

# Chapter 2

## Literature Review

### 2.1 Introduction

For deep excavations in soft clay with an underlying aquifer, there are three major concerns: excavation-induced ground deformation, soil-structure interaction and base instability due to hydraulic uplift. In this chapter, previous studies on these three aspects are reviewed. More importantly, major short-comings of previous works are indentified.

### 2.2 Ground Deformation Due to Excavation

#### 2.2.1 Field Investigation

##### (1) Effect of construction of diaphragm wall on ground deformation

One major reason for ground deformations adjacent to an excavation is due to construction of the wall. Poor construction techniques for diaphragm walling were shown to result in large ground movements adjacent to the wall.

Based on several case histories, O'Rourke (1981) reported that total settlements due to construction of diaphragm wall are in the range of 50 to 70 % of the total settlements observed during the entire process of excavation. It was further detailed that during trench excavation, settlements in soft clay and sand occurred due to stress release. During the subsequent placement of concrete, "soil squeeze" (soil moved away from the trench) may occur in soft clay, depending on the amount of time the trench remains open before concreting.

O'Rourke (1981) found that the 18 observed surface settlements due to diaphragm walling were related to the time period when trenches remained unsupported. It is measured that ground surface settlement was approximately 0.2 % of the wall depth. The measured ground surface settlement was smaller than that

estimated. This is probably due to the reason that excavated trench was supported laterally by slurry pressure prior to placement of concrete.

## (2) Typical deformation profiles due to excavation

Based on field observation from several case histories, Clough and O'Rourke (1990) summarised typical profiles for lateral wall movement and ground surface settlement solely due to excavation. It can be seen that during the first excavation, prior to installation of the first prop, the wall deforms as a cantilever. Correspondingly, settlement trough behind the wall may be represented by a triangular shape. When the excavation reaches deeper elevations, lateral wall movement in the propped region is restrained horizontally. Deformation profile of the wall changes from cantilever to deep-seated type of deflection profile, with the maximum lateral wall displacement located near the formation level of the excavation. In the meantime, settlement trough behind the wall changes from a triangular shape to a trapezoidal shape.

The typical deformation profiles summarised by Clough and O'Rourke (1990) are confirmed by field observations reported by O'Rourke (1981) and Finno et al. (1989).

## (3) Empirical method for predicting ground surface settlement

The first rational approach for estimating ground surface settlement adjacent to excavations was proposed by Peck (1969), based on filed data measured adjacent to temporary braced sheet pile and soldier pile walls. Based on soil conditions and workmanship employed when constructing the wall, filed data collected by Peck (1969) are divided into three categories. Category I includes excavations in sand, stiff clay, and soft clay of small thickness. Category II includes excavations in very soft to soft clay extending a small distance below the bottom of the excavation or with a stability number,  $N_b$ , less than 6 or 7. Category III includes excavations in very soft to soft clay that extend to a significant depth below the bottom of the excavation, and with stability numbers greater than the critical stability number for basal heave. For Category I soil, the maximum surface settlement is limited to 1 % of the final excavation depth. The maximum surface settlement and influence zone of settlement trough behind the wall increase from excavations in Category I to those in Categories II and III.

Clough and O'Rourke (1990) found that by plotting normalised ground settlement (by the maximum value) against normalised distance behind the wall (by final excavation depth  $H$ ), a relatively well-defined grouping of settlement data can be obtained. Separate profiles were proposed for sand, stiff to very hard clay, and soft to medium clay, respectively. With given maximum ground surface settlement, the proposed charts can be used to estimate the actual surface settlement. The figure shows that influence zone of settlement trough behind the wall is  $3H$  and  $2H$  for excavations in stiff to very hard clay and in sands and soft to medium clay, respectively. It can also be seen that settlement trough in soft clay was bounded by a trapezoidal envelope. Inside the envelope, two zones of settlement can be identified. The maximum settlement is located within the zone where  $0 \leq d/H \leq 0.75$

( $d$  denotes distance behind the wall). Within the zone when  $0.75 < d/H \leq 2.0$ , ground surface settlement decreases from maximum to negligible values. It should be noted that the design diagrams proposed by Clough and O'Rourke (1990) are only valid for estimation of ground settlement due to main excavation. Settlement induced by other construction activities such as diaphragm walling and dewatering is not taken into account.

Based on field observations, Hsieh and Ou (1998) suggested that there are two types of settlement profiles due to excavations: (i) spandrel shape, in which maximum settlement occurs very close to the wall; (ii) concave shape, in which maximum settlement occurs at a certain distance away from the wall. Spandrel shape of settlement profile was found where a large amount of cantilever type of wall deflection occurs at the first stage of excavation while further lateral wall displacement due to subsequent excavation is relatively small. Concave shape of settlement profile was found where after the initial stages of excavation, additional cantilever wall deflection is restrained by installation of support as the excavation proceeds to deeper elevations. The concave shape of settlement profile is corresponding to deep-seated deformation profile of the wall. It is pointed out by Hsieh and Ou (1998) that distance from the wall to the location where the maximum ground surface settlement occurred was approximately equal to half the excavation depth. Assuming the maximum lateral wall deflection occurs near the formation level of the excavation, the maximum ground surface settlement occurs at about half the final excavation depth behind the wall. Based on field data, settlement near the wall was estimated as half the maximum ground surface settlement. The primary influence zone of settlement trough was approximately two times of excavation depths behind the wall. The design diagram also shows that settlement becomes negligible at a distance of four excavation depths behind the wall ( $4H$ ). For simplicity, a linear relationship was assumed between each turning point.

#### (4) Database based on large number of excavations worldwide

Long (2001) collected measured data from 296 case histories. The database shows that for excavations in stiff clay, the normalised maximum lateral wall displacement ( $\delta_{h-\max}/H_e$ ,  $H_e$  denotes final excavation depth) and normalised maximum ground surface settlement ( $\delta_{v-\max}/H_e$ ) are in the range of 0.05 0.25 % and 0 0.2 %, respectively. For excavations in soft clay,  $\delta_{h-\max}/H_e$  may be up to 3.2 %. Moreover, dependency of  $\delta_{h-\max}$  on system stiffness proposed by Clough et al. (1989) and flexibility number are examined. It is concluded that the deformations of deep excavations in non-cohesive soil as well as in stiff clay are independent of the stiffness of the retaining system (i.e., stiffness of wall and props, as well as prop spacing). For excavations in soft clay, lateral wall displacements of deep excavations appear to have some correlation to stiffness terms proposed by Clough et al. (1989), but with relatively large scatters. The stiffness of retaining system only significantly affects deformations of excavation in soft clays with a low factor of safety against base heave.

Moormann (2004) studied lateral wall displacement and ground surface settlement of 530 case histories in soft soil ( $c_u < 75$  kPa). The database shows that  $\delta_{h-\max}/H_e$  lies in a range between 0.5 and 1.0 %, with an average value of about 0.87. The locations of  $\delta_{h-\max}$  occur at  $0.5H_e$  to  $1.0H_e$  below the ground surface.  $\delta_{v-\max}/H_e$  ranges from 0.1 to 10 %, with an average value of about 1.1 %. The maximum ground surface settlements are located within  $0.5H_e$  behind the wall, with some exceptional cases in soft clay (i.e., up to  $2.0H_e$ ). The  $\delta_{v-\max}/H_e$  ratios roughly range from 0.5 to 1.0. Similar to conclusion made by Long (2001), it is found that deformations of excavation in soft clay seems to be largely independent of the system stiffness of the retaining system. Instead, deformations are mostly related to ground conditions.

Wang et al. (2010) summarised an extensive database of deformations of 300 deep excavations in Shanghai soft soils. According to construction methods and types of retaining systems, the 300 case histories are divided into five categories, namely top-down method, bottom-up method, sheet pile walls, compound soil nail walls, and deep soil mixing walls. Average  $\delta_{h-\max}/H_e$  of these five categories is 0.27, 0.40, 1.5, 0.55 and 0.91 %, respectively. Average  $\delta_{h-\max}/H_e$  of the 300 case histories is about 0.42 %. Influence zone of settlement trough behind the wall varies between  $1.5H_e$  to  $3.5H_e$ . The  $\delta_{v-\max}/\delta_{h-\max}$  ratios are in the range from 0.4 to 2.0, with a mean of 0.9. Dependency of measured deformations on system stiffness (Clough et al. 1989) and factor of safety against basal heave are examined. It appears that the maximum lateral wall displacement is slightly relevant to the two factors.

### 2.2.2 Numerical Analysis

#### (1) Influence of system stiffness and ground conditions

Mana and Clough (1981) carried out numerical parametric study to investigate effects of wall stiffness, prop spacing, prop stiffness, prestress and elastic soil stiffness on ground deformations due to excavation. It was revealed that ground deformations decreases with increased wall bending stiffness or decreased prop spacing. This effect is more significant when factor of safety against basal heave is relatively low. Ground deformations decrease with increased prop stiffness, with a decreasing rate at high stiffness. Moreover, ground deformations are also significantly affected by soil modulus. Higher modulus leads to smaller movement.

Based on results from numerical parametric study (by finite element analysis), Clough et al. (1989) developed a design chart. The chart correlates lateral wall displacement of excavation in soft to medium clay to system stiffness and factor of safety against basal heave. The factor of safety against basal heave used in the figure is derived by Terzaghi (1943).

For wide excavations ( $B/H_e > 1$ ), factor of safety against basal heave is given as:

$$FS = c_{ub}N_c / (H(\gamma - c_{uu}/(0.7B))) \quad (2.1)$$

For wide excavations where there is a strong stratum near the base of the excavation, the factor of safety is given as:

$$FS = c_{ub}N_c / (H(\gamma - c_{uu}/D)) \quad (2.2)$$

where  $c_{ub}$  and  $c_{uu}$  are undrained shear strength below and above formation level of excavation, respectively;  $H_e$  denotes final excavation depth;  $B$  is width of excavation;  $\gamma$  is bulk density of clay;  $D$  is distance from the bottom of the excavation to a relatively hard stratum;  $N_c$  represents stability factor, which is equal to:

$$N_c = \gamma H_e / c_u \quad (2.3)$$

It should be noted that this design chart does not take effect of diaphragm walling into account. The chart illustrates that for a given system stiffness, a lower factor of safety against basal heave results in a larger lateral wall displacement after excavation. For excavations in soft clays, where the factor of safety against basal heave is low, a larger stiffness of the support system results in a smaller lateral wall displacement.

Clough and O'Rourke (1990) conducted a series of finite element parametric studies on excavations in stiff clay. The computed results show that parameters such as wall stiffness and prop spacing have only a small influence on deformations around excavations in stiff clay. This is because the model soils are stiff enough to minimise the need for stiff retaining systems. It was found that soil modulus and coefficient of lateral earth pressure have a more significant impact on the ground movements, compared to stiffness of retaining systems. Clough and O'Rourke (1990) also pointed out that base instability is usually not concern for excavations in stiff clays.

Addenbrooke (2000) proposed a new term, displacement flexibility,  $\Delta = h^5/(EI)$ , to quantify overall stiffness of a retaining system. Dependency of lateral wall displacement on the displacement flexibility was confirmed by computed results from a series of elastic-perfectly plastic finite element analyses. Correlation of the displacement flexibility to lateral wall displacement was checked by extensive case histories worldwide (Long 2001; Moormann 2004). Both suggest that measured maximum lateral wall displacement appears to be independent of the displacement flexibility.

## (2) Influence of excavation geometry

Mana and Clough (1981) conducted a two-dimensional numerical parametric study to investigate effects of excavation geometry such as excavation width and depth of the underlying firm layer on ground deformations due to excavation. It was

revealed that ground deformations increase with excavation width and depth to an underlying firm layer.

Hashash and Whittle (1996) carried out a series of two-dimensional numerical parametric studies to study effects of wall embedment depth and prop spacing on ground deformations due to multi-propped excavations. Constitutive model (MIT-E3) adopted in the numerical analyses is capable of considering anisotropic stress-strain relationship, stress path dependency and strain dependency of clay. Computed results show that wall length has a minimal effect on the pre-failure deformation for excavations in deep layers of clay, but does have a major effect on the location of failure within the soil. Use of very depth wall can improve base stability. However, large bending moment can be resulted and may cause flexure failure of the retaining wall. Not only is basal affected by final excavation depth, but also influenced by vertical prop spacing. Larger vertical prop spacing can result in additional basal heave.

Jen (1998) carried out numerical parametric studies to study effects of excavation geometry, retaining system and stress history of clay on ground deformations due to excavation. The computed results reveal that distribution of ground deformation is significantly affected by depth of hard stratum below soft clay. While magnitude of ground deformation is governed by excavation width, excavation depth and stress history of clay. Based on the computed settlement troughs behind the wall, new design charts are proposed to correlate ground settlement to excavation depth, wall length, depth of hard stratum and soil profile.

### ***2.2.3 Analytical Solution***

Osman and Bolton (2006) proposed an analytical solution for estimating ground deformation around excavation in ‘undrained’ clay. The basic idea of this solution follows early study on using ‘mobilised strength’ to predict ground strain and hence displacement (Bolton and Powrie, 1988). A simplified deformation mechanism of excavation was assumed in the mobilised strength design (MSD) method. Ground deformations are linked to stress via stress-strain data from soil tests on undisturbed samples taken from representative elements. This approach has the advantage that one can use a single stress-strain curve from a single soil test, together with a simple hand calculation, to estimate both stability and soil deformation. However, the solution ignored structural energy stored in the retaining system. Ground deformation due to dissipation of excess pore water pressure during and after excavation is not taken into account. Moreover, the solution considers the total energy terms starting from the beginning of excavation. Therefore, the progressive accumulation of mobilised shear strain at different excavation depths is not considered.

Lam (2010) made further improvement of the MSD method developed by Osman and Bolton (2004), using observed ground deformation mechanisms from a series of centrifuge tests of multi-propped excavations in soft clay. In the improved MSD method, structural energy stored in the retaining system is considered. Progressive accumulation of mobilised shear strain at different excavation depths is also taken into account. Ground deformations calculated by the improved MSD method are compared with those computed by an advanced MIT-E3 model. In general, the predictions fell within 30 % of the finite element computed results.

### ***2.2.4 Centrifuge Modelling***

Powrie and Daily (2002) reported eight centrifuge model tests on berm-supported embedded retaining walls in over-consolidated clay. Variables between centrifuge tests are depth of wall embedment, berm volume and groundwater level. The test results reveal that for a wall deep enough not to collapse, increasing the size of the berm will result in a more significant reduction in wall and soil movements than increasing the depth of embedment of a wall supported by a smaller berm. An increase in the depth of embedment of a wall of given stiffness supported by a berm of a given size will lead to an increase in wall bending moments, but only a small reduction in wall and soil movements, especially in the short term. Increasing the depth of embedment of a wall supported by a berm of a given geometry is of more benefit if prolonged support is required or perhaps where the time taken to excavate is long. However, even in the long term, the percentage decrease in movements is likely to be rather less than the percentage increase in wall bending moments. The initial groundwater regime, which influences the undrained shear strength and stiffness of the soil, and the timescale for excavation in relation to the timescale for excess pore water pressure dissipation, both have a significant effect on the performance of the wall during and shortly after excavation. Also, the influence of a berm in limiting movements and possibly preventing collapse becomes more significant as drainage occurs within the soil and long-term pore water pressure equilibrium is approached.

Lam (2010) reports five centrifuge tests simulating multi-propped excavation in soft clay. The major objective of the tests is to study ground deformation mechanism due to excavation in clay with varied system stiffness and depth of hard stratum below the soft clay layer. It is observed that incremental wall deformation profiles generally followed O'Rourke's (1993) cosine bulge equation. New deformation mechanisms were postulated with respect to the wall toe fixity condition and excavation geometry. Settlement profiles in a shallow clay bed show a tapering-off trend as the extent of the deformation mechanism is limited by the shallow depth of soft layer.

## 2.3 Soil-Structure Interaction of Multi-propped Excavation

### 2.3.1 Field Investigation

#### (1) Excavation in stiff clay

Tedd et al. (1984) reported a multi-propped excavation in stiff clay at Bell Common. A typical panel was heavily instrumented, for measurements of lateral earth pressure, pore water pressure, lateral wall movement, ground surface settlement and roof loading have been during the construction of retaining wall (secant pile wall) and the subsequent main excavation. It is found that the ground movements occurred during the construction of the secant pile wall itself were surprisingly large and formed a significant proportion of the total movements that were monitored during the entire excavation period. Both construction of wall and the subsequent main excavation caused lateral wall movement into the excavation. Based on measured pore water pressure and lateral total earth pressure adjacent to the excavation, effective stress paths were deduced. During the main excavation, earth pressure behind the wall within propped region moved towards and approached Rankine's active failure boundary. In front of the wall and near the formation level, soil approached passive state at the end of excavation. The wall is very stiff and its movement by the end of construction was mainly a rotation about its toe. Bending moments in the wall were small.

Ng (1998) reported field measurement of a 10 m deep excavation in over-consolidated stiff-fissured gault clay in Lion Yard, Cambridge. The field monitoring programme includes lateral total earth pressure and pore water pressure on both sides of the wall, prop load, lateral wall displacement, ground settlement behind the wall and basal heave inside excavation. The field measurement reveals that due to construction of diaphragm wall, a significant reduction of lateral total earth pressure at the soil-wall interface is resulted. The piling operation inside the site caused a negligible reduction of lateral total earth pressure. Due to low lateral stress in the ground before the main excavation, relatively low prop load and lateral wall displacement are resulted during excavation. The reduction of lateral earth pressure due to excavation was substantially less than that induced by diaphragm walling.

Based on measured total earth pressure and pore water pressure at soil-wall interface, Ng (1999) interpreted effective stress path (in  $h$ - $h'$  space) of the case history documented previously (Ng 1998). It was found that immediately after the installation of the instrumented panel, there were substantial reductions in horizontal effective stress at the soil-wall interface at all seven measurement points. During the main excavation, vertical effective stress behind the wall increased significantly with little change of horizontal effective stress. This is as a result of reducing pore pressures accompanied by a small increase in horizontal effective stress. The states of stress behind the wall at the soil-wall interface reached, or were

close to, the assumed active condition. In front of the wall, all three stress paths show an increase in horizontal effective stress with a decrease in vertical effective stress during main excavation. At the end of excavation, soil element located near the formation level appears to have reached passive failure. In contrast, two lower soil elements show a continuation of the same stress paths but do not appear to have reached passive failure.

Richards et al. (2007) presents measured changes of pore water pressure and lateral earth pressure on both sides of the wall during an excavation in stiff clay. The measured results show that during excavation, pore water pressure and lateral earth pressure in the soil on both sides of the wall reduced significantly. As expected, the reductions in lateral earth pressure and pore pressure decreased with increasing distance from the wall. In front of the wall, the reduction in lateral stress due to the removal of overburden was a more dominant effect than the increase in lateral stress due to inward movement of wall.

## (2) Excavation in soft clay

Ou et al. (1998) presented field performance of an excavation constructed by top-down method in Taipei soft clay. The field measurement include lateral total earth pressure and pore water pressure on both sides of the wall, prop load, lateral wall displacement, ground settlement behind the wall and basal heave inside excavation. Measured results show that prop load corresponds to Peck's apparent earth pressure diaphragm. The lateral ground movement located at 2 m away from the wall was close to that of the wall. The maximum lateral wall displacement is about 0.51 to 0.57 % of final excavation depth ( $H_e$ ), located near the formation level of the excavation. Lateral earth pressure behind the wall within propped region decreased first and then increased to at-rest earth pressure, due to 'soil arching'. Below propped region behind the wall, lateral earth pressure decreased to less than Rankine's active earth pressure after excavation. The authors explained this as a result of ignorance of soil-wall friction in Rankine's theory. Major influence zone of settlement trough behind the wall is about  $2.5H_e$ , with the maximum ground settlement located at  $0.63H_e$  to  $0.78H_e$  behind the wall.

### 2.3.2 Numerical Analysis

Potts and Fourie (1984) reported finite element analyses (FEA) simulating single propped excavations. An elasto-plastic model with Mohr-Coulomb failure criteria was adopted in the FEA. Objective of the study is to investigate the effects of construction methods (i.e., excavation and backfilling) and of in situ at rest lateral earth pressure coefficient  $K_o$  on soil-structure interaction. It is found that for backfilled walls,  $K_o$  of soil has only a small influence on wall deformation. On the contrary,  $K_o$  has a significant effect on wall deformation for excavated walls. For backfilled walls and excavated walls in a low  $K_o$  soil, the simple design calculations underestimate values of prop forces and bending moments. For excavated walls in

high  $K_o$  soils, prop forces and bending moments greatly exceed those calculated using the simple limit equilibrium approach currently in use. Moreover, relatively large soil movements are induced. The behaviour of excavated walls in a high  $K_o$  soil is dominated by the excavation-induced vertical unloading. By installing more props, horizontal movements of the wall and soil can be reduced. But this has a much smaller effect on basal heave. Increasing the embedment of excavated walls in high  $K_o$  soils does not reduce the prop force or bending moments, even though the factor of safety is increased. For excavated walls in high  $K_o$  soils, soil in front of the wall reached passive state at small excavation depths. The lateral earth pressures behind the wall differ substantially from the classic active distribution. However, for backfilled walls and excavated walls in low  $K_o$  soils, lateral earth pressures shows agreement with the classical distributions. Soil behind the wall reached active failure at small excavation depths, prior to passive failure of soil in front of the wall.

Hashash and Whittle (2002) studied effective stress paths around a multi-propped excavation, by adopt MIT-E3 soil model in a finite element program. The adopted MIT-E3 model can capture strain dependency, stress path dependency and strength anisotropy of soil. It is found that soil in front of the wall (1E) follows typical path of plane strain passive mode of shearing. For soil element behind the wall within propped region, the stress paths move towards active failure line first and then experiences a stress reversal, shifting to passive failure line (soil arching). The arching mechanism behind the wall is further interpreted. It is revealed that the major principal stresses are directed toward the lowest level of strut, while an underlying compressive arch transfers loads onto the embedded section of the wall. After installation of the lowest strut, a deeper arching mechanism forms due to the next stage of excavation. The lowest props therefore carry the majority of the supportive earth pressure removal in the subsequent stages of excavation.

### 2.3.3 *Centrifuge Modelling*

Richards and Powrie (1998) reported a series of centrifuge tests of twin-propped excavations in stiff clay. The tests aim at studying effects of the groundwater regime, initial lateral earth pressure coefficient, embedment depth of wall and propping sequence on soil-structure interaction associated with multi-propped excavations. Locations of two levels of prop are at the crest of the wall and at the formation level of the excavation, respectively. The physical investigation reveals that an increase in embedment depth of wall can lead to an increase in wall bending moment and a reduction in bottom prop load. Increase of embedment depth of wall without also increasing the wall stiffness can not effectively limit excavation induced ground deformation. The maximum bending moments and prop loads generally increase with the pre-excavation lateral earth pressure coefficient above the excavated surface. By lowering the groundwater behind the retaining wall, bending moments of the wall and prop loads reduce significantly. The bending

moments and top prop loads may be greater in the short term, soon after excavation, than in the long term after dissipation of excess pore water pressure.

Lam (2010) reported a series of centrifuge tests simulating multi-propped excavation in lightly over-consolidated clay. Variables between tests are stiffness of wall and props, and distance of excavation to hard stratum. Based on the centrifuge tests, it is found that the empirical estimation by Peck's envelope underestimates apparent earth pressure for a stiff support system by 30 % in relation to the build-up of a vertical arching mechanism. Reducing the bending stiffness of the wall or the axial stiffness of the props caused a reduction in apparent earth pressures. Only cases of excavation with a flexible support system agree reasonably well with Peck's recommendations. By assuming that the total vertical stress is dominated by the overburden weight of the soil, effective stress paths were deduced for soil elements at different locations. Effective stress paths of soil elements in front of the wall move towards passive failure line, as expected. While effective stress paths of soil behind the wall within propped region move towards active failure line first, then shift to move towards passive failure line due to increased horizontal effective stress by soil arching. The rate of mobilisation of shear strength is insignificant for soil elements around excavations with a rigid supporting system. For excavations with flexible retaining system, the rate of mobilization of a soil element at mid depth of the wall is more significant than for shallower and deeper depths.

## **2.4 Excavation Base Instability Due to Hydraulic Uplift**

### ***2.4.1 Field Investigation***

Milligan and Lo (1970) summarises eight case histories in Canada related to base instability caused by hydraulic uplift. Based on the reported field observations, base instability associated with hydraulic uplift is accompanied by excessive basal heave, inward movement of the wall and hence ground surface settlement behind the wall. If the large basal heave is not effectively controlled, cracks can be developed inside excavation and create preferential flow path. Following the path, water in the aquifer would flood into the excavation. In some cases, artesian pressure in the aquifer is released by dewatering. But the dewatering in turn induced excessive ground settlements. The field observation seems to infer that undrained shear strength of clay along the soil-wall interface may help to resist hydraulic uplift to some extent. If the retaining wall was penetrated through the clay layer, base instability may occur, due to greatly reduced undrained shear strength along the soil-wall interface. From observations on a limited number of case histories summarised in the study, it has been proposed that it is possible to dig a narrow trench or excavation in undisturbed intact clay significantly below a depth limited by the ratio of  $t/h = 0.5$  (where  $t$  is the distance from the bottom of the excavation to the top of a water bearing stratum and  $h$  is the critical head at the top of the water

bearing stratum). It is suggested that, with the sides of an excavation adequately supported, sheeting or foundation piles should not be driven below the base of the excavation, unless provision is made to relieve uplift water pressure acting on the impervious layer or the pore pressure set up by pile driving. In spite of the summarised observations, initiation and failure mechanism of each case history still remains unknown.

Haydon and Hobbs (1977) reported uplift of excavation base of a 9 m deep excavation for a large nuclear reactor underlain by alternating beds of Lower Lias limestone and clay shale in UK. In the latter case water was found to flow through open joints in the limestone under hydrostatic pressure relative to a water table near ground level. An attempt made during excavation to prevent hydraulic uplift by drilling relief wells was only partly successful due to the difficulty of intersecting the water-bearing joints.

Moore and Longworth (1979) reported a local hydraulic uplift failure in the base of a 29 m deep, large brick pit excavation in Oxford Clay. The failure was preceded by at least a 150 mm rise of the pit base as water accumulated under pressure beneath impermeable clay surface layers. Hydrogeological studies have indicated that the source of water was a thin underlying bed of limestone. Rupture of the capping clay finally resulted in the rapid release of about 7000 m<sup>3</sup> of water which flooded part of the pit and caused an immediate settlement of up to 100 mm of the pit base.

Clough and Reed (1984) presented a braced excavation in very soft clay in California. Base instability problem due to hydraulic uplift was reported to be involved in the excavation. However, details information was not provided.

Davies (1984) reported one excavation in clayey decomposed granite in Singapore. It was observed that when the excavation reached about 6.5 m below ground surface, the southern half of the excavation base suddenly 'heaved', accompanied by a rapid water flow into the excavation. Increase of water content of the clay inside excavation resulted in a reduction of undrained shear strength. Accordingly, construction traffic sunk into the very soft clay. Subsequent investigations showed that there is an aquifer located underneath the excavation. Prior to the excavation, designers were not aware of the presence of the aquifer and hence no base stability calculation against hydraulic uplift was conducted. Ramaswamy (1979) reported a similar case history in Singapore where damage to a raft occurred due to high artesian pressure in permeable laminations within stiff clay.

Qu et al. (2002) presented a case history in Shanghai soft clay, which involves concern on base instability due to hydraulic uplift. To study ground settlement induced by artesian release, pumping tests were carried out. Some empirical relationships were obtained between reduced water table in the aquifer and resulted settlement around the pumping well. However, water tables in each pumping wells were found to interact with each other. Measured settlement at a certain location may be affected by various pumping wells, from which artesian release was conducted at different moments. This makes it rather difficult to interpret the measured results. The established empirical equations were therefore questionable.

Gue and Tan (2004) reported two case histories in soft clay underlying with an aquifer in Malaysia. Before reaching formation level of the excavations, hydraulic failure had already taken place and water flow into the clayey subsoil at the base of the excavation pit was evident. Subsequently, continued pumping was conducted to release artesian pressure. The dewatering caused significant drawdown in the groundwater in the surrounding retained ground. The lowering of the groundwater in the retained ground induced significant settlement (both immediate and consolidation) resulting in large cracks and differential settlement of adjacent buildings. It is found that dewatering may extend the influence zone of ground settlement to about 30 times the final excavation depth.

### ***2.4.2 Recommendations in Design Codes***

To design for excavations in clay with an underlying aquifer, calculation checking against base instability due to hydraulic uplift is required. In existing design codes such as codes for designing excavations in Shanghai (SMCC code), UK (BSI code) and Europe (EC code), hydraulic pressure which would initiate base instability is simply estimated as overburden pressure of clay inside excavation. This simplified estimation may be too conservative.

CIRIA Report 515 (Preene et al. 2000) suggests to consider effect of undrained shear strength of clay along soil-wall interface in resisting hydraulic uplift. But no specific guidelines are given. Moreover, mobilisation of undrained shear strength is accompanied by shear strain and hence basal heave. Therefore, the suggestion may not be applicable for cases where basal heave is concerned.

## **2.5 Techniques for Simulation of Multi-propped Excavation in Centrifuge**

To model a multi-propped excavation in a centrifuge, process of excavation and propping ideally has to be carried out in-flight. As far as the author is aware, only three cases of in-flight simulation of multi-propped excavation in centrifuge have been reported in the literatures.

Richards and Powrie (1998) simulated in-flight twin-propped excavations in stiff clay. In the test, effect of stress release due to excavation was simulated by draining away zinc chloride fluid with equivalent density as that of clay to be excavated. Prop was achieved with the aid of hydraulic locking unit. Before propping, the locking devices on each prop were held in the open position, so that the props were free to slide and would not support the wall. While there is a need for prop installation, the locking device on prop was activated in-flight to hold the prop in position.

Mcnamara (2001) developed an experimental setup for in-flight simulation of multi-propped excavation in centrifuge. Same as the setup developed by Richards and Powrie (1998), effect of excavation was modelled by draining away zinc chloride fluid (creating stress release). Model props were connected to hydraulic cylinders. By pressurising hydraulic cylinders, model props can be advanced against the retaining wall to simulate prop installation.

Lam (2010) reported development of a 2D actuator to simulate real in-flight excavation. A scraper was connected to the 2D actuator, which is able to move vertically and horizontally. By operating the 2D actuator, the scraper can remove soil in front of the wall in-flight. Model props were connected to a set of horizontally placed hydraulic actuators. When the in-flight excavation reaches each target level, hydraulic actuators at the corresponding levels were pressurised to simulate in-flight prop installation.

## 2.6 Summary

A review of the literatures to-date on ground deformation, soil-structure interaction and base instability (due to hydraulic uplift) of deep excavation in soft clay is reported in this chapter. Short-comings from existing literature are discussed and summaries as follows.

- (1) Given the vast amount of case histories collected by various researchers, it is still not easy to differentiate and understand ground deformations induced by excavations in densely built urban areas and so-called greenfield sites.
- (2) It has been reported from eight multi-propped excavations in Shanghai that excavation-induced ground deformations are relatively small than those in similar case histories in soft clay worldwide. Possible reasons are postulated but not justified.
- (3) To date, reliable field measurements of earth pressure on walls in excavations in soft clay are rarely reported.
- (4) Previous studies on base instability due to hydraulic uplift are mainly based on coarse field observations, from which ground deformation mechanism within soil body can not be obtained. In addition, without accurate real-time measurement of uplift movement of clay inside excavation and artesian pressure in the aquifer, the initiation of hydraulic uplift in all reported case histories still remains unknown. To date, systematic study on the initiation and failure mechanism of excavation base instability due to hydraulic uplift is not available in the literatures.
- (5) In current design codes (BSI, 1985; SMCC, 1997; EC 7, 2004), hydraulic pressure which initiates base instability is simply estimated as overburden pressure of clay inside excavation. This simplified solution may be too conservative.

- (6) The most commonly used method to prevent base instability due to hydraulic uplift is releasing artesian pressure inside excavation within cut-off walls. This method is reported to be ineffective under some circumstance (Haydon and Hobbs 1977) and maybe time consuming and costly. Existing alternative methods for base stabilisation are very rare.

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