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.

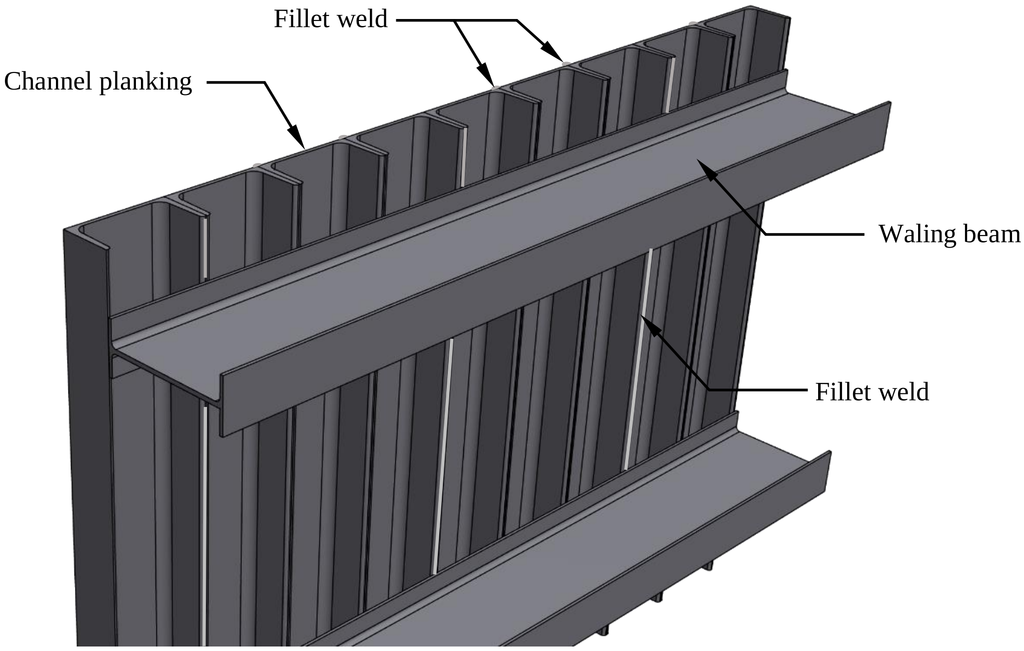

Figure 3.1 Typical Channel Planking Wall

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.

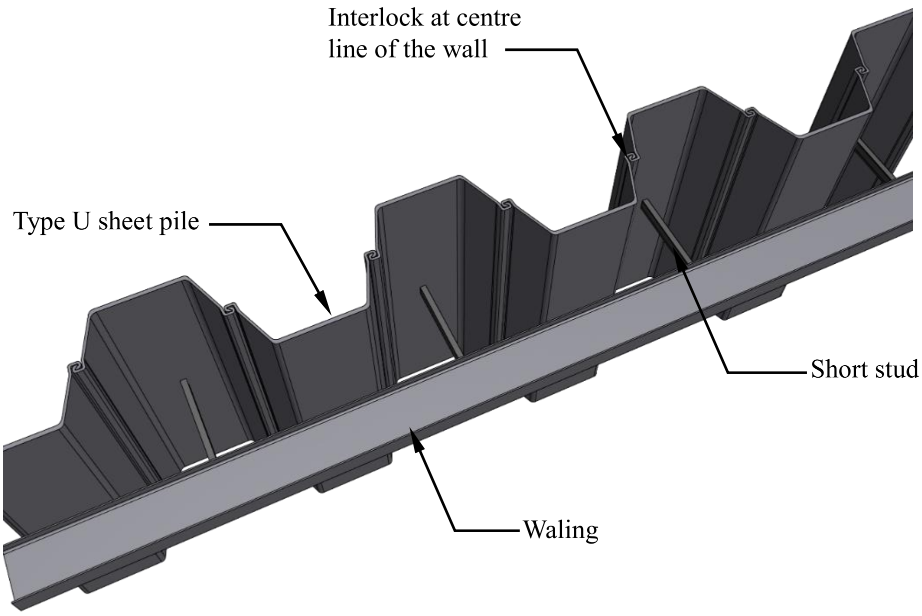

Figure 3.2 Typical Type U Sheet Pile Wall

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.

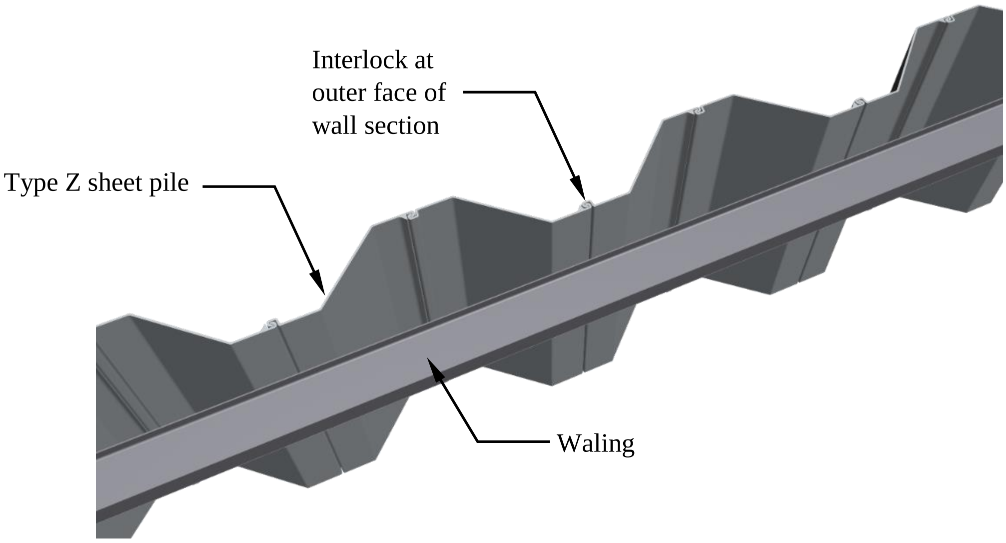

Figure 3.3 Typical Type Z Sheet Pile Wall

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.

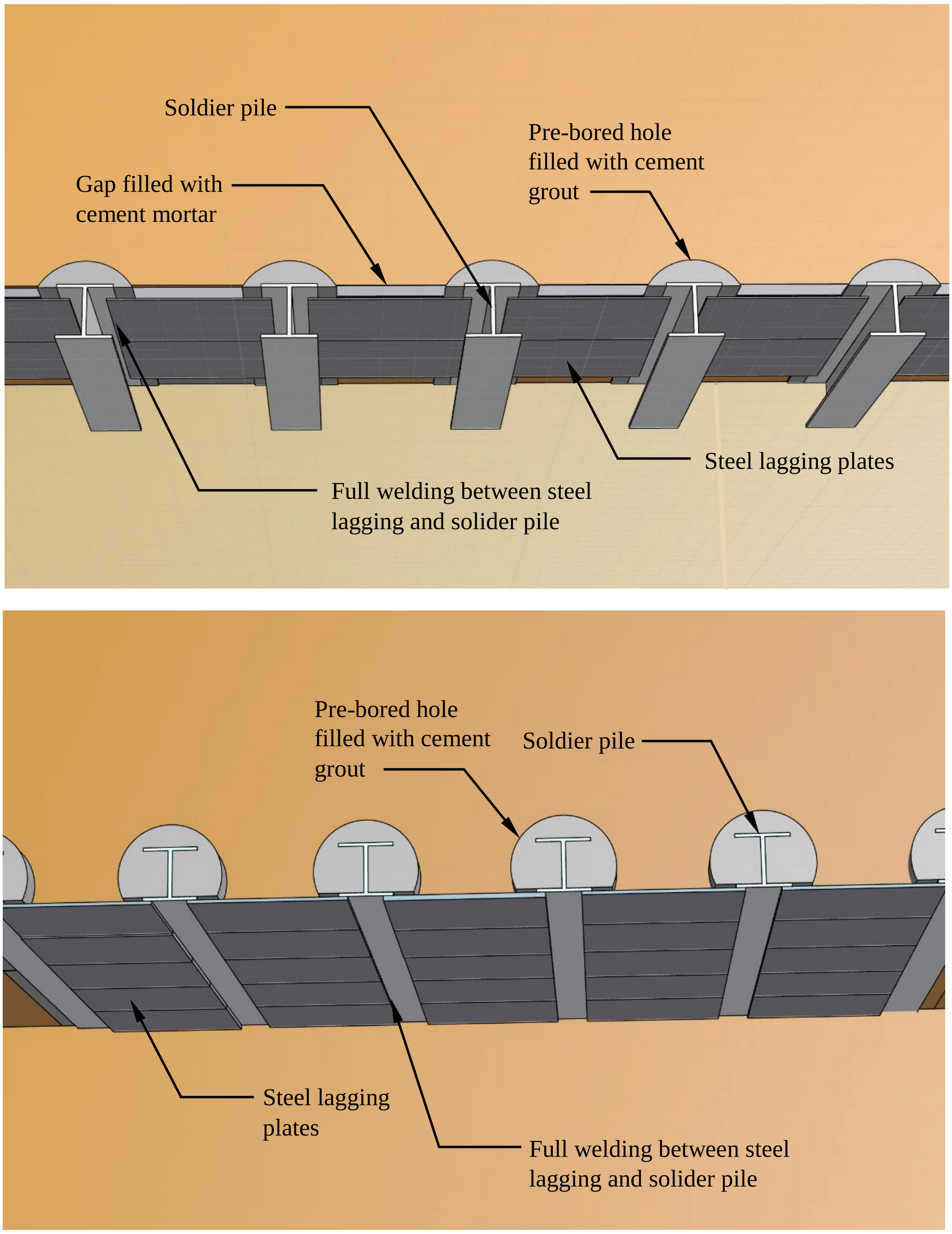

Figure 3.4 Typical Soldier Pile Wall with Steel Lagging

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.

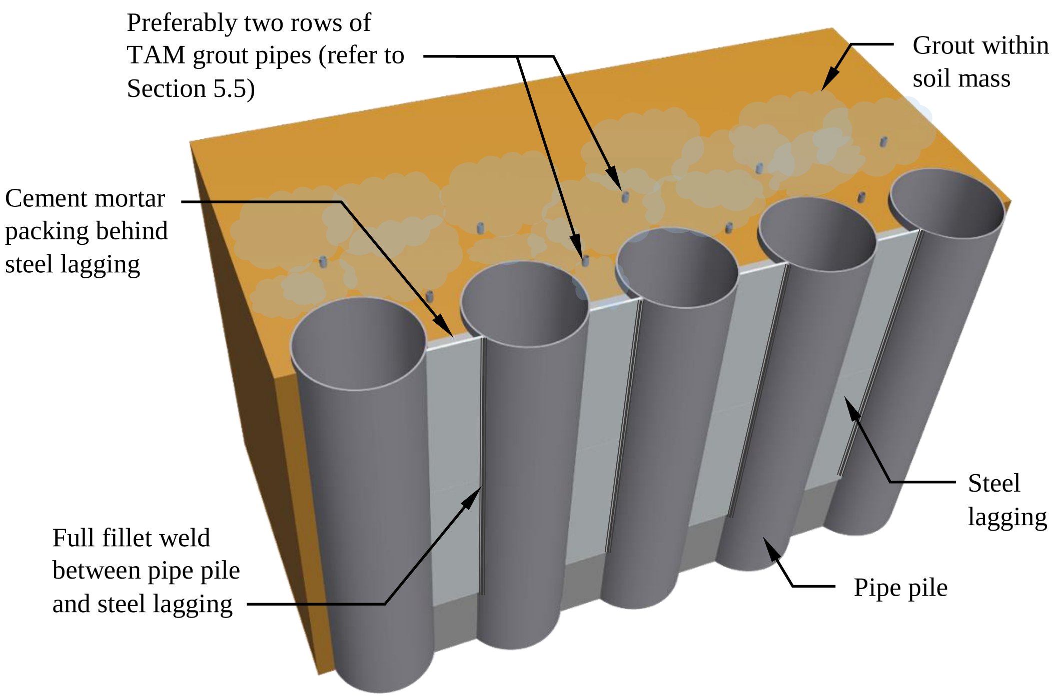

Figure 3.5 Typical Pipe Pile Wall with Grout Curtain

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.

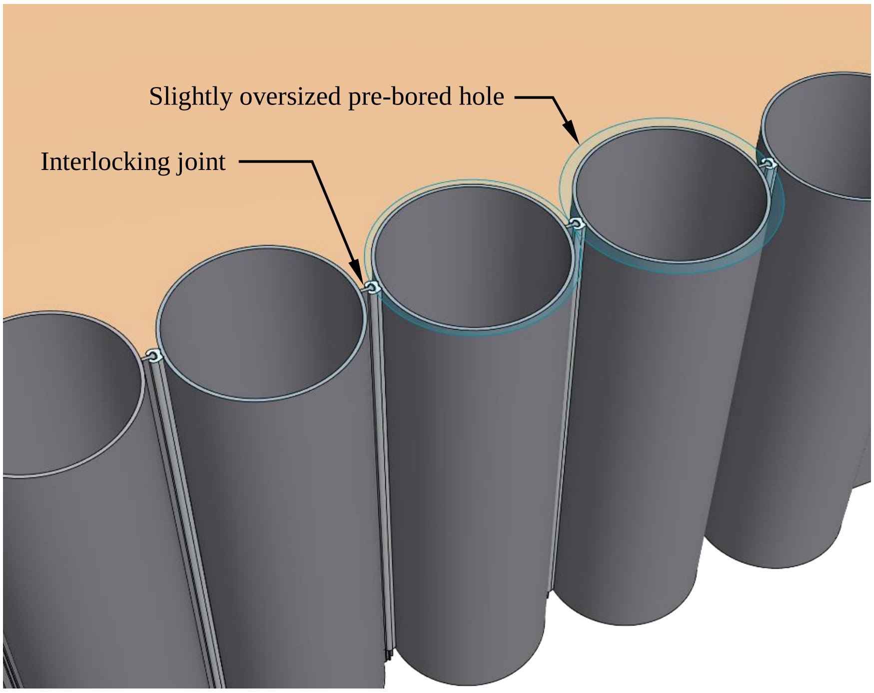

Figure 3.6 Typical Interlocking Pipe Pile Wall

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.

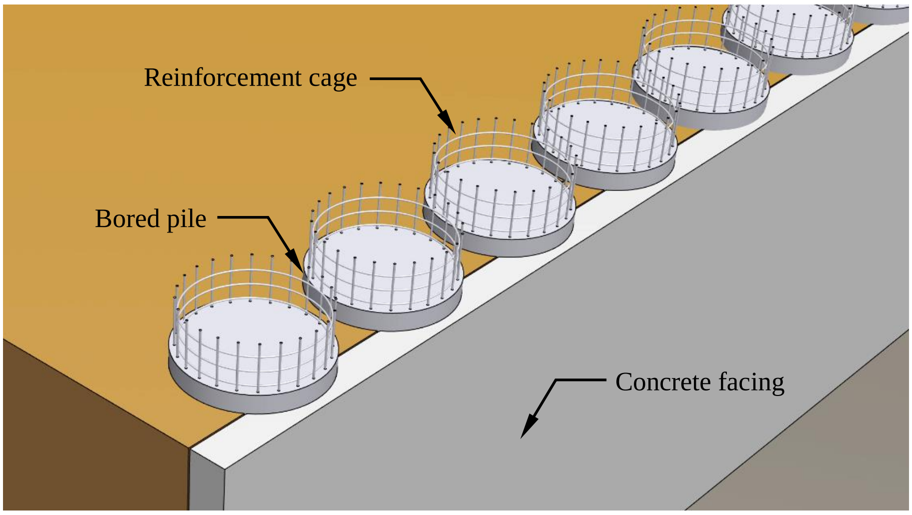

Figure 3.7 Typical Contiguous Bored Pile Wall

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.

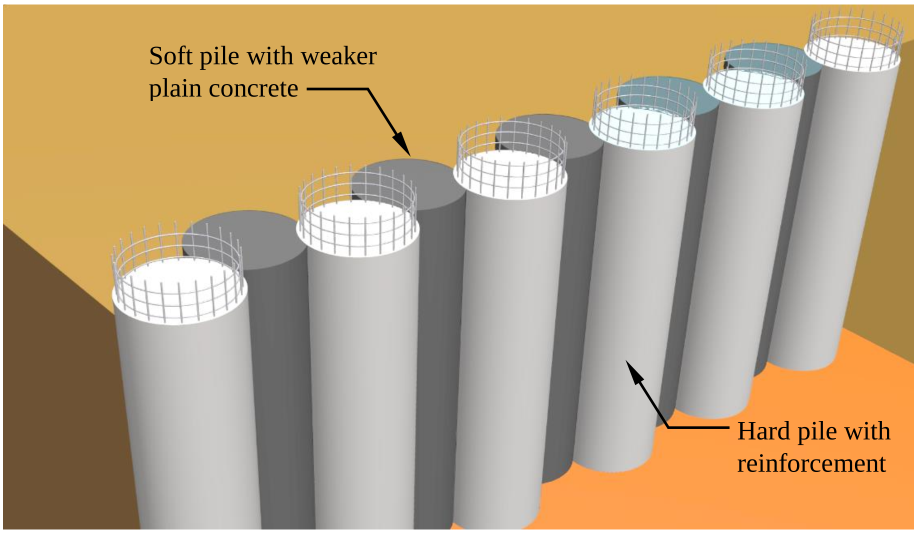

Figure 3.8 Typical Secant Bored Pile Wall

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.

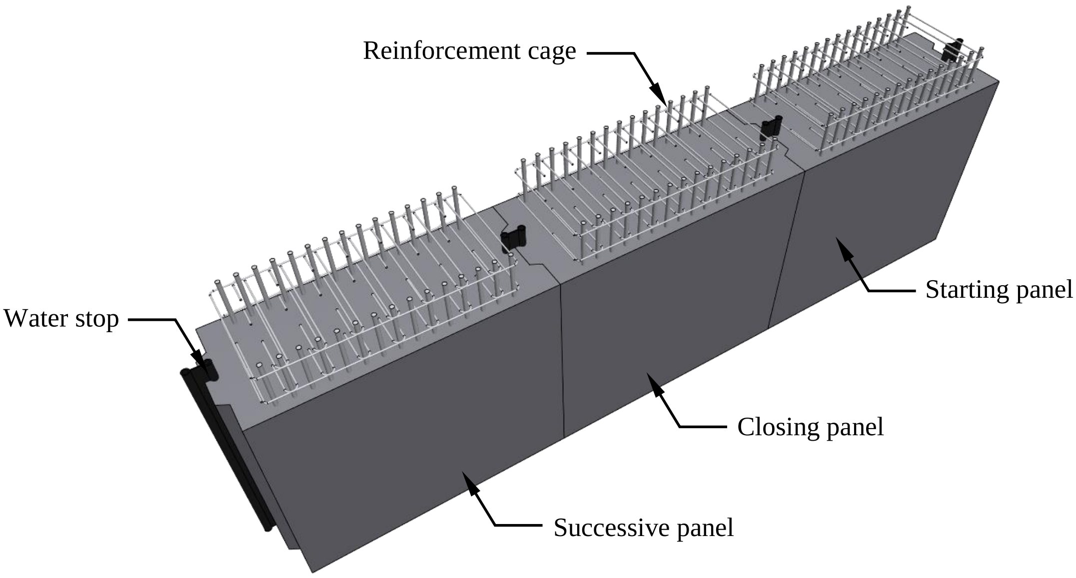

Figure 3.9 Typical Diaphragm Wall

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.

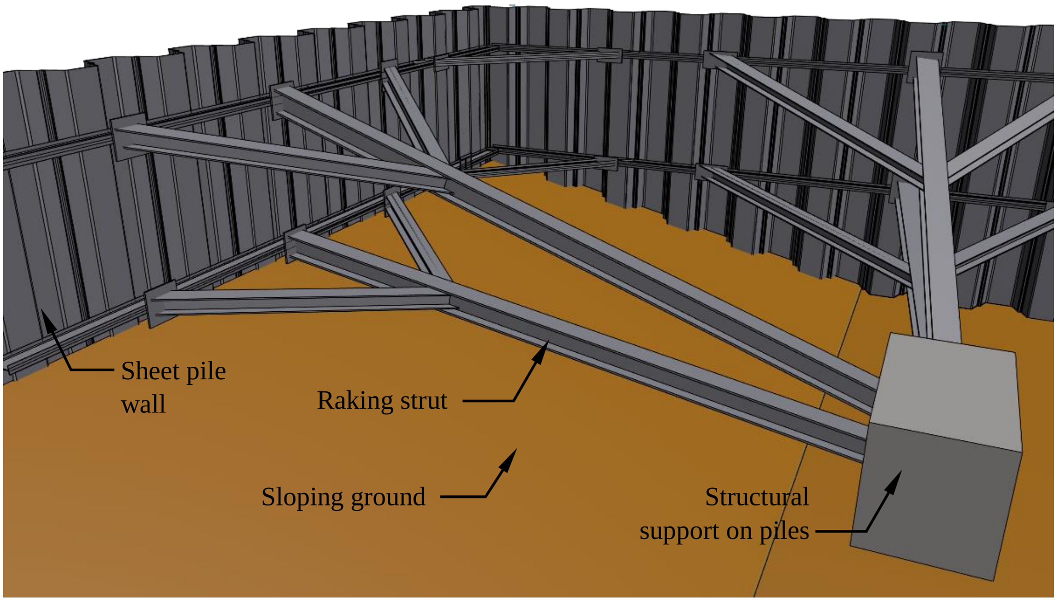

Figure 3.10 Strutting System with Raking Struts

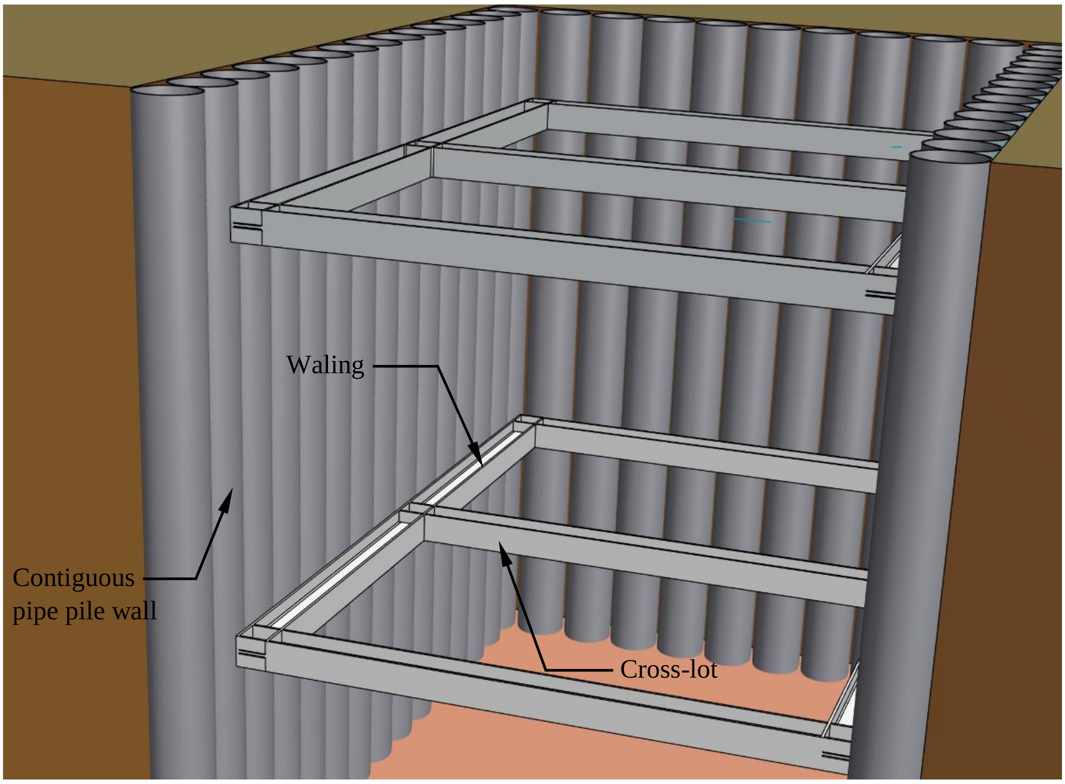

Figure 3.11 Strutting System with Cross-lot

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.

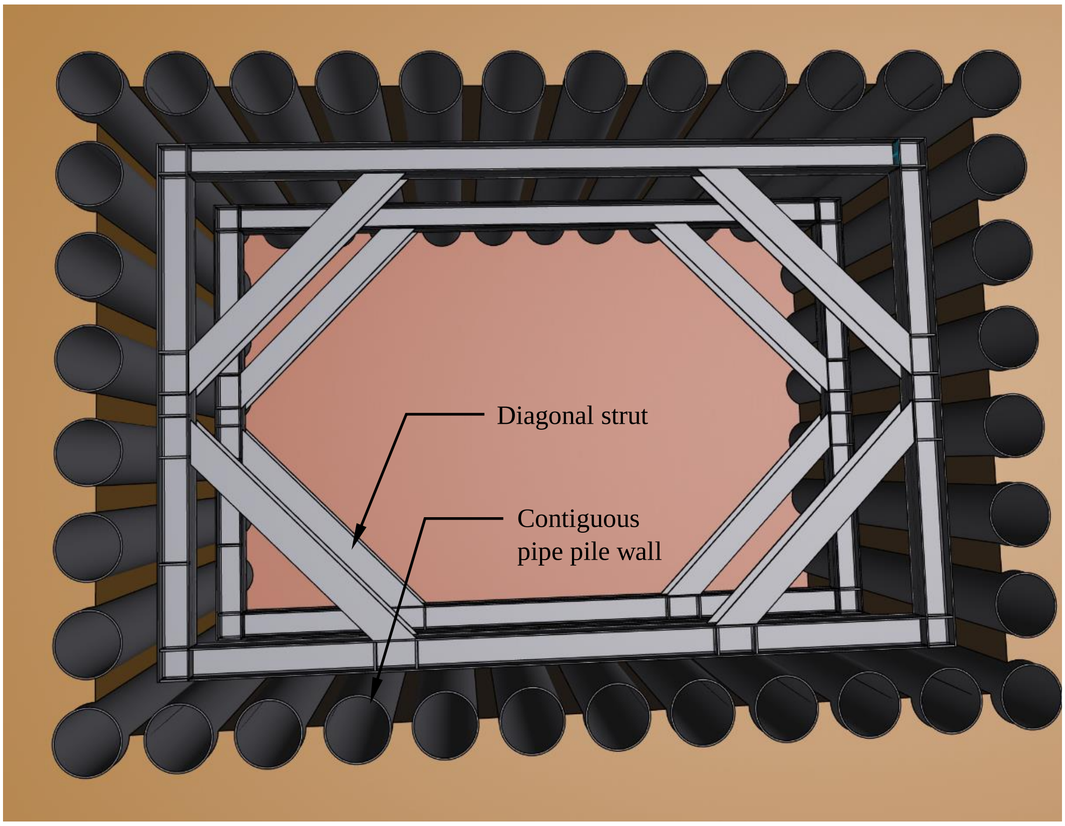

Figure 3.12 Strutting System with Diagonal Struts

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.

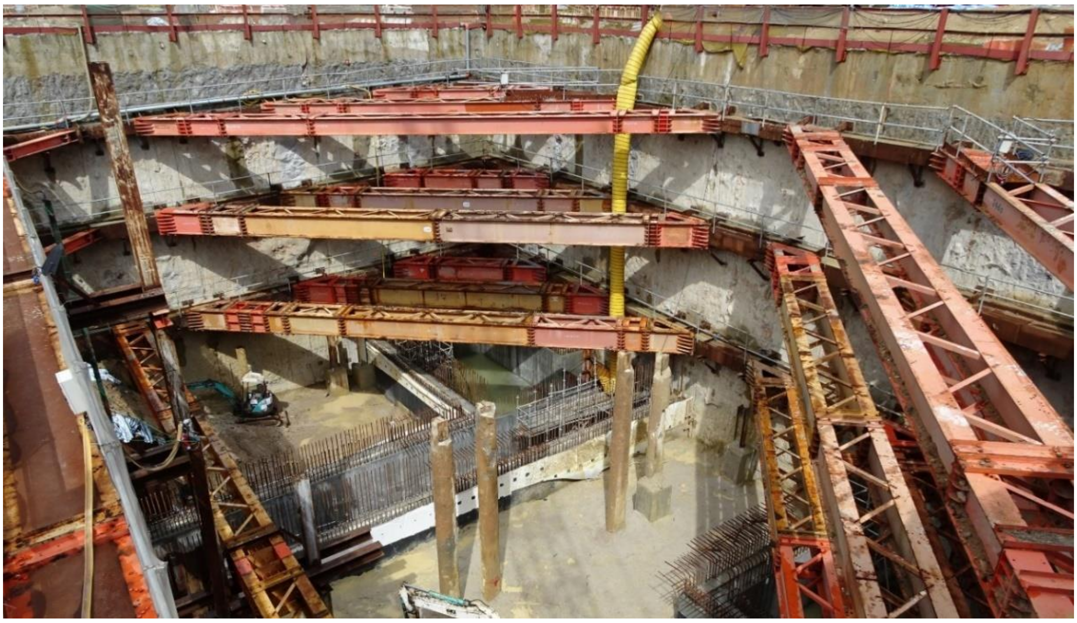

Figure 3.13 Bottom-up Construction

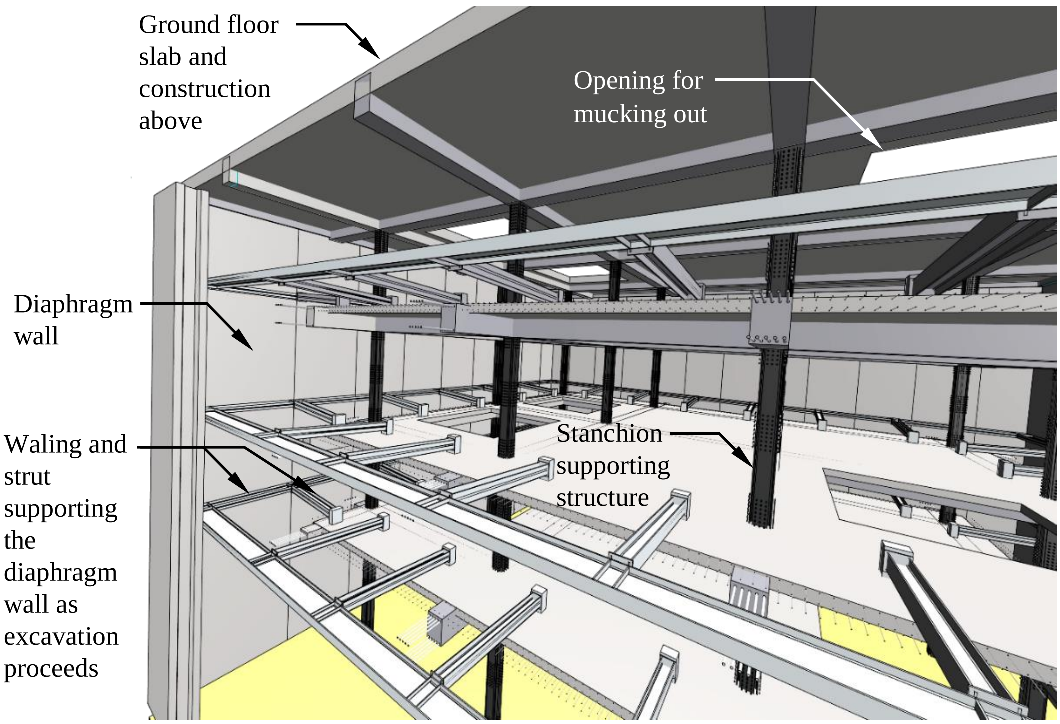

Figure 3.14 Top-down Construction

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

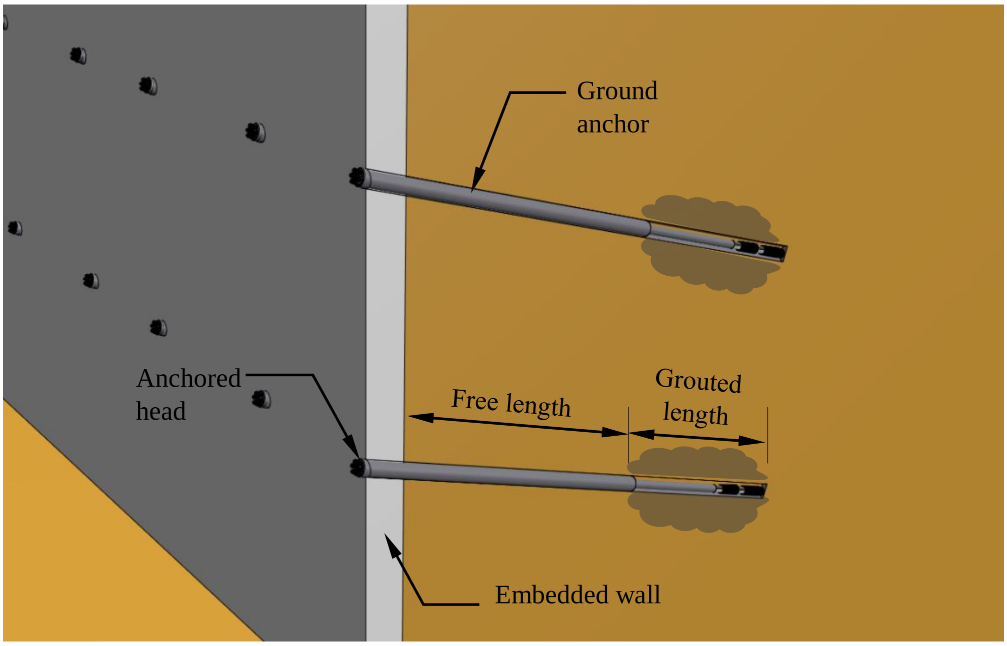

Figure 3.16 Tied-back Wall Support System Using Ground Anchors

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).

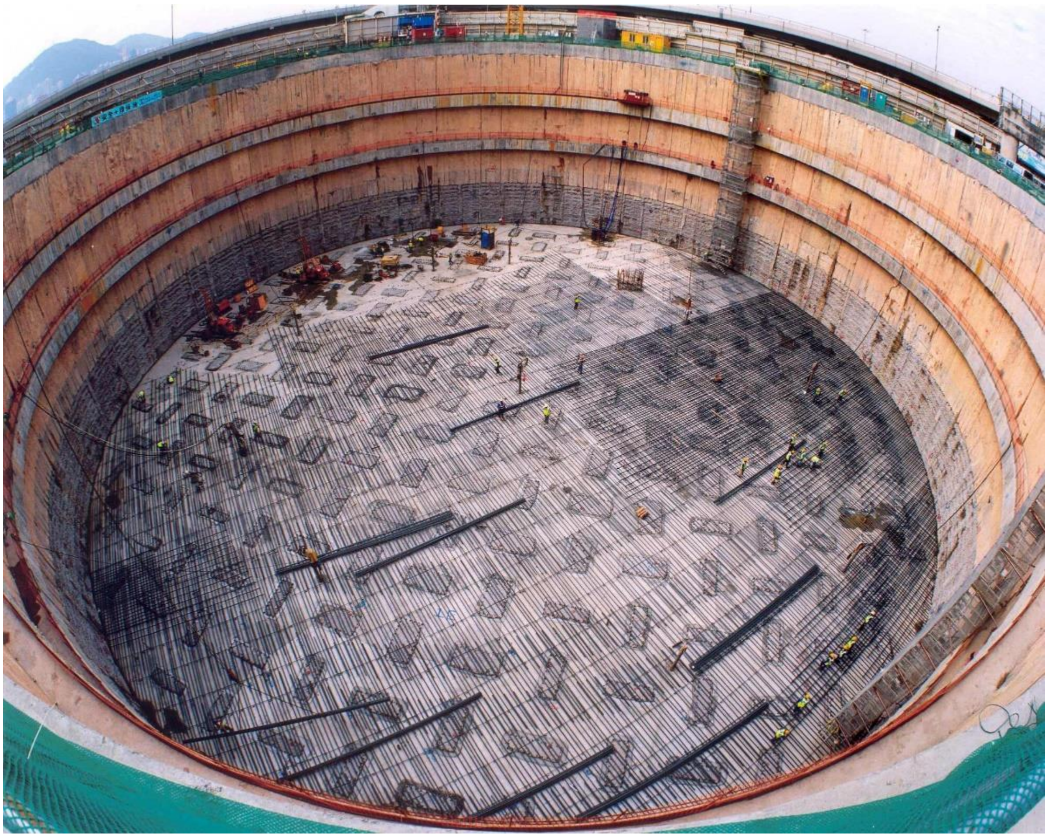

Figure 3.18 Circular Shaft for Basement Construction at International Commerce Centre