An Account of Deep Excavations using Steelworks in Hong Kong

James W.C. Sze & Albert N.L. Ho
Ove Arup & Partners Hong Kong Limited
james.sze@arup.com, albert.ho@arup.com

ABSTRACT
Basement and other forms of underground structures commonly formed part of the building and infrastructure developments in the built areas of Hong Kong. To facilitate the construction of these underground structures, lateral support works are usually required. Steel embedded walls and shoring systems are being the commonly adopted forms of lateral support works for these deep excavations. This Paper gives an overview of the commonly adopted steel embedded walls and steel shores in Hong Kong with an illustration by a few case histories.

1 INTRODUCTION
Deep excavation in Hong Kong is usually defined by an excavation depth exceeds 4.5m from the existing ground level. This is probably due to the fact that in many cases, such depth would have encountered the groundwater table in many places over the territory. With scarced land, building developments in Hong Kong usually accompanied by a basement for commercial use such as shopping arcade or carpark. Cut and cover tunnels or other forms of underground structures are also common in association with the infrastructure developments.

Temporary retaining structure is usually demanded to facilitate a bottom-up construction of these underground structures. In this regard, steel embedded walls and steel shores are usually adopted. This Paper gives an account of the commonly adopted steel embedded walls and steel shores in Hong Kong. The key features of these systems will be highlighted in illustrations of a few case histories of deep excavations.

2 COMMON TYPES OF STEEL RETAINING WALL FOR DEEP EXCAVATIONS
There are two main categories of embedded retaining walls to facilitate deep excavations in local practice -

i. Cast in-situ reinforced concrete walls - such as bored pile wall and diaphragm wall. These walls are usually used as a temporary retaining wall as well as forming permanent basement wall, with or without an internal dress-up block wall.

ii. Steel walls of fabricated sections - such as sheet pile wall, soldier pile wall and pipe pile wall. These walls are commonly adopted as temporary retaining structures to facilitate an excavation.

Two ways to construct a basement structure are named as bottom-up and top-down construction. In some cases, a bottom-up construction of the basement is more warranted, apart from the general cost-effectiveness consideration, with the following conditions -

i. to allow maximum flexibility of the final basement layout;

ii. to have better control on the quality of the permanent cast in-situ basement wall, in particular the waterproofing;

iii. to avoid the use of heavy duty piles to sustain the column load during top-down construction in case small diameter piles and hence multi-piles per column are found to be more cost-effective;

iv. to allow deep foundation to be constructed at the formation level to avoid abortive lengths.

For bottom-up construction, steel type embedded retaining walls with steel shores are generally more favorable over the cast in-situ RC structures, with the following reasons :

i. fast to install;

ii. relatively straight forward quality control procedure to achieve uniform properties;

iii. no curing time required;

iv. able to be extracted/dismantled and re-used afterwards;

v. easy and fast to dismantle.

2.1 Sheet pile wall
With the individual sheet pile sections interlocked side by side, sheet pile wall is very effective in terms of unit weight to stiffness/capacity ratios. The common sheet pile sections used in Hong Kong ranged from FSP-II to FSP-VL or equivalent. It provides reasonably good seepage cut-off performance and hence particularly suitable for developments with high groundwater table. An example of deep excavation using sheet pile wall is shown in Figure 1.

Commonly sheet pile sections are installed using vibratory or hammering techniques. With hammer driven method, typically it can be installed in stiff/dense soils with Standard Penetration Test (SPT) 'N' upto 60. These installation methods are very productive but inevitably will induce vibration and possible movement to the adjacent ground and structures. An overview during the installation process is shown in Figure 2.

In recent years, the reaction-based press-in methods, such as Giken, which generates less vibration, settlement and nuisance to surrounding areas has become more popular but generally limited to the use near ultra-sensitive structures.

Although steel sheet pile wall is most common type of temporary walls used in Hong Kong, the major shortcoming of it is on the installation technique which is difficult overcome underground obstructions such as corestones which are not common in Hong Kong. Pre-boring using auger or a down-the-hole hammer is usually carried out in such cases.
2.2 Pile Wall

Pipe pile wall utilizes closely spaced circular hollow sections (CHS) of typical sizes from 273mm to 610mm outer diameter. The thickness of the CHS ranges from 6mm to 20mm. In some cases, Universal Column (UC) sections are installed to enhance the combined bending stiffness and the capacity. The wall is usually accompanied by a grout curtain at the external side of the wall to provide seepage cut-off. An example of deep excavation using pipe pile wall is shown in Figure 3.

Although sometimes augering method can be adopted, the most common installation method for pipe piles is the overburden drilling method. An ODEX drill bit, Figure 4, is in fact a down-the-hole (DTH) hammer with expandable wings which can be extended sideway to form a hole slightly bigger than the steel casing. The casing has a ring shoe at the bottom hence the ODEX drillbit can bring the casing down without external means. The hammering action is powered by a high pressure driven piston. The drill rod rotates while hammering and the drill bit will crush the rock mass progressively. The rock chip/debris is also flushed out by the high pressure air. The use of ODEX drilling method can overcome corestone easily without change of equipment.

2.3 Soldier Pile wall

Soldier pile walls usually comprise UC sections installed in the pre-formed holes at a spacing of around 3 times the width of the UC. In competent soils, the spacing can be further increased. The typical widths of the sections ranged from 254mm to 358mm. The soil between the soldier piles upon exposed during excavation in lifts is retained by lagging in the form of steel channels or steel plates. Soldier pile is usually used in a site with relative low groundwater table. Figure 5 shows an overview of excavation face of a soldier pile wall.

In some cases, soldier piles are driven into the ground using hammer but more popular is to adopt the same installation technique as pipe piles, i.e. ODEX drilling, except that the temporary steel casing is extracted after installation of the UC into the hole and the void being filled up with low grade cement grout.
3 COMMON STEEL SHORING SYSTEMS

The most common steel shoring system in association with a bottom-up construction is probably cross-lot horizontal steel struts, as shown in Figure 6. The struts and walers are usually in the form of Universal Beam (UB) or Universal Column (UC) sections but sometimes tubular sections are adopted for the struts to minimize the requirement on the kingposts. Pre-fabricated jumbo sections with bolt connection have also become more popular which are particular suitable for regular shape of site or require high degree of preloads.

The sizes of shoring depend on the strength as well as stiffness requirements. It is not uncommon to pre-load the struts in order to control the wall deflection and hence the induced movements on adjacent ground and structures, as shown in Figure 7.

Short-struts are commonly adopted between the waler and temporary wall to facilitate the continuing basement wall construction before dismantling the main struts. To facilitate the mucking out in a limited site space, temporary steel platform is usually required. These are supported on the kingposts.

For large sites, inclined struts on partial completed basement structures might result in overall cost-effectiveness. Temporary anchors or tie-backs are less common in Hong Kong due to the land issue in built-up environment as well as difficulties in getting approval from the government authorities. An example of using temporary tie-back in sloping ground is shown in Figure 8.

4 CASE EXAMPLES

4.1 Development at Nos. 12-16 Tai Tam Road

The site is approximately 45m by 57m in area and bounded by Tai Tam Road and a natural rock slope dipping downwards to the sea. The new development comprised six blocks of 3-storey high residential buildings. The level difference between the final formation level and Tai Tam Road is about 13m.

The stratigraphy of the site was a thin layer of fill overlying 2 to 4m completely decomposed tuff and then bedrock. The bedrock was generally of moderately weak to strong Tuff and was about 5 to 8m below the existing road level. The
groundwater table was found to be low but a perched water of about 2m above the rockhead was assumed for the design purpose.

In view of the sloping topography and the excavation immediately adjoining to the existing road for the car ramp, a temporary soldier pile wall was proposed. The soldier pile wall with channel lagging supported by tie-back along roadside was adopted as the temporary retaining structure to facilitate the excavation and unobstructed construction of the permanent L-shaped RC retaining wall as well as to give a maximum working area for mucking up and the construction of the substructures.

The soldier pile wall consisted of Grade 43 254x254x73kg/m UC section installed at horizontal 750mm c/c spacing. The installation procedure was first by forming pre-bore hole using an ODEX drilling with temporary casing support in soil and followed by a down-the-hole hammer in bedrock to the designated level. The steel sections were lowered into the hole and extended by welding. The annulus was then backfilled with non-shrink cement grout and the temporary steel casing was extracted. One every third number of the soldier piles was extended to below the formation level in order to maintain the stability of the wall in case adverse rock joints persisted. The remaining soldier piles were stopped high with adequate factor of safety against kick-out upon exposing the rockhead. Channel lagging was installed for the top part of the wall retaining soil. The lagging was connected to the soldier piles by welds and the void behind the lagging was backfilled using lean concrete. An overview of the wall is shown in Figure 9 and the exposed rock face is shown in Figure 10.

The tie-backs consisted of 25mm & 50mm high yield steel reinforcement bar (Fy = 460MPa) installed in 100mm diameter drillhole at 20 degree inclination at 750mm and 1500mm respectively formed by percussive drilling using air as flushing medium. The hole is backfilled with non-shrinkage cement grout after installation of steel bar with plastic centralisers. This is a common construction technique for soil nail tie-backs in Hong Kong. No pre-load was applied to these tie-backs and hence they worked in passive manner. The tie-backs were installed in layers with excavation progressed in 2m lifts. A closer view of the tie-backs is shown in Figure 11.

During the excavation in rock, mapping of the rock joints was carried out in every 2m in order to assess the stability of the rockmass and stabilization measures involving rock dowel/tie-back can be implemented to deal with the potential unstable blocks. Hand held tools were used when the excavation is close to the soldier pile wall, as shown in Figure 12, to minimise the extent of overbreak which might undermine the wall and hence affecting the stability.

The maximum wall deflection recorded was less than 10mm which is much smaller than the designed values. This is largely due to the conservatism in the assumed water table as well as low stiffness of the material near the rockhead in the design calculation.

4.2 Ma Chau Spurline Eastern Approach Tunnel

To connect the 7.4km Lok Ma Chau (LMC) Spurline with the existing East Rail mainlines at the north of the existing Sheung Shui station, 700 m long cut and cover approach tunnels, up to 14 m deep and 20 m wide, were required to be constructed within a very narrow corridor between the East Rail mainlines and Dongjiang Watermains which is supplying potable water from mainland China to Hong Kong. A general view of the
The completed structure is shown in Figure 13 with the new downtrack tunnel covered by the new centre ballast track. A view of the excavation works in the proximity of the watermains is shown in Figure 14.

![Fig. 13 Completed approach tunnel in-between the existing railway track and trunk water mains](image)

![Fig. 14 Excavation for shallower track adjoining the watermains](image)

The ground conditions along the cut and cover tunnels comprised up to 5m of fill overlying 4 and 8m of alluvial deposits, which in turn overlies completely decomposed Volcanics. The rockhead was encountered between 19 and 35m below ground level. The ground water level was typically between 2 and 3 m below ground level.

Ground movements and disturbance due to works were of prime concern to the Railway Authority and to the Water Supplies Department (WSD) and a strict sequence of works, rigorous selection of plant and stringent monitoring controls were necessary to ensure that the live railway and the watermains were unaffected by the works.

Sheet pile wall of steel sections Grade 43 FSP-III & IV was adopted for the temporary retaining wall to facilitate the excavation. A maximum of 4 levels of struts were used with a maximum pre-load of 250kN/m applied to the lower 3 levels. The installation of the sheet piles was commenced using conventional vibratory techniques. Following the use of several types of high frequency hammers it was concluded that the vibration limits could not be complied with where driving was directly adjacent to the railway and the watermains. The reaction-based press-in method was therefore employed to install the sheet piles in the locations directly adjacent to the railway and the watermains, as shown in Figure 15.

![Fig. 15 Installation of sheet pile using press-in method](image)

Despite the clear safety advantages of this system ground conditions played an important role in the success of such technique. The sheet pile installation was complicated by the presence of a layer of rock and boulder fill below the railway tracks for which the press-in method could not penetrate through. Pre-drilling using ODEX drilling in advance of the sheet pile installation was required but caused some settlements to the tracks.

A photo showing the proximity of the excavation of the deeper tunnel to the existing track with the live train running is shown in Figure 16. Comprehensive monitoring was devised including an automated monitoring of the existing and diverted East Rail tracks at 4m centres, ground deformation monitoring on adjacent existing structures and utilities, piezometers on the adjacent ground and inclinometers along the wall. The maximum wall deflection measured was about 25mm. The cumulative track settlement recorded was around 45mm, out of which half of the value was caused by the pre-drilling for the sheet pile wall installation as mentioned above. Tamping of the ballasted tracks at the agreed triggering levels was required throughout the works. A full account of the construction works related to this part of the cut & cover tunnel is given by Storry et al. (2006).

4.3 Hong Kong Polytechnic University (HKPU) School of Hotel and Tourist Management

The site accommodated a former staff quarter of the University built in 70s at an isolated premise near the southern tip of the campus. It is bounded by roads along two sides, a playground to the north and the Fire Services Headquarters to the south. The site was proposed to be redeveloped into the School of Hotel...
and Tourist Management which has a 4-level basement to accommodate lecture theaters as well as carpark spaces. The maximum excavation depth is about 25m below the existing ground level.

The ground investigation boreholes revealed the ground was underlained by 6 to 7m sandy fill overlying up to 5m predominantly marine sand with clay lenses. Underneath varying thickness, up to 10m, of alluvium sand and clay there is a thin layer of Completely Decomposed Granite (CDG) overlying granitic bedrock. The bedrock level varied from -15mPD to -20mPD. The groundwater level ranged from +1.3mPD to +2.5mPD which is about 2 to 3m below the existing ground level. A typical cross-section of the basement with inferred geology is shown in Figure 18.

The foundations of the development consisted of footing/raft sitting on bedrock to resist the gravity loads and mini-piles to carry the tension load due to uplift exerted on the base of the basement structure. The mucking out of soil had also been complicated by the presence of existing small concrete piles within the majority area of the site. A bottom-up construction of the basement was hence proposed and adopted to achieve a shorter programme as well as allowing mini-piles to be constructed at the final formation level.

Pipe pile wall comprised Grade 50, 610mm outer diameter 12mm thick CHS at 800mm spacing was proposed for the temporary retaining wall. Since the depth of excavation extended 7m in average into bedrock, the pipe piles were terminated at the rockhead and UC section was used below the rockhead to achieve a faster construction rate. Grout pipes at 800mm horizontal spacing with tube-a-manchettes at 1m vertical intervals were installed behind the pipe pile wall to form a grout curtain along the site boundary.

The shoring adopted was cross-lot struts arranged in a way that maximum clearance is allowed to facilitate the installation of tension mini-piles foundation at the formation level, as shown in Figures 19 and 20. Maximum 6 layers of struts with a maximum pre-loading force of 500kN/m was adopted.

The predicted maximum wall deflection and associated ground settlement were 40mm and 25mm respectively. The maximum observed wall deflection was 25mm and the ground movement was around 12mm, as shown in Figure 21.
5 ISSUES

There are some key features and issues related to the steel embedded walls and shoring systems worth to note:-

5.1 Bending Stiffness

The steel embedded walls described in this paper are relatively flexible compared to the reinforced concrete walls, such as bored pile wall and diaphragm wall. The typical range is shown in Figure 22. For instance, the bending stiffness a 1.0m thick diaphragm wall is around $2 \times 10^6$ kNm/m. The flexibility of these walls warranted more closely spaced struts and the use of pre-loading on the struts in controlling the wall deflection and hence the induced ground movements beyond the excavation. For excavations in weak soils the vertical strut spacing can be as close as 1.5m and the constructability issues need to carefully considered and balanced in choosing an appropriate ELS system and excavation sequence.

![Typical range of bending stiffness of different temporary steel walls](image)

Fig. 22 Typical range of bending stiffness of different temporary steel walls

5.2 Ground Movement due to Wall Installation

In quantifying the amount of movement induced by an ELS design, it is not uncommon for a designer to disregard the ground movement during the wall installation process. The underlying reason could be such effect is difficult to quantify since there are many attributing factors, such as installation method and plant adopted, workmanship, variability of the ground.

Inevitably the installation of walls might cause significant ground movement and which might attribute a significant part of the total effect of the excavation works. For instance, in LMC Spurline case, the pre-drilling through the cobbles layer adjacent to the railway track to facilitate installation of the press-in sheet piles had caused a cumulative settlement of 20mm which is significant compared to the settlement induced by the bulk excavation in the later stage.

In the HKPU case, the ground settlement induced during the wall installation is around 5mm and equivalent to 40% of the total settlement induced by the entire excavation works, as shown in Figure 21. The curtain grouting behind the wall is usually considered to be beneficial since it fills up the potential over-breaks outside the wall during the ODEX drilling but it has no effect in rectifying the immediate settlement that had already occurred.

It is considered more case histories shall be collected to form a database which might indicate a typical range of settlement induced by each type of walls during installation.

5.3 Structural Design of the Steelworks

In case of cross-lot struts are adopted, the member sizes of these struts are usually dictated by the required stiffness in limiting the wall deflection instead of the strength requirement. The similar argument also applies to the steel walls for which the required bending stiffness usually played an important role.

These are the reasons for the commonly adopted steel grades for the steel works in ELS design being Grade 43 and 50 (now refer to S275 and S355 respectively). In short spanned excavation where the member size is usually determined by strength, it might be more cost-effective to adopt the readily available steel bearing H-pile sections which has a high yield strength ($460$ grade).

The structural design of the steel works follows the Code of Practice for Structural Use of Steel (BD, 2005). A load factor of 1.4 is adopted in deriving the forces in ultimate limit state from working load for these temporary works.

5.4 Pre-loading of Struts

As mentioned earlier, pre-loading of steel struts is usually carried out with the primary aim to limit the wall deflection during a deep excavation. An added advantage by doing this is to achieve a more balanced moment envelope on both faces of the wall and hence might result in a more economic design of the steel embedded walls.

When estimating the required pre-loading force, a higher load is usually resulted due to the conservatism in the design assumptions in the model, such as conservative soil parameters, worst groundwater regime and allowance for high construction surcharge. Usually these assumptions are found to be conservative during the actual site works and the forces applied to strut could be too high. Figure 23 illustrates the wall deflection profile of the HKPU site prior to and after preloading of the second layer of strut. It can be seen that the wall near the strut level was evenly pushed back into the weak soil behind the wall. This has caused the wall top moved into the site further and incurred more settlement at the pavement near the wall where laid most of the underground utilities around site.

![Effect of preloading on wall deflection](image)

Fig. 23 Effect of preloading on wall deflection

The large movement of the wall bending outwards during the preload may also pose problem to the grout curtain since cracks on the earth side could be resulted and affecting the seepage cut-off performance. It is considered that observational
 approch shall be adopted with lower bound and upper bound pre-loading forces specified on the construction drawing. The force application shall depend on the deflection profile of the wall and recorded movements outside the site at the corresponding stage.

5.5 Connection Details of Shoring

Poorly detailed shoring and poor workmanship during installation of struts had been accounted for many cases of excessive displacement and different scales of collapse in deep excavations in Hong Kong and elsewhere in the nearby regions (GEO, 1999). In recent years, emphasis has been put onto the detailing of the steel shoring during the design. Figure 24 illustrates the typical details of the stiffeners in the struts and walers in order to prevent local buckling and shear failure of these members in order to achieve a more robust shoring system.

Apart from the modular strut systems, weld connections are extensively used to fabricate the shoring system. The quality of the welds is of prime importance and as part of the quality assurance system, only qualified welders can carry out the works. Qualified welders usually refer to those who possess the Trade Test Certificate for Steel Welder issued by the Construction Workers Registration Authority (CITA) and have been registered under the specific designated trade. This is not only a test on workmanship but also on the welding procedure.

5.6 Temporary steel embedded wall as permanent structure

Although they are termed as temporary, in many cases, especially in urban areas, these steel walls are left behind with top 2m trimmed since the disturbance during extraction could be quite severe. In view of these, in some projects these walls are considered and designed as a permanent seepage cut-off system with under-drainage system to be devised under the basement in order to relieve the uplift load acting on the basement slab and minimise the requirement on the use of tension piles.

5.7 Back-analysis of deep excavations using steel embedded walls

The sectional properties of the steel embedded walls are much more consistent and uniform than that of the cast in-situ RC walls. This eliminates the uncertainty in bending stiffness of the wall and provided a good basis for back-analysis for other design assumptions, such as the soil stiffness.

The soil stiffness in Hong Kong is usually correlated to SPT ‘N’ values for easy assessment. The back-analysis of wall deflection of LMC eastern approach tunnels was conducted by Pan et al. (2006). One of the back-analysed profiles is shown in Figure 25. The elastic moduli were found to be 1500x‘N’ (kPa) for fill and 4000x‘N’ (kPa) for coarse alluvium and CDV/CDG. A similar exercise has been done for the HKPU site, one of the back-analysed wall deflection profiles is shown in Figure 26. The actual groundwater level and loads in the struts as measured by strain gauges were incorporated and the best fit soil moduli were again found to be 1500x‘N’ (kPa) for fill and marine deposit, and 4000x‘N’ (kPa) for CDG.

When compared to other deep excavation case histories elsewhere in Hong Kong using diaphragm walls, such as those presented in Chan (2003) and Pan et al. (2001), these back-analysed stiffness correlation factors are similar for superficial deposits and slightly on the high side for CDV/CDG. It is considered these values are credible however the application of these factors in the design of excavation shall be carefully examined since the built-in allowance for other factors
attributing settlement, such as wall installation might be taken away and result in aggressive design.

6 CONCLUSION

The commonly used steel type embedded walls and shoring systems for deep excavations in Hong Kong with three case histories were described. Some key issues and related features were discussed and commented with an aim to arouse the interest of the industrial practitioners elsewhere in the design and construction practices applied in Hong Kong.

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REFERENCES

Buildings Department. 2005. Code of Practice for Structural Use of Steel. 357pp


