallow excavation (about 5 m to 6 m deep typically for a one level basement and foundation works), it is considered reasonable to set minimum values for Action Levels 2 and 3 at 25 mm and 30 mm respectively. On the other hand, the prevailing Alert and Alarm Levels could remain so as to serve as an initial caution. However, the trigger value of the Alert Level should be set to 10 mm in order to enhance early detection of any serviceability problems. In addition, the trigger value of the Alarm Level should be set to 15 mm so as to differentiate it from the Action Level 1 value (i.e. 20 mm). Given this more proactive approach, it may be feasible at the design stage to estimate the number of occasions when repaving of roads or other remedial works might be necessary. Such a predicted frequency should be made known to the relevant stakeholders prior to the commencement of construction.

In cases where the ELS works are likely to affect the ground in private lots, the same principle of prior setting of trigger values for ground settlement could be adopted, provided that agreement is obtained from the relevant private lot owners. In this regard, it is desirable to commence discussions with adjoining private owners at an early stage of the design of the ELS works, so as not to impede commencement of construction. In case it is found difficult to obtain agreement from the relevant private lot owners, a tighter trigger value for ground settlement of affected nearby pavements and surface areas, as given in PNAP APP-137 (BD, 2018), may be adopted.

Regardless of whether any of the 5A Levels are exceeded, if any obvious damage to road or pavements is observed, the relevant stakeholders should be immediately notified and consulted to identify any potential hazard and assess the need for urgent repair and repaving works. In such cases, no further construction activities that could aggravate the ground settlement, including further lowering of the excavated level, should be allowed. Any remedial works should be completed to the satisfaction of the relevant stakeholders before the resumption of construction. In addition, GI (e.g. GCO probe, and GPR survey) and design review should be carried out to investigate the causes, inspect the pavement integrity and identify the possible presence of sinkholes at depth.

The aforementioned empirical limits do not include upward ground deformation caused by heaving. Generally, ground heaving outside the excavation is uncommon in ELS works. However, there are instances where ground heaving may occur due to factors such as excessive preloading on struts, lack of control in grouting works, and elevated groundwater pressure resulting from damming effects. It is essential to conduct a detailed investigation to identify the underlying causes of unexpected ground heaving, estimate potential further ground deformation, evaluate the potential impact on nearby sensitive buildings/structures/utilities, implement necessary remedial measures and amend the design as appropriate.

9.2.2.4 Empirical Limits for Underground Services

The serviceability of underground services such as water mains, cooling mains, gas mains, sewage pipes and cable ducts will be adversely affected if excessive differential settlement or angular distortion occurs. Total and even settlements of underground services are generally less critical than differential settlements, except for some special components such as relatively inflexible iron pipe joints and cable joints.

The “Conditions of working in the vicinity of water works installation” issued by the WSD (2020) specifies that differential settlement affecting water mains made of different materials should be controlled within a range of 1:400 (e.g. for asbestos cement (AC) and PVC pipes) to 1:200 (e.g. ductile iron (DI), galvanised iron (GLI) and mild steel (MS) pipes). Given the generic nature of these materials, it is recommended to adopt differential settlement (in terms of angular distortion) as the key criterion for monitoring services, using 1:300 as the trigger value of Action Level 3. The trigger values for Action Levels 1 and 2 are pragmatically set at 1:400 and 1:350 so as to align with the prevailing Alert and Alarm Levels, while the trigger values for the Alert Level and Alarm Level are revised to 1:600 and 1:500 respectively, in order to better differentiate it from the Action Level 1 value (i.e. 1:400).

On the other hand, designers should also take into consideration the tolerable limits of total settlement of services, in particular vulnerable components that could pose a hazard to the public in case of damage (e.g. joints of DI, GLI and MS water mains, iron gas mains and high-voltage power cable ducts). Where these services are laid in the vicinity of the excavation, the relevant stakeholders should be consulted regarding tolerable limits of total settlement, and any precautionary measures and specific response actions required.

Similar to road pavements, repair works to services should be arranged promptly by the project team in consultation with the relevant stakeholders when the Action Level 1 is triggered. By doing so, the serviceability of the services can be readily assured. In respect of vulnerable components of services that could pose a hazard to the public, the project team should also be required to take response actions under Action Level 1 whenever the total settlement has reached 20 mm, so as to determine and agree the way forward with the relevant stakeholders (e.g. regarding the next trigger value, requirements and timing of remedial works). Unlike ground settlement monitoring, the records of services monitoring markers should be reset if the affected services have been reinstated, such that the serviceability condition can be properly assessed and monitored again.

9.2.2.5 Empirical Limits for Buildings

Differential settlement affecting buildings and structures (e.g. fence walls, retaining walls, highway structures and bridge abutments) should be controlled within acceptable limits. The empirical values of building tilting recommended in PNAP APP-137 are generally considered reasonable and practicable. It is therefore recommended to set the trigger values of Action Levels 1 to 3 at 1:600, 1:550 and 1:500 on tilting respectively, to align with the prevailing Alert Level (1:1,000) and Alarm Level (1:750).

Similar to services, the tolerable limits for total settlement of buildings and structures should be considered, in particular those with special concerns (e.g. historical buildings, dilapidated structures, tunnels, railway structures, district cooling mains, footings sensitive to total settlement, structures with movement joints where different types of foundation are used to support them), and the relevant stakeholders should be consulted at an early stage of the excavation project.

9.2.2.6 Recommended Empirical Limits for the Enhanced Empirical Approach

The recommended empirical trigger values for the 5A approach are summarised in Table 9.1. These are not intended for application to specific or vulnerable sensitive receivers (e.g. historical buildings, dilapidated structures and services, hospitals, tunnels and railway structures, service reservoirs), for which more stringent requirements on tolerable limits of settlement and distortion would normally be imposed. An engineering approach can be adopted to assess tolerable limits of the sensitive receivers with respect to their stability and serviceability, and to establish site-specific trigger values, together with consultation with relevant stakeholders on a case-by case basis. In general, however, all site works undertaken should not impair the stability of, or cause damage to, either the structural or non-structural elements of services, buildings and structures.

Table 9.1 Recommended Empirical Limits for Setting Trigger Values (5A Approach)

Instrument	Criterion	Alert	Alarm	Action(4)		
				Level 1	Level 2	Level 3
Ground monitoring marker(1)	Total settlement	10 mm	15 mm	20 mm	$0.3\%H_e$(5) subject to a range of 25 mm to 60 mm	$0.5\%H_e$(5) subject to a range of 30 mm to 100 mm
Services monitoring marker(2)	Angular distortion	1:600	1:500	1:400	1:350	1:300
Building monitoring marker(3)	Angular distortion	1:1000	1:750	1:600	1:550	1:500

Notes:

(1) Only for road pavements. For other less sensitive usages such as open space, carparks and playgrounds, less stringent trigger values can be adopted with consent from stakeholders. If private roads are affected, the same trigger values could be adopted provided that agreement has been obtained from the private owners before commencement of construction works. In the event that agreement from the relevant private lot owners is found difficult to obtain, the trigger values as given in PNAP APP-137 can be adopted for the Action Levels.

(2) Trigger values of total settlement for individual services should be specified, if necessary, after consulting the relevant stakeholders.

(3) Includes free-standing structures (e.g. fence walls, guard houses), earth retaining structures, highways structures and bridge abutments). Trigger values of total settlement for individual buildings or structures should be specified, if necessary, after consulting the relevant stakeholders.

(4) Special trigger conditions:

(a) The "Action Level 1" response actions should be taken if the total settlement of vulnerable components of those services that could pose a hazard to the public has reached 20 mm or there is over 20 mm heaving in any monitoring marker;

(b) The "Action Level 2" response actions should be taken if there is any undue settlement as indicated by any monitoring markers (e.g. an increase of 5 mm between two consecutive daily readings), or significant seepage of groundwater;

(c) The "Action Level 3" response actions should be taken if any obvious ground loss, excessive ingress of groundwater and signs of distress or damage are observed in any nearby sensitive receivers; and

(d) If the estimated maximum ground settlement or angular distortion of services/buildings is exceeded, a comprehensive design review should be carried out to investigate the causes, estimate further ground settlement, and assess the likely impact to nearby sensitive receivers based on the performance of the ELS works (e.g. measured cumulative wall and ground settlement). The impact assessment and review should demonstrate that the re-estimated maximum ground settlement and angular distortion of services and buildings are still within the tolerable serviceability limits of the sensitive receivers. Mitigation and remedial works should be implemented to minimise any excessive ground settlement and angular distortion. The findings of the design review should be reported to the responsible party/authority and included in the site supervision report.

(5) H_e is the maximum excavation depth and calculated value of ground settlement should be rounded to the nearest integer value.

(6) If any obvious damage to a road or pavement is observed, the relevant stakeholders should be immediately notified and consulted to identify any potential hazard and assess the need for urgent repair and repaving works. In such cases, no further construction activities that could aggravate the ground settlement, including further lowering of the excavated level, should be allowed.

9.2.2.7 Action Levels for Changes in Groundwater Levels

Changes of groundwater level have an indirect effect on sensitive receivers and are normally allowed for in both ULS and SLS design of ELS works. Any rise of groundwater above the assumed DGWL for ULS design may affect the stability of the ELS works. On the contrary, drawdown below the lowest allowable GWL for SLS design may result in ground settlement greater than originally anticipated. Therefore, it is prudent to set trigger values of Action Level for these two assumed groundwater levels, such that timely review of design assumptions and construction quality could be carried out to confirm the safety of the ELS system and determine any precautionary measures needed. The recommended trigger values for Action Levels of DGWL are:

(a) Where another 0.5 m of groundwater level rise will reach the assumed DGWL for ULS design; and

(b) Where another 0.5 m of groundwater level drawdown will reach the lowest allowable GWL for SLS design.

Suspension of works solely due to exceedance of the groundwater Action Level is unnecessary and impractical, unless there is sudden ingress of excessive groundwater, as the impacts on the sensitive receivers are safeguarded by the control mechanism. If the changes in groundwater levels have caused ground settlement reaching the trigger values of the control mechanism, the agreed response actions should be implemented to ensure the serviceability of the sensitive receivers.

9.2.2.8 Response Actions under the Enhanced Empirical Approach

Response actions after reaching the serviceability limits of sensitive receivers typically include conducting a design review, inspecting nearby sensitive receivers for confirmation of their structural/operational safety, increasing frequency and number of monitoring checkpoints, carrying out ground treatment and other mitigation/remedial works, and liaising with relevant stakeholders. Under the 5A Approach, a set of specific and targeted response actions with respect to exceedance of the corresponding trigger values has been prepared to enhance the effectiveness and efficiency of the control mechanism and is summarised in Table 9.2. Other project or site-specific response actions could also be included if deemed necessary and appropriate. For deep excavations, the project team should place more consideration on serviceability issues of the affected sensitive receivers in formulating the Action Plan and Emergency Plan after Action Level 1 and Action Level 2 are reached. Whereas for shallow excavations when the actual ground settlement is relatively smaller (e.g. less than 30 mm), more emphasis should be put on reviewing the ULS of the ELS system.

Table 9.2 Recommended Response Actions for Exceedance of the Trigger Values (for Ground/Services/Buildings Monitoring Stations) (Sheet 1 of 3)

Trigger Level	Response Actions
Alert Level	(a) The Contractor shall promptly notify the Project Manager (PM)[1] or the AP/RSE/RGE[2] if the Alert Level is reached. (b) The Contractor shall inspect and record the conditions of the affected sensitive receivers. (c) The Contractor shall propose and implement necessary remedial measures as agreed by the PM[1] or the AP/RSE/RGE[2].
Alarm Level	(a) The Contractor shall promptly notify the PM[1] or the AP/RSE/RGE[2] and relevant stakeholders for the affected sensitive receivers, if the Alarm Level is reached. (b) The Contractor shall inspect and record the conditions of the affected sensitive receivers. (c) The Contractor shall propose and implement necessary remedial measures as agreed by the PM[1] or the AP/RSE/RGE[2]. (d) The Contractor shall carry out preparation work for reaching Action Level 1 (e.g. plan for additional I&M works, compile a stakeholders consultation contact list) as agreed by the PM[1] or the AP/RSE/RGE[2].
Action Level 1	(a) The Contractor shall promptly notify the PM[1] or AP/RSE/RGE[2] and relevant stakeholders for the affected sensitive receivers if Action Level 1 is reached. (b) The Contractor shall conduct a joint site inspection (e.g. visual inspection, CCTV inspection of services, leakage detection) with the PM[1] or the AP/RSE/RGE[2] and the relevant stakeholders and check the stability and serviceability of the affected sensitive receivers (e.g. as indicated by cracking of road pavements or buildings, leakage of services). (c) The Contractor shall propose and implement necessary remedial works (e.g. seal-up cracks, re-levelling of paving blocks) with prior agreement of the PM[1] or the AP/RSE/RGE[2] and after consulting the relevant stakeholders. (d) The Contractor shall formulate and implement an Action Plan for reaching Action Level 2 as agreed by the PM[1] or the AP/RSE/RGE[2] and after consulting the relevant stakeholders. The Action Plan should include: (i) Actions to be taken by the Contractor before Action Level 2 is triggered (e.g. requirements on additional serviceability checks, additional I&M works); (ii) Detailed investigation works to be implemented when Action Level 2 is triggered (e.g. GPR, GCO probe, open pit/trench excavation, CCTV inspection of services); (iii) Remedial works to be implemented before and when Action Level 2 is triggered (e.g. repaving, open trench repair); and (iv) Works to be suspended when Action Level 2 is triggered, if judged to be necessary.

Table 9.2 Recommended Response Actions for Exceedance of the Trigger Values (for Ground/Services/Buildings Monitoring Stations) (Sheet 2 of 3)

Trigger Level	Response Actions
Action Level 2	(a) The Contractor shall immediately notify the PM[1] or AP/RSE/RGE[2], relevant authorities (e.g. BD/GEO) and relevant stakeholders for the affected sensitive receivers if Action Level 2 is reached. (b) The Contractor shall conduct a detailed investigation (e.g. works proposed in the Action Plan, examination of the cause of any undue settlement and significant seepage), check the stability and serviceability of the affected sensitive receivers (e.g. any significant damage of subgrade and services, potential sinkholes), arrange a joint site inspection with relevant authorities and submit the investigation report to the PM[1] or the AP/RSE/RGE[2] and the relevant authorities for acceptance. (c) The Contractor shall propose and implement necessary remedial works (e.g. works agreed in the Action Plan, subgrade repairs, grouting) with prior agreement of the PM[1] or the AP/RSE/RGE[2] and relevant authorities and after consulting the relevant stakeholders. (d) The Designer of the ELS shall review and revise, if necessary, the design and method statements for the ELS works (e.g. check workmanship, estimate further ground settlement, assess impacts to nearby sensitive receivers) and seek the agreement of the PM[1] or the AP/RSE/RGE[2] and the relevant authorities to the review and any necessary design amendment. The trigger values for Action Level 3 should also be reviewed, and revised if necessary, after consulting the relevant stakeholders and submitting any amendment to the relevant authorities for approval. (e) The Contractor shall formulate and implement an Emergency Plan for reaching Action Level 3 as agreed by the PM[1] or the AP/RSE/RGE[2] and the relevant authorities and after consulting the relevant stakeholders. The Emergency Plan should include: (i) Actions to be taken by the Contractor before Action Level 3 is triggered (e.g. requirements on additional serviceability checks, groundwater recharging, retrofitting works); (ii) Full-scale investigation works to be implemented before and when Action Level 3 is triggered; (iii) Remedial and/or strengthening works to be implemented when Action Level 3 is triggered (e.g. ground improvement, additional structural support); and (iv) Works to be suspended when Action Level 3 is triggered (e.g. works that would further aggravate the ground settlement and are within 50 m of the affected sensitive receivers), if necessary. (f) The PM[1] or AP/RSE/RGE[2] shall suspend the relevant works if the required response actions are ineffective or are not implemented by the Contractor within a reasonable time frame as agreed by the PM[1] or the AP/RSE/RGE[2]. The Contractor shall not resume the suspended works unless agreed by the PM[1] or the AP/RSE/RGE and the relevant authorities.

Table 9.2 Recommended Response Actions for Exceedance of the Trigger Values (for Ground/Services/Buildings Monitoring Stations) (Sheet 3 of 3)

Trigger Level	Response Actions
Action Level 3	(a) The Contractor shall suspend relevant works (e.g. works agreed in the Emergency Plan, works affecting public safety) and immediately notify the PM[1] or the AP/RSE/RGE[2], the relevant authorities and the relevant stakeholders for the affected sensitive receivers. (b) The Contractor shall conduct a full-scale investigation (e.g. works proposed in the Emergency Plan, examination of the cause of any signs of distress and excessive water ingress), check the stability and serviceability of the affected sensitive receivers (e.g. any significant damage of subgrade and services, potential sinkholes) and arrange a joint site inspection with the relevant authorities. The Contractor shall prepare and submit an investigation report for acceptance by the PM[1] or the AP/RSE/RGE[2], and approval by the relevant authorities. (c) The Contractor shall propose and implement necessary remedial/strengthening works (e.g. works agreed in the Emergency Plan, backfilling of potential sinkholes) with prior agreement of the PM[1] or the AP/RSE/RGE[2] and the relevant authorities after consulting the relevant stakeholders. (d) The Designer of the ELS shall re-examine and revise the design and method statements of the ELS works for approval by the PM[1] or the AP/RSE/RGE[2] and the relevant authorities. (e) The Contractor shall formulate a Works Resumption Plan for agreement by the PM[1] or the AP/RSE/RGE[2] and the relevant authorities after consulting the relevant stakeholders. The Works Resumption Plan should include: (i) A condition survey of the affected sensitive receivers after remedial and/or strengthening works; (ii) Revised design and method statements as approved by the relevant authorities; (iii) Trigger values for further response actions; and (iv) Details of further response actions. (f) The Contractor shall not resume the suspended works unless all the necessary remedial/strengthening works have been completed and the Works Resumption Plan is accepted by the PM[1] or the AP/RSE/RGE[2] and approved by the relevant authorities.

Notes:
(1) Condition 1 is applicable to public works projects managed by the Project Manager of the contract.
(2) Condition 2 is applicable to private projects under the ambit of the Buildings Ordinance.
(3) During any period of works suspension, continuous monitoring should be carried out in order to keep appraising changes of the site conditions and to assist in implementing any necessary response actions.
(4) Works that are essential for maintaining stability (e.g. strut installation and preloading) should not be suspended.

9.2.2.9 Response Actions for Changes in Groundwater Levels

Where a groundwater rise approaches the assumed highest DGWL, the design review should include examining the safety margins and conducting sensitivity checks to assess whether the overall stability of the ELS system remains satisfactory. In addition, the cause of the unexpected rise should be investigated, such as possible leakage of water-carrying services, improper arrangement of site drainage, or water damming by the embedded wall. In some instances, suitable openings formed at appropriate levels in the embedded wall could help alleviate the problem. However, such dewatering effect to the nearby sensitive receivers due to wall opening should be duly assessed.

On the other hand, excessive groundwater drawdown would likely cause additional ground settlement. The cause of additional groundwater drawdown should be investigated, such as leakage between contiguous piles, deterioration of grout curtains, and insufficient embedment depth of wall. Precautionary measures should be determined based on the causes of the problems, such as regrouting at preserved TAM grout pipes or operating a recharge well to counteract the groundwater drawdown.

The recommended typical response actions with respect to groundwater trigger levels are summarised in Table 9.3.

Table 9.3 Recommended Response Actions for Exceedance of the Groundwater Trigger Values (for Groundwater Monitoring Stations)

Trigger Level	Response Actions
ULS Design (0.5 m of groundwater level below the highest DGWL) or SLS Design (0.5 m of groundwater level above the lowest allowable GWL)	(a) Inspect and examine the performance of the ELS works and the response of nearby sensitive receivers with respect to their structural stability and serviceability. (b) Investigate causes and any correlations with observed changes in groundwater levels. (c) Carry out design review to estimate further potential change of groundwater level and assess impacts on nearby sensitive receivers if necessary. (d) Enhance monitoring by increasing the frequency of measurements and the number of monitoring stations if necessary. (e) Implement necessary mitigation and remedial measures and consult the relevant stakeholders. (f) Review and revise the trigger values of groundwater level change with justifications based on results of the design review and impact assessment on nearby sensitive receivers.

9.2.3 Engineering Approach

For particularly sensitive receivers requiring special care (e.g. historical buildings, dilapidated structures, tunnels, railway structures, service reservoirs) or for deep excavations where the estimated maximum ground settlement is larger than the empirical limits, an engineering approach can be adopted to assess tolerable limits of the sensitive receivers with respect to their stability and serviceability, and to establish site-specific trigger values for differential settlement (in terms of angular distortion), as well as total settlement if necessary. Suitable trigger values of the Action Levels could then be established, with consultation and agreement from the relevant stakeholders. For example, PNAP APP-24 (BD, 2022) provides guidance on adopting an engineering approach for protecting the MTR structures and facilities. In using the engineering approach, it is technically acceptable to set the maximum estimated ground settlement as the trigger value of Action Level 3 if such settlement is determined to be tolerable by nearby sensitive receivers and accepted by the relevant stakeholders and authorities.

However, in deriving site-specific Action Levels, it is important to keep in mind that different structures, facilities and utilities will have different tolerance limits for accommodating ground settlement. Thus, the acceptable level of settlement may vary and the assessment should be considered on a case-by-case basis. The structural stability and serviceability of sensitive receivers should be assessed by a detailed engineering analysis based on impact assessment of the design estimation of ground settlement.

If the estimated maximum ground settlement of the ELS works is found to have exceeded the tolerable limits of sensitive receivers, precautionary measures should be proposed, such as ground improvement or underpinning of concerned buildings/structures/services before works commencement, after consultation with the relevant stakeholders. This should help to minimise the estimated ground settlement or enhance the tolerance of sensitive receivers.

For complicated ELS works, it is a good practice to formulate a set of stepped Action Levels, which should be specified on plan to cater for each pre-defined critical stage (e.g. piling, bulk excavation), with due consideration of the construction sequence and likely impact on nearby sensitive receivers (e.g. the provisions set out in PNAP APP-24 (BD, 2022)). This should assist the project team in reviewing the performance of the ELS works at different critical construction stages, and for implementing appropriate response actions where necessary.

9.2.4 Additional Control Measures for Potential Sinkhole Formation

Since the implementation of enhanced geotechnical control on excavation works, including submissions for statutory approval of ELS works and imposition of qualified supervision, incidents of collapse of ELS systems have become rare. However, cases of excessive ground loss and sinkhole formation associated with deep excavations are still commonly encountered, some of which have caused damage to properties/roads and even resulted in injuries to the public. In 2019, the GEO conducted a review of such incidents and identified the common contributory factors of excessive ground loss and sinkhole formation (Lee, 2019).

Among the common aspects of these incidents, disturbance to adjacent ground during piling operations was evident in many cases, especially for operations involving extraction of soil from the ground, e.g. concentric drilling with high flushing air pressure, and grabbing and augering in bored piles. The cavities that formed within the soil mass were often not immediately detected or reflected in ground settlement monitoring during the piling operation, due to soil arching over the cavities. It took time for the cavities to collapse and propagate towards the ground surface, eventually affecting buried water-carrying services or road pavements. Leakage from affected water-carrying services could further aggravate the problem of sinkhole formation.

There have also been a few cases of sinkholes formed during the bulk excavation stage, where the ground deformation caused by the excavation or by application of preloading to the embedded wall resulted in changes of lateral stress in the soil mass, leading to collapse of the metastable arching effect above the cavities.

As part of the enhanced control mechanism, additional control measures for the detection of underground cavities are recommended for sites with ground conditions that are prone to sinkhole formation (GEO, 2023b). Besides the usual precondition survey of the ground and services prior to commencement of site works, investigations using GPR or CCTV survey should be conducted on completion of the embedded wall and at regular intervals (e.g. once every three months) during bulk excavation with dewatering. Where the results indicate abnormalities, GCO probing or SPTs with precautionary safety measures should be conducted to help identify the presence of possible cavities at depth. In addition, a professionally qualified land surveyor should be engaged to check and certify the monitoring results at regular intervals. Such measures are aimed at providing opportunities for early detection of cavities and prevention of injuries to the public or workers on site.

9.3 Concurrent Construction Activities

Concurrent construction works in close proximity pose particular challenges for the design and construction of ELS works, as sensitive receivers may be adversely affected by both the site works and the adjacent construction works. In any case, the cumulative effect of concurrent construction works on individual sensitive receivers should always be duly considered in the design of each project. In such situations, strong cooperation and close coordination between the project teams of the concurrent projects are of utmost importance for successful execution of the respective projects. The senior management teams, consultants and project offices should be involved at an early design stage in order to resolve any difficulties and differences. The estimated ground deformation and the control mechanism at each site should take due account of the works sequence and construction programmes of both projects. It is possible that delays to individual construction stages may invalidate some of the design assumptions. Therefore, suitable hold points or staged consent procedures may need to be established for each project to confirm whether the assumed conditions are satisfactory, before proceeding with the next stage of excavation works.

When drawing up the I&M plan, consideration should be given to suitable positioning of instruments such that the effects of the excavation could be better delineated, such as installing inclinometers at suitable distances away from the excavation, as well as at the site boundary. Monitoring of instruments should also be carried out at opportune times, for instance, to capture the ground deformation on completion of certain stages of excavation. A joint survey mechanism should be established and implemented by all parties involved in the concurrent excavation projects to avoid inconsistent monitoring of the same sensitive receiver by different parties.

Concurrent foundation and excavation works may complicate assessment of the causes of excessive settlement. In some cases, it may be unrealistic to identify with certainty the parties responsible for a particular problem. Subsequent dispute and argument may delay the implementation of necessary response actions, which could lead to further deterioration of the site conditions. Close coordination between different project teams is needed and it is essential to undertake response actions specified in the control mechanism in a comprehensive and timely manner. Where necessary, senior members of the project management teams, consultants, contractors, project offices and relevant government authorities should be involved in order to expedite appropriate follow-up actions.

10 Instrumentation, Monitoring and Novel Technology

10.1 General

I&M is essential to verify the design assumptions (e.g. groundwater conditions, magnitude of ground deformation) and evaluate the performance of ELS works. Instruments are also installed on sensitive receivers to detect any adverse impacts on them. Monitoring results should be fed back into the design review process and assessment of the need to implement additional precautionary measures where necessary. A proper I&M system should be formulated and appropriate response actions should be included as part of the design of ELS works. The principles of the control mechanism and recommendations for response actions are set out in Chapter 9.

10.2 Instrumentation and Monitoring Plan

10.2.1 General Requirements

Geotechnical I&M should be carefully planned, with a clearly defined purpose for reviewing the performance of the ELS system and its impact on nearby sensitive receivers. Guidance on the planning of monitoring for the design of retaining walls, as given in Geoguide 1, is also generally applicable for ELS works. A site-specific I&M plan should be formulated and should include the following general requirements:

(a) Monitoring checkpoints for sensitive receivers located within the influence zones of the construction site (especially for structures vulnerable to ground settlement where the relevant stakeholders should be consulted, e.g. on-grade retaining structures with movement joints, masonry walls, railway structures);

(b) Types of instruments for monitoring each set of measurements (e.g. movement, tilting, vibration);

(c) Monitoring frequencies and the reporting cycle;

(d) The parties responsible for installing, monitoring and maintaining the instruments;

(e) The control mechanism for each monitoring station installed on sensitive receivers and the corresponding response actions and action parties when the trigger values are approached or reached (see Chapter 9); and

(f) A response action plan to allow prompt actions (e.g. strengthening works, sealing of cracks on buildings, water recharging) for any unexpected signs of distress to nearby sensitive receivers, especially for vulnerable buildings or structures.

10.2.2 Types of Instruments

Different types of instruments are available to suit the specific requirements of monitoring works, which usually include measurements of the following:

(a) Movements and vibrations of sensitive receivers;

(b) Groundwater level, piezometric pressure and groundwater inflow rate;

(c) Deflection of the embedded wall; and

(d) Loads and strains in critical structural elements of the lateral support system (e.g. pre-loaded struts, ground anchors).

Table 10.1 provides a summary of various types of instruments that are commonly used to monitor the performance of ELS works. The instruments should be properly installed, calibrated and maintained. Regular checks and repairs should be conducted to ensure proper functioning of the instruments. Further guidance on calibration, installation and operation of instruments is given in the Geotechnical Manual for Slopes. Other literature on the design and specification of geotechnical instrumentation can be found in Dunnicliff (1993) and Dunnicliff et al (2012).

The choice of instruments should be based on the required accuracy of the measurements, reliability and response time of the instruments and site conditions. Typical working accuracies of some commonly used monitoring methods are discussed in Geoguide 1.

10.2.3 Groundwater Monitoring

Piezometers and standpipes are often adopted for measuring groundwater levels. However, their locations, length and positioning of response zones should be properly planned with due consideration given to the hydrogeological condition of the groundwater regime adjacent to the site. Where the ground conditions comprise stratified marine clay underlain by sandy soils (e.g. alluvial sand), the porewater pressure in the sandy soil may have been influenced by dewatering within the excavation. In such cases, piezometers should be installed in the sandy soil so as to identify any drop in piezometric head.

10.2.4 Horizontal Deformation Monitoring

Inclinometers are often used for monitoring horizontal deformation below the ground surface induced by the excavation. They should be installed to a fixed datum, e.g. by extending to bedrock, to ensure that they do not move laterally. This will allow computation of the absolute horizontal movement at any point along the inclinometer. Inclinometer guide casings have tracking cross grooves that control the orientation of the probe. Care should be taken during installation to ensure the grooves are properly aligned with the excavation face. Inclinometers should preferably be placed in soil at varying distances from the embedded wall, and where possible, a number of inclinometer sets should be installed to assist in determining the overall influence zone of the excavation. Such an arrangement is also useful in differentiating the impact of concurrent construction activities.

Table 10.1 Instruments for Monitoring of ELS Works

Objectives	Measurement Type	Instruments
To verify the estimated ground deformation and performance of the ELS system	Ground deformation or the deflection of the exposed parts of the embedded wall	▪ Deformation monitored by using conventional surveying methods, laser scanner, micrometer stick ▪ Surface extensometers (e.g. tape extensometers) ▪ Automatic deformation monitoring surveying (ADMS) by using robotic total station and target prisms
	Subsurface vertical deformation of the ground including heaving	▪ Extensometers
	Subsurface horizontal deformation of the ground or embedded wall	▪ Inclinometers
	Load and strains in the embedded wall and structural elements	▪ Strain gauges ▪ Load cells
To monitor the change in groundwater level	Groundwater level	▪ Standpipes (can be installed with automatic groundwater monitoring devices)
	Piezometric pressure	▪ Piezometers (can be installed with automatic groundwater monitoring devices)
	Groundwater inflow rate	▪ Flow meter
To monitor the response of sensitive receivers	Total and differential movement	▪ Movement monitored by conventional surveying methods, or an ADMS, laser scanner survey, or micrometer stick ▪ Photogrammetric survey ▪ Surface extensometers (e.g. tape extensometers)
	Tilting	▪ Movement monitored byconventional surveying methods, or an ADMS, laser scanner, or micrometer stick ▪ Tiltmeters or tilt sensors ▪ Cable-free tilt sensors
	Cracking	▪ Telltales ▪ Demec gauges
	Vibration	▪ Vibrographs or suitable smartphones with accelerometers
	Strain	▪ Strain gauges

Timing of groundwater level measurement should be properly planned such that the most critical situation could be captured. For excavation sites in reclaimed land where the groundwater table is highly influenced by tidal action, groundwater levels should be measured at least twice daily, at high tide and low tide.

Where inclinometers are embedded in a bored pile wall or diaphragm wall, they are often installed in steel preservation tubes so as to minimise the subsequent drilling works. The casing guide within the steel preservation tube is sometimes filled with cement bentonite balls as a sealant. However, it should be noted that the stiffness of the sealant and the presence of the steel preservation tube are incompatible with the concrete placed in a bored pile wall or diaphragm wall. In such cases, the measured deflection from the inclinometer may not adequately reflect that of the embedded wall. Alternatively, inclinometers may be installed in proof core drill holes, which are sometimes specified as quality control measures.

10.3 Novel Technology

The use of wireless digital technology such as the Internet-of-Things (IoT) and digital twin platforms with BIM allows instant remote access to real time monitoring data through online platforms. This shortens the time required for reporting and processing monitoring data and enhances the effectiveness of information dissemination among different decision makers and action parties involved in implementation of the monitoring plan. It can also facilitate the evaluation of the monitoring results for early decisions and quick actions.

Ammar et al (2022) reported digital twin applications for real-time monitoring of water levels during dewatering for excavation works, while Haryono et al (2021) reported the application of a digital twin model in a deep shaft excavation project involving complex geology to quickly assess the potential effects of movement on construction arising from the difficult ground conditions. Real-time data can be sent to the project office which enables prompt analysis of site conditions and review of the site performance against design predictions.

Real time monitoring of digital instrumentation sensors, including strain gauges and inclinometers, in a local deep excavation project in Hong Kong was reported by Toh et al (2023) for evaluating the performance of the excavation. Application of the digital sensors also extended to water level monitoring, which facilitated the deployment of faster flood risk mitigation measures (Toh et al, 2023). Furthermore, they also showcased the application of BIM throughout the construction project life cycle to assist in formulating an optimised sequence of construction.

In addition, the use of digital tools, aided by block chain technology, has facilitated the making of traceable digital site supervision and monitoring records and instant uploading of such records onto an IoT server for robust record keeping purposes. It is recommended, however, that monitoring data obtained directly from digital tools should be checked regularly by conventional surveying methods in order to verify their accuracy and reliability.

Advanced remote sensing techniques such as Light Detection and Ranging (LiDAR) scanning by unmanned aerial vehicles or hand-held devices have also become popular in recent years and have great potential to facilitate the interpretation of monitoring data and assess construction site progress. Li et al (2022) reported the application of handheld LiDAR scanning of the peanut-shaped deep excavation illustrated in Figure 10.1. Toh et al (2023) incorporated 3D scanning from handheld laser scanners into BIM models for clash detection. These techniques facilitate visualisation of a 3D construction sequence and are useful tools for prompt cross-checking of construction progress and compliance with the approved construction scheme. However, their positional accuracy sometimes does not match that provided by conventional surveying methods and they are currently more suitable to serve as a complementary method in monitoring deflection of an embedded wall or ground deformation.

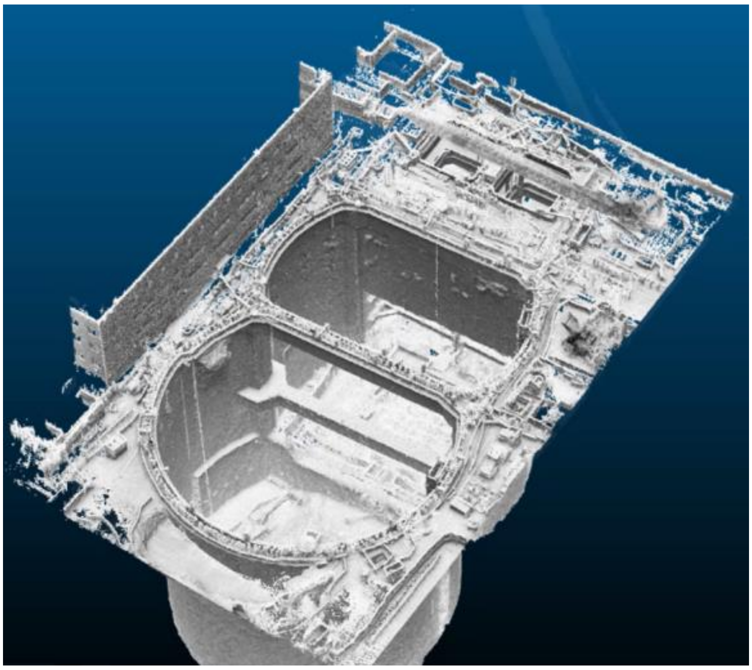


Figure 10.1 Application of Handheld LiDAR Scanning in ELS Works

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Appendix A Database of Ground Deformation and Wall Deflection Monitoring Collected in Selected ELS Works

Case	Location	ELS Works	Geology[2]	Maximum excavation depth (m)	Maximum ground settlement (wall installation) (mm)	Maximum total ground settlement (mm)	Reference[1]
1	Chater Station	37 m deep and 1.2 m thick diaphragm wall, top down construction with 3 layers of support	Fill/MD/CDG	27	78	182	Davies, & Henkel (1980)
2	HSBC HQ (abutting Des Vouex Road)	30 m deep and 1 m thick diaphragm wall, top down construction with 3 layers of support	Fill/MD/CDG	17	17	32	Humpheson et al (1986)
3	HSBC HQ (abutting Queen's Road Central)		Fill/MD/CDG	16	17	38	
4	Sheung Wan crossover	43 m deep diaphragm wall, top down construction with 4 layers of support	Fill/MD/ALL/CDG	30	30	56	Fraser (1992)
5	Dragon Centre	47 m deep and 1.2 m thick diaphragm wall, top down construction with 4 layers of support	Fill/MD/CDG	27	No available data	39	Lui & Yau (1995)
6	Tsuen Wan West Station (new reclamation)	22 m & 35 m deep and 1.2m thick diaphragm wall, bottom up construction with 2 layers of support	Fill/MD/ALL/CDG	19	52	85	Pickles et al (2003)
7	Tsuen Wan West Station (old reclamation)		Fill/MD/ALL/CDG	19	10	36	
8	Redevelopment at Nathan Road redevelopment	25 m deep and 0.4 m dia. pipe pile, bottom up construction with 5 layers of support	Fill//ALL/CDG	20	No available data	8	Yau & Sum (2010)
9	Development at Jordan Road	33 m deep and 1 m thick diaphragm wall, bottom up construction with 4 layers of support	Fill/MD/ALL/CDG	23	No available data	20	
10	Development at Luen Wo Hui	40 m deep and 1.2 m thick diaphragm wall, top down construction with 4 layers of support	Fill/ALL/CDV	17	No available data	6	Leung (2005)
11	Charter House	48 m deep and 1.2 m thick diaphragm wall, top down construction with 3 layers of support	Fill/MD/ALL/CDG	15	No available data	30	Sze & Young (2003)
12	Development at Murray Road	40 m deep and 1.5 m dia. bored pile wall, top down construction with 5 layers of support	Fill/COLL/CDG	20	5	9	GEO study
13	Development at Wanchai	48 m deep and 0.6 m dia. pipe pile wall, bottom up construction with 12 layers of tie-backs	Fill/CDG/MDG	45	No available data	5	GEO study

Case	Location	ELS Works	Geology[(2)]	Maximum excavation depth (m)	Maximum ground settlement (wall installation) (mm)	Maximum total ground settlement (mm)	Reference[(1)]
14	Development at Admiralty	39 m deep and 1.2 m thick diaphragm wall, bottom up construction with 7 layers of support	Fill/MD/ALL/CDG	31	30	102	GEO study
15	Development at To Kwa Wan	42 m deep and 1.2 m thick diaphragm wall, top down construction with 4 layers of support	Fill/ALL/CDG	32	29	63	GEO study
16	Roadworks at Kwun Tong	49 m deep and 1.5 m thick diaphragm wall in peanut shape, bottom up construction with 6 layers of support	Fill/MD/ALL/CDG	36	8	27	GEO study
17	Roadworks at the artificial island of HK Boundary Crossing Facilities	56 m deep and 1.5 m thick diaphragm wall in caterpillar shape	Fill/MD/ALL/CDG	42	No available data	30	GEO study
18	Development at Hong Kong Airport	19 m deep and 0.61 m dia. pipe pile wall, bottom up construction with 3 layers of tie-backs	Fill/CDG/MDG	14	No available data	11	GEO study
19	Development at Power Station	38 m deep, 0.61 m dia. pipe pile wall in circular shape	Rockfill/CDG	28	25	37	GEO study
20	Development at Festival Walk	36 m deep and 1.2 m thick diaphragm wall, 2 layers of tied-backs and 5 layers of permanent slabs	Fill/CDG	32	No available data	8	Lee et al (2001) & Wang, Y. (2000)
21	Argyle Station	30 m deep and 1.1 m dia. secant bored pile, top down construction with 3 layers of support	Fill/MD/CDG	25	No available data	18	Morton et al (1980)
22	Wong Tai Sin Station	27 m deep and 0.9 m thick diaphragm wall, top down construction with 2 layers of support	Fill/ ALL/CDG	19	14	58	
23	Development at Stubbs Road	43 m deep and 3 m dia. bored pile wall, bottom up construction with 19 layers of tie-backs	CDG/HDG/MDG	38	No available data	5	Lam (2018)
24	Development at Hoi Fai Road, West Kowloon	33 m deep type IV sheet pile wall, top down construction with 3 layers of support	Fill/ALL/CDG/MDG	14	No available data	18	Sze & Lau (2010)
25	Development at Tseung Kwan O	44 m deep and 1 m thick diaphragm wall, bottom up construction with 3 layers of support	Fill/MD/ALL/CDV	10	No available data	35	GEO study

Case	Location	ELS Works	Geology[2]	Maximum excavation depth (m)	Maximum ground settlement (wall installation) (mm)	Maximum total ground settlement (mm)	Reference[1]
26	Development at Kai Tak Area 4C	32 m deep Type VIL sheet pile wall, bottom up construction with 5 layers of support	Fill/MD/ALL/CDG	16	No available data	21	GEO study
27	Development at Kai Tak Area 4A	25 m deep Type IV sheet pile wall, bottom up construction with 4 layers of support	Fill/MD/ALL	13	No available data	16	GEO study

Notes:

(1) For the references marked "GEO study", information has been extracted by the GEO from available monitoring records of ELS works.

(2) MD is marine deposits, ALL is alluvium, COLL is colluvium, CDG is completely decomposed granite, CDV is completely decomposed volcanic rock, HDG is highly decomposed granite and MDG is moderately decomposed granite.

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The Geotechnical Division of the Hong Kong Institution of Engineers (HKIE) established a Task Force to conduct a technical review and recommend updates of the GCO Publication No. 1/90. A Technical Review Report of Design Methods for Excavation was produced by the Task Force in 2021. The Technical Review Report made suggestions on the key updates of the GCO Publication No. 1/90, which greatly facilitate the Working Group in updating this publication. The effort of the Task Force is gratefully appreciated. The HKIE Task Force was led by Ir Dr Richard Pang, with the following members contributed a great deal of time and effort to the preparation of the Technical Review Report:

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