Subsurface ground movements due to circular shaft construction

Abstract

The rapid development of modern metropolises has led to a shortage of surface space and in response engineers have pursued alternatives below ground level. Shafts are commonly used to provide temporary access to the subsurface for tunnelling and, as permanent works, are utilised for lifts or ventilation purposes. The construction sequence of axisymmetric shafts makes them a dramatically simple solution. In addition, circular shafts are inherently stiffer than other plan geometries. Those, perhaps, are reasons why circular shafts are preferred in situations of restricted space or unfavourable ground conditions. However, due to the lack of case histories reporting ground movements induced by shaft construction, no empirical prediction method for subsurface soil displacements exists. The work presented here seeks to provide clearer insights into surface and subsurface soil displacements induced by circular shaft construction by means of analysis on measurements obtained from centrifuge tests and available field data. Novel empirical equations and procedures are suggested for practical use.

Keywords

Shaft construction; Ground movements; Centrifuge modelling

LIST OF SYMBOLS

a	Constant indicates the depth at which maximum horizontal displacement occurs
b	Constant governs the height of the Gaussian curve
d	Distance from shaft wall
D	Shaft diameter
H	Shaft depth
K_0	Ratio between horizontal and vertical effective stresses at rest
OCR	Overconsolidation ratio
n	Multiple of shaft depth, H, to a distance d from the shaft wall where settlement becomes zero
S	Soil displacement
S_v	Vertical soil displacement
S_h	Horizontal soil displacement
S_v^{dz}	Vertical displacement at depth z and at a distance d from shaft wall
S_h^{dz}	Horizontal displacement at depth z and at a distance d from shaft wall
S_u	Undrained shear strength of clay
α	Empirical constant
$\emptyset_c'$	Critical state angle of shearing resistance
σ_h'	Horizontal effective stress
σ_v'	Vertical effective stress
σ_{v0}'	Maximum consolidation pressure for clay model in centrifuge test

INTRODUCTION

In urban environments shafts are often, by necessity, constructed adjacent to existing underground structures such as tunnels, deep foundations and basements. This makes the understanding of subsurface ground deformations and how they relate to surface displacement profiles increasingly important in assessing the possible effects of shaft excavation on nearby structures.

Faustin (2017) found that the magnitude and extent of ground deformations depends greatly on the shaft construction technique which can be classified by two categories: pre-installed shaft lining and concurrent shaft lining. In the former category, the shaft lining is installed before the shaft is excavated. The shaft lining can be formed by precast lining, diaphragm wall or sheet piles. The concurrent shaft lining involves excavation and then construction of the shaft lining. In concurrent shaft lining methods, spray-concrete lining (SCL) or precast segments are often used to form the lining.

The sources of ground deformations induced by shaft construction are depicted in **Figure 1** (after Faustin, 2017) and described below;

i) Radial unloading:

- For concurrent shaft construction: removing soil causes stress relief that results in soil movements into the shaft cavity before the lining is installed.
- For pre-installed shaft lining: when soil within the lining is removed, the unbalanced horizontal stresses are transferred to the shaft lining resulting in lining compression, leading to horizontal and vertical soil displacements. As the soil is supported by the shaft lining, the magnitude of soil displacements is expected to be smaller than that in concurrent shaft construction where the horizontal stress was reduced to zero without support prior to the shaft lining being installed. This was confirmed by back analysis on field data reported by Faustin (2017).

ii) Vertical unloading of the excavated base causes heave at the shaft plug which also contributes to the total soil deformation.

iii) Changes in the ground water table due to dewatering causes settlement. However, dewatering is not necessarily performed in all cases.

iv) Consolidation due to the changes in pore water pressure in the ground re-establishing equilibrium as a result of the excavation process. In the available case histories, only Schwamb (2014) reported long-term settlements which were considered minor compared with those which occurred during diaphragm wall construction. Because of this lack of reliable long term data, the current work only reports and analyses short-term soil displacements due to shaft excavation and does not consider long-term movements due to either consolidation or creep.

Up to 2016 there have been only a few empirical approaches for surface settlement $S_v^{surface}$ prediction including the widely used equation suggested by New & Bowers (1994).

$$S_v^{surface} = \alpha H \left(1 - \frac{d}{H}\right)^2 \tag{1}$$

It is important to note that **Equation 1** was derived from field measurements from only one shaft with $H = 26m$ and $D = 11m$, constructed using the concurrent shaft sinking technique in London Clay. Prediction of surface settlements using this equation would be dependent on the adopted value of α. In the original work, the reported value of $\alpha = 6 \times 10^{-4}$ provided the best fit with the field data presented in New & Bowers (1994) but the literature does not contain any further reported values (Schwamb et al., 2016). **Equation 1** is acknowledged to be quite conservative, particularly for pre-installed shafts (Schwamb, 2014; Faustin, 2017) because, for those conditions, the settlements are expected to be smaller as discussed earlier.

New (2017) studied field data from 13 shaft construction projects, with diameter range of $D = 6.5\ to\ 16.6m$, and found that the magnitude of surface settlement increases with larger shaft diameter (**Figure 2**). An extension to New & Bowers (1994) original equation was suggested by New (2017) which introduces a new variable, n, which governs that the surface settlement becomes zero at a distance of nH from the shaft wall, as described in **Equation 2**;

$$S_v^{surface} = \alpha H \left(1 - \frac{d}{nH}\right)^2 \tag{2}$$

The field measurements presented by New (2017) are all from projects in stiff London Clay with a similar construction technique. As such, the original value of α was retained in this work but values of n and α would be expected to increase in softer soils. New (2017) suggested that designers can consider **Equation 2** as a predictive tool with the values of n and α to be chosen dependent on the required degree of conservatism and that they should be supported by field data from similar shaft projects. New (2017) also acknowledged that surface settlement predictions are varied and difficult to make due to a lack of available field data. Whilst this work enables designers to assess surface settlements, there is no empirical approach to predict subsurface soil displacements even though more shafts are being constructed in crowded and sensitive urban areas with existing buried structures. These structures may require assessment of the effect of adjacent shaft construction in order that their serviceability is maintained.

The main purpose of the study presented in this paper is to gain a better insight into subsurface soil displacements induced by shaft construction by the means of centrifuge modelling and back analysis on available case histories.

CASE STUDIES

An extensive literature review on shaft construction, carried out by Faustin (2017), shows that there have been only 18 case histories on circular shaft construction published between 1980 and 2016. There have been some additional cases in 2017 (Faustin, 2017; New, 2017). Most of these case studies report surface settlement, only 3 cases presenting subsurface soil displacements and only 1 case reported surface horizontal displacement. Details of case histories used in this section are presented in **Table 1** with shaft geometries, construction techniques and soil conditions included**.**

It is worth noting that not all measurements from these publications are reported in this paper. Even though there were data from four extensometers in Schwamb et al. (2016), the readings from two of them were less than 0.5mm which is well below the resolution of the instrumentation and are therefore not presented here. Hence only readings from two extensometers in Wong &

Kaiser (1988) and Schwamb et al. (2016) are used for subsurface vertical displacements analysis.

Only one in two inclinometer measurements reported by McNamara et al. (2008) and Wong & Kaiser (1988) are also utilised in this study. This is because the other inclinometer readings were either affected by existing piles (McNamara et al., 2008) or not fully reported; possibly due to poor accuracy (Wong & Kaiser, 1988) and hence these are not used.

Even though there were two data sets for horizontal and vertical surface displacements available in New & Bowers (1994), only one set was used because the other was deemed unreliable due to the effects of heavy plant movements and nearby excavations.

The rarity of high quality field measurements from shaft construction in the literature is possibly due to the high cost of monitoring schemes especially for deep shaft construction where deep drilling, for casings to house inclinometers and extensometers, is required to be below the shaft plug level in order to achieve representative results. In addition, shaft construction sites are normally occupied with activities that may affect the measurements leading to unrepresentative data and the existence of the underground structures that may alter the soil deformation mechanisms which causes difficulties in the interpretation of the measurement results.

The challenges in obtaining representative soil displacements due to shaft construction can be overcome by the centrifuge modelling technique due to its advantageous capabilities in modelling soil behaviour in geotechnical events (Taylor, 1995). Recent developments in technology allows accurate measurement of soil deformations at any position in small scale centrifuge models (Stanier et al., 2015; Le et al., 2016).

CENTRIFUGE TESTING

A bespoke centrifuge model (**Figure 3**) was designed and used to investigate soil deformations induced by shaft construction and is described here.

Test series

The tests were performed using a fixed geometry but varying undrained shear strength S_u of the clay. The clay model (Speswhite kaolin) was one dimensionally consolidated in a soil container (known as a strong box) using a hydraulic consolidometer to a maximum vertical effective stress σ'_{v0} equal to $350kPa$ and $500kPa$ for test CR350 and CR500, respectively. The samples were swelled back to a vertical stress of $250kPa$ for both tests. The consolidation pressures were chosen for three reasons:

- To achieve overconsolidated soils, representative of real soil in urban environments (Parry, 1970);
- For the clay to be stiff enough for model making;
- For the clay not to be so stiff such that the soil deformations, induced by the simulated shaft excavation, would not be too small to measure accurately.

The water table was set at soil surface level. Properties of the Speswhite kaolin used can be found in Grant (1998). More details on the testing apparatus and procedure can be found in Divall & Goodey (2016) and are described briefly below.

Test apparatus

A schematic of the centrifuge model is illustrated in **Figure 3**. The excavation was simulated by a semi-circular cavity cut into the clay which could be viewed through the front Perspex window of the centrifuge model container. The dimensions of the excavation are $D = 80mm$ and $H = 200mm$. The excavation is supported by two components:

- The shaft liner (**Figure 3b**): $200mm$ high, $71mm$ in diameter. The cavity in the shaft plug (**Figure 3b**) has an internal diameter of $65mm$ and $45mm$ deep to allow basal heave to develop during the excavation simulation.
- A latex bag encloses the shaft liner with the cavity filled with a heavy fluid (commercially known as Sodium Polytungstate or SPT). This SPT fluid was prepared to have a density equal to the clay used in the model, $17.5kN/m^3$, to provide support to the soil.

The latex bag has a thickness of 1.5mm and together with the liner with radius of R equal to $35.5mm$ leaves a void of 3mm between the excavation and the liner. This is initially supported by the heavy fluid of which the head was set to be level with the ground surface. The excavation process was simulated by draining the heavy fluid to generate radial and vertical unloading that results in ground deformations including heaving at the bottom of the shaft.

It is worth noting that using heavy fluid to support the soil implies an assumption that $K_0 = 1$, i.e. $\sigma_v' = \sigma_h'$, within the soil mass which is slightly different from the K_0 calculated by **Equation 3** (Mayne & Kullhawy, 1982) and shown in **Figure 4**.

$$K_0 = \left(1 - sin\emptyset_c'{}'\right)OCR^{sin\emptyset_c'} \quad (3)$$

For Speswhite kaolin, $\emptyset_c' = 23°$ (Grant, 1988). It can be seen that K_0 values calculated by **Equation 3** for the soil along the shaft depth (0 to 200mm) are close to 1 (**Figure 4**). Near the surface the values of K_0 are much larger. However, as the vertical stresses near the surface are very small the effect of this dissimilarity in K_0 is negligible and was confirmed by good agreement between the centrifuge test results and field measurements which are presented later in this paper.

Test procedure

On the test day, the strong box was removed from the hydraulic consolidometer to begin the model making procedure. All exposed surfaces of the clay sample were sealed with silicone oil to prevent drying and from this point onwards the model making process was carried out as rapidly as possible in order to preserve the stress history of the soil. The clay was then trimmed to the correct model height and the semi-circular cavity was cut for the shaft support system to be placed within. The front face of the clay model was sprayed with dyed Leighton Buzzard Sand (fraction B) to create the texture necessary for optimising the geoPIV post-test analysis. The front Perspex window was then bolted to the model container before the heavy fluid was injected into the rubber bag.

The models were accelerated to 100*g* and left running until the clay had reached effective stress equilibrium. The excavation process was then simulated by draining the heavy fluid. Data relating to deformations of the clay model and heavy fluid level were recorded at 1 second intervals for later analysis. In practice, the unloading rate varies in different projects due to different soil conditions, shaft geometries, and construction techniques. Therefore, the construction rate for these centrifuge tests was selected to ensure an undrained response to unloading. The total time required to simulate the complete unloading event was 25s in the centrifuge which represents around 2.5 days at prototype scale. The model was then spun down and shear vane readings were carried out to determine undrained shear strength, S_u, of the clay model. The average S_u of the model clay from surface to the shaft plug were $44.5kPa$ and $57.8kPa$ for tests CR350 and CR500, respectively.

Measurement of soil movements

GeoPIV_RG (Stanier *et al*., 2015) was used to analyse soil movements at the front face of the model from digital images taken during the test (**Figure 3a**). One of the drawbacks of the use of digital image analysis in centrifuge modelling is the friction at the interface between the Perspex window and the soil model that may affect the soil movement mechanism. However, results from image analysis reported by Grant (1998), Divall (2013) and Le (2017) showed that once the soil at the interface moved, it continued to displace at the same rate as the rest of the model. Therefore, the friction at the interface is negligible to the development of soil displacements and its mechanisms. Le (2017) conducted a series of shear box tests to examine the friction at the interface and found that the texture material was the key factor. In this research Leighton Buzzard Sand was used as it induced less friction compared with other texture material (e.g. glass balotini) owing to its lower angle of friction.

TEST RESULTS

Typical soil displacements, immediately after the all of the fluid was drained out of the rubber bag, in the centrifuge test CR500 are presented in **Figure 5a** and the corresponding displacement contours are presented in **Figure 5b.** It can be seen that soil displacement is symmetrical (**Figure 5a**). From **Figure 5b**, soil displacements in the vertical and horizontal directions become very small (less than 0.01mm) at a distance of 150 to 200mm from the shaft centreline. This confirms that the soil container was large enough and boundary effects were negligible.

Figure 6 illustrates vertical and horizontal soil displacements in test CR500 at various distances up to 80mm ($0.4H$) away from the shaft wall. For clarity, only data on one side of the model is presented. Vertical displacement increases towards the shaft wall and decreases with depth which is similar to observations made by previous researchers (New & Bowers, 1994; New, 2017; Faustine, 2017; Schwamb et al., 2016). Interestingly, the profile of displacements, S_h and S_v, with depth, z, at various distances from the shaft wall (up to $d = 40mm = 0.2H$), show similar distribution patterns. For data at a distance beyond 40mm from the shaft wall, for example $d = 80mm$ (**Figure 6**), the distribution of displacement with depth shows a different shape. Further analysis on subsurface and surface soil displacements is discussed below.

Subsurface soil vertical displacements

Figure 7 presents subsurface vertical movement profiles at various distances $d = 0.05H\ to\ 0.2H$ from the shaft wall at the end of the two centrifuge tests along with data from Wong & Kaiser (1988) and Schwamb et al. (2016). Vertical movement at depth z and a distance d from the wall, S_v^{dz}, is normalised by maximum settlement at that distance, S_{vmax}^{d}, and z is normalised by H. The results from both centrifuge tests fit well with data from Wong & Kaiser (1988) but not with data from Schwamb et al. (2016) and the likely reason is explained below.

In Schwamb et al. (2016), the extensometer readings were baselined with bottom anchors which were installed at depths higher than that of the base of the shaft. The extensometer readings can only reflect absolute movements if the bottom anchors are fixed. However, finite element analysis showed that removal of the overburden pressure at excavation surface caused the adjacent ground to heave and the bottom anchor of the rod extensometers to move upwards by approximately 3mm (Schwamb et al., 2016). The heave behaviour near the shaft plug is confirmed by the centrifuge tests (**Figures 5, 6 & 7**). If the extensometer data are corrected by adding 3mm to the readings then the profile of subsurface vertical movements from Schwamb et al. (2016) (labelled as corrected) are also in good agreement with other data (**Figure 7**). It is worth noting that the bottom extensometer in Wong & Kaiser (1988) was installed into clay shale layer, presumably a very stable stratum, and below the shaft plug level.

Despite the differences in soil conditions, construction techniques and excavation dimensions in the considered shafts, vertical movements at depth z, S_v^z, when plotted in the manner of **Figure 7**, show a consistent distribution which can be described by **Equation 4**;

$$\frac{S_v^{dz}}{S_{vmax}^d} = 1.15 - \frac{0.15}{1 - z/H} \tag{4}$$

(applicable for $z \leq 0.9H$)

Equation 4 and **Figure 7** show that maximum vertical movement occurs near the ground surface (when $z = 0$) and decreases with depth.

Subsurface soil horizontal displacements

Figure 8 presents subsurface horizontal soil movements in the considered shafts, reported by McNamara et al. (2008) and Wong & Kaiser (1988), together with results from two centrifuge tests. Horizontal movement at depth z and at a distance d from the wall, S_h^{dz}, is normalised against the maximum horizontal displacement at that radial distance, S_{hmax}^d and the depth z is normalised against H. Despite there being some anomalies from field measurements, most of the data points agree well with the trend shown by the centrifuge test results.

The profile of horizontal soil movement with depth shows a similar distribution to a Gaussian curve with the maximum value at $z/H = 0.6\ to\ 0.8$. This is thought to be analogous to the horizontal load distribution against a retaining wall where the load acts at depth $z/H = 2/3 \approx 0.67$. A best fit Gaussian curve (**Equation 5**) is proposed and also plotted in **Figure 8**.

$$\frac{S_h^{dz}}{S_{hmax}^d} = \exp\left[-\left(\frac{z/H - a}{b}\right)^2\right] \tag{5}$$

where $a = 0.6$ implies that S_{hmax}^d occurs at $z = 0.6H$;

$b = 0.4$ governs the height of the Gaussian curve.

The value of a and b can be varied to find a best fit Gaussian curve.

New (2017) commented that there is inadequate field data for reliable prediction of horizontal soil displacements and these are normally assumed to have similar magnitude to the vertical soil displacement. Similarly, GCG (2007) suggested that for ground movements due to shaft excavation at the surface $S_{hmax}^{surface} = S_{vmax}^{surface}$. In order to examine this assumption, **Figure 9** plots vertical and horizontal displacement at the surface from test CR500 and the field measurements from New & Bowers (1994). Again, for clarity, only data from one side of test CR500 is presented along with field measurements. Whilst the data plotted on Figure 9 are not directly comparable (due to significantly large differences in undrained shear strength) it is clear from both the centrifuge test results and field measurements that the maximum surface vertical displacement is significantly larger than maximum horizontal displacement. Therefore, the assumption $S_{hmax}^{surface} = S_{vmax}^{surface}$ may lead to overestimation of horizontal displacement especially at subsurface as S_{hz} increases with depth z as shown in **Figure 8**.

Most of the centrifuge test data (with $d < 0.2H$), some of which is presented in **Figure 6**, shows values of $S_{hmax}^{d}/S_{vmax}^{d}$ in the range $1\ to\ 1.9$. As shown in **Figure 7** and **Equation 4**, maximum settlement occurs at surface $S_{vmax}^{d} = S_{v}^{d-Surface}$. With a surface settlement profile estimated by **Equation 2,** assuming $S_{hmax}^{d} = (1\ to\ 1.9)S_{v}^{d-surface}$ allows a range of horizontal displacements at a distance d at any depth z to be estimated using **Equation 5** (which would ideally be supported by similar case studies). The data from **Figure 9** shows soil displacements in the vertical and horizontal directions to be considerably smaller in the field compared with those measured in the centrifuge. New and Bowers (1994) reported values of S_u in London Clay varying from 50kPa to 250kPa whereas those in centrifuge test CR500 had an average S_u of approximately 58kPa. Engineers could make a judgement based upon their site soil conditions when estimating soil displacements given the information relating to undrained shear strengths of clay in the centrifuge tests and the literature contained in this paper. The assumption $S_{hmax}^{d} = (1\ to\ 1.9)S_{v}^{d-surface}$ is examined in a back analysis on field measurements later in this paper.

COMPARISON BETWEEN CENTRIFUGE TESTS AND SHAFT EXCAVATION IN PRACTICE

There are, clearly, significant differences between the reported experiments and the construction of a shaft in practice. These primarily relate to the method and rate of construction and the stiffness of the shaft lining. As previously stated, the rate of unloading in the tests was chosen in order to, as much as possible, replicate an undrained event. The field data utilised comes from a variety of projects in a variety of soil conditions which may or may not behave in an undrained way. Nevertheless, good agreement between this field data and the centrifuge tests has been reported which suggests that the unloading rate had negligible impact on the soil displacements during shaft excavation.

When considering the shaft lining, it could be assumed that the relative hoop stiffness will have an effect on the magnitude of soil displacements around the shaft excavation (a fact also noted by Schwamb et al., 2016). The focus of the current work is the pattern, rather than the magnitude, of subsurface soil displacements induced by shaft excavations. From **Figures 7** and **8**, it can be seen that despite the (assumed) difference in relative hoop stiffness of the shafts in the reported case histories compared with the centrifuge tests (arising from the use of different shaft linings and construction methods), the patterns of subsurface soil displacements were observed to be similar. This implies that relative hoop stiffness has negligible impact on the pattern of subsurface soil displacements induced by shaft excavation.

EXAMPLE APPLICATION OF NEW EQUATIONS

Figure 10 presents a flow chart on how to use **Equations 2, 4 & 5** to predict subsurface vertical and horizontal displacements. The data set from Wong & Kaiser (1988) is used to demonstrate their applicability.

The first stage of the prediction is to generate suitable values of n and α for use in **Equation 2**. As previously stated New (2017) acknowledged that these values should be selected with reference to similar case histories, however in this example there is no such data available. As such, the original values of New (2017) are adjusted by assessing the ground conditions and geometry of the shaft reported by Wong & Kaiser (1988). The shaft diameter of Wong & Kaiser (1988) is approximately four times smaller than the cases reported in New (2017) and the undrained strength of the soils is estimated to be 50% of the strength of London Clay. A narrower shaft is likely to lead to a narrowing of the surface settlement extent (i.e. a reduction of n) and a decrease in settlements generated (i.e. a reduction in α). The decrease in soil strength is likely to lead to an inverse effect (i.e. increase in settlements and extent reflected by increases in n and α). Using this rationale, estimates of n and α are derived from the original values of n = 1.5 and α = 6 x 10^{-4} by doubling these values (to account for soil strength reduction) and then reducing them by a factor of 4 (to account for reduction in shaft diameter). This leads to an overall factor of 0.5 and thus values of n = 0.75 and α = 3 x 10^{-4}.

Using these values in **Equation 2** leads to the profile of surface settlement shown in **Figure 11**. Also plotted are the data from Wong & Kaiser (1988) which shows reasonable agreement with the profile generated by **Equation 2** whilst acknowledging that the basis for selection of n and α values is open to interpretation. A best fit exercise to the measured data was carried out resulting in very good agreement between the data and **Equation 2**. The values of n and α arising from this exercise were 0.85 and 2.55 x 10^{-4} respectively however, for the purposes of this discussion, the original estimated values are used.

Surface settlement at the positions of inclinometer SI#1 ($d = 0.5m$) and extensometer MS#1 ($d = 1.5m$) are determined as $S_{v-surface}^{SI\#1} = 5.61mm$ and $S_{v-surface}^{MS\#1} = 4.86mm$ by using distance d in **Equation 6**. Thus, **Equations 4 & 5** with the determined $S_{v0}^{MS\#1}$ and $S_{hmax}^{SI\#1}$ give subsurface vertical and horizontal displacements which are plotted in **Figures 12a & b** along with the corresponding field measurements. The limits of the range identified from the centrifuge tests $S_{hmax}^{d}/S_{v}^{d-surface} = (1\ to\ 1.9)$ are used to generate the two curves in **Figure 12b**.

The predicted vertical displacement with depth is marginally smaller than the measured values. Nevertheless, the predicted vertical displacement with depth is very similar with the field measurement in terms of magnitude and shape.

For subsurface horizontal displacement, the assumption $S_{hmax}^{d} = S_{v}^{d-surface}$ provided a very good fit with the field measurement whereas $S_{hmax}^{d} = 1.9 S_{v}^{d-surface}$ overestimated the magnitude of soil deformations. More field data are needed to assess whether S_{hmax}^{d}=$S(1\ to\ 1.9)_{v}^{d-surface}$ and caution should be exercised when applying this relationship.

CONCLUSION

The results of centrifuge tests carried out in this research show good agreement with field data from various shaft projects which provides a clearer insight into subsurface soil displacements due to shaft excavation. Based on experimental evidence and field measurements, two novel empirical equations have been suggested to describe unique distributions of soil movements with depth regardless of soil conditions, construction techniques and shaft dimensions. A flow chart on how to use these equations to predict soil movements in any direction at any point is provided for practical use.

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LIST OF FIGURES

No	Reference	Location	Construction method	Ground conditions	Shaft geometry		Available ground movements data
					D (m)	H (m)	
1	Wong & Kaiser (1988)	Edmonton, Canada	Concurrent shaft lining Corrugated and Flanged steel plates	Sand & clay (6.5m) Glacial matrix (13m) Clay shale	2.4 to 3.2	20	Surface: Sv Subsurface Sh, Sv
2	Schwamb et al. (2016)	London, UK	Pre-installed shaft lining Diaphragm wall	London basin deposits	30	73	Subsurface Sh, Sv
3	McNamara et al. (2008)	London, UK	Concurrent shaft lining Pre-cast segments	London clay (30m) Lambeth Group (18m)	8.2	37.5	Subsurface Sh
4	New & Bower (1994)	London, UK	Concurrent shaft lining Pre-cast segments (16m) SCL (10m)	Superfical deposits (3.5m) London Clay	10.65	26	Surface Sh, Sv
5	This study	City, University of London	Pre-installed shaft lining	Speswhite kaolin	8*	20*	Surface Sv, Sh Subsurface Sv, Sh

* dimension in equivalent prototype.

Table 1. Case histories used in this paper.

Figure 1

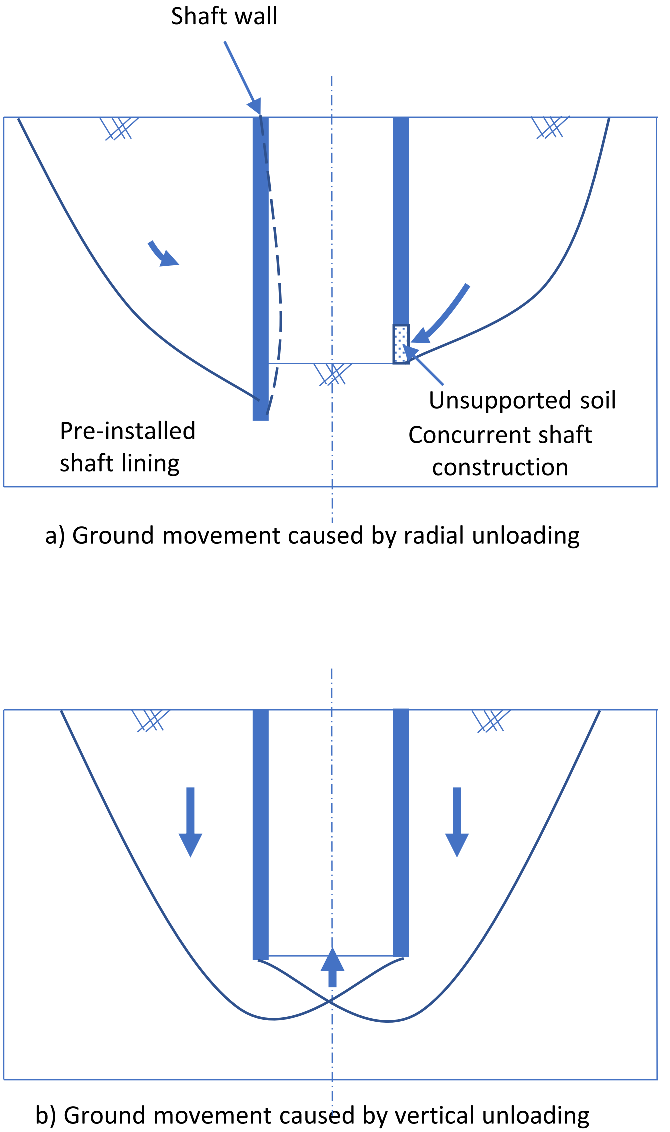


Fig. 1: Sources of ground movements due to shaft construction (after Faustin, 2017).

Figure 2

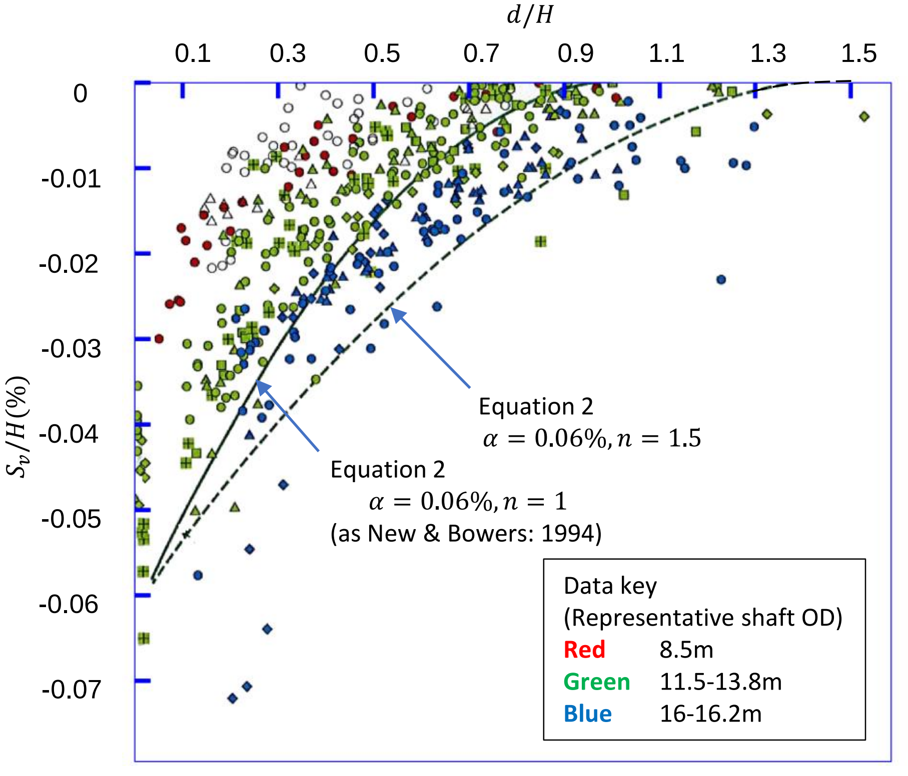
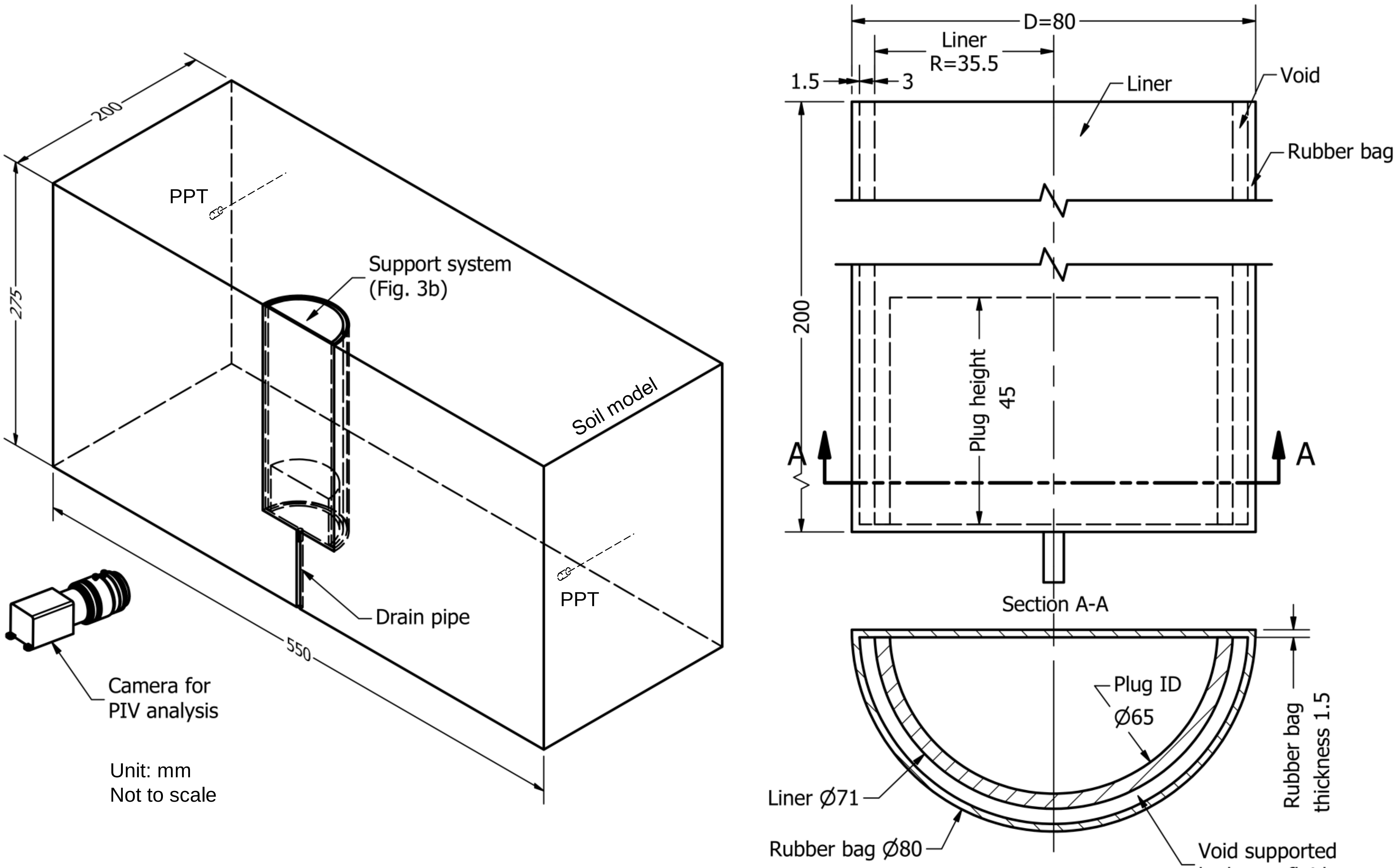


Fig. 2: Surface settlement data from 13 shafts (after New, 2017).

Figure 3



a) Test apparatus

b) Support system

Fig. 3: Schematic of centrifuge test apparatus.

Figure 4

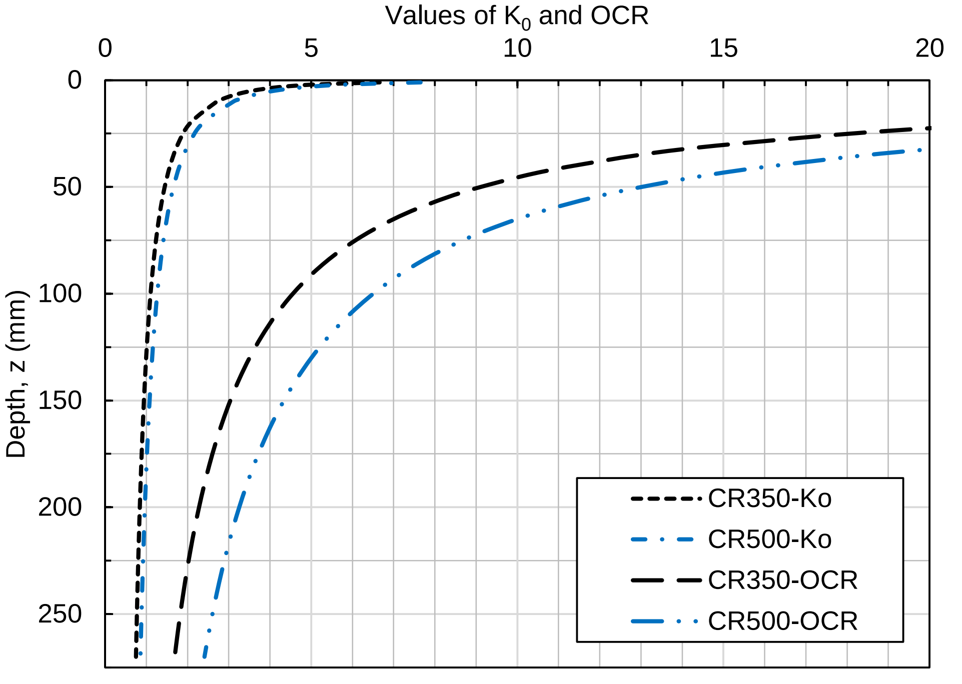
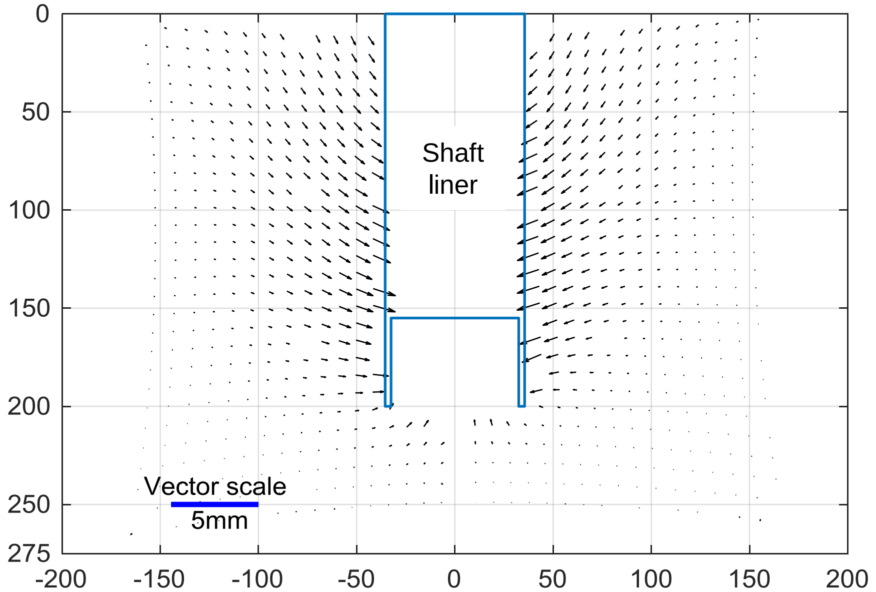
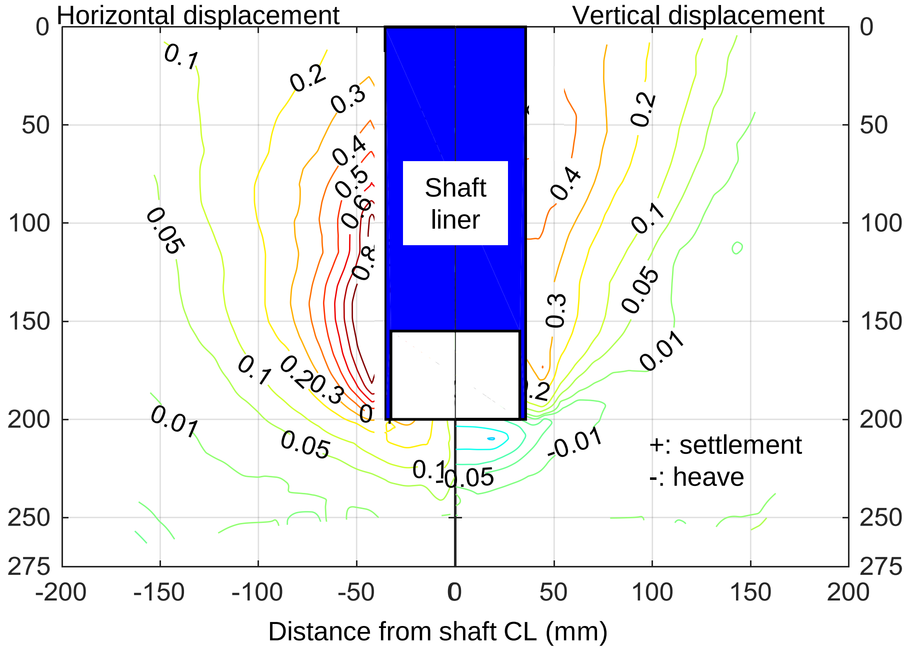


Fig. 4: Profiles of K_0 and OCR with depth in centrifuge models.

Figure 5



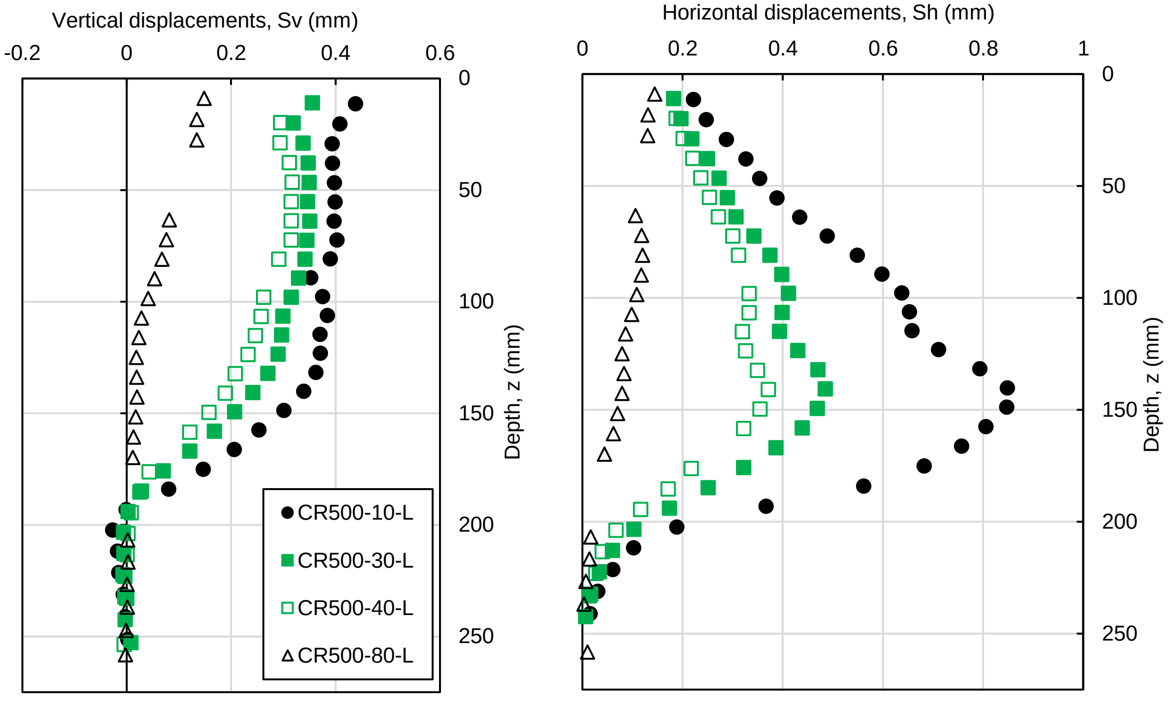
a) Soil displacement field



b) Horizontal and vertical displacement contours

Fig. 5. Soil deformations in test CR500 after all fluid was drained out.

Figure 6



Vertical displacement
+: settlement
-: heave

Fig. 6: Typical subsurface soil displacements in test CR500.

Figure 7

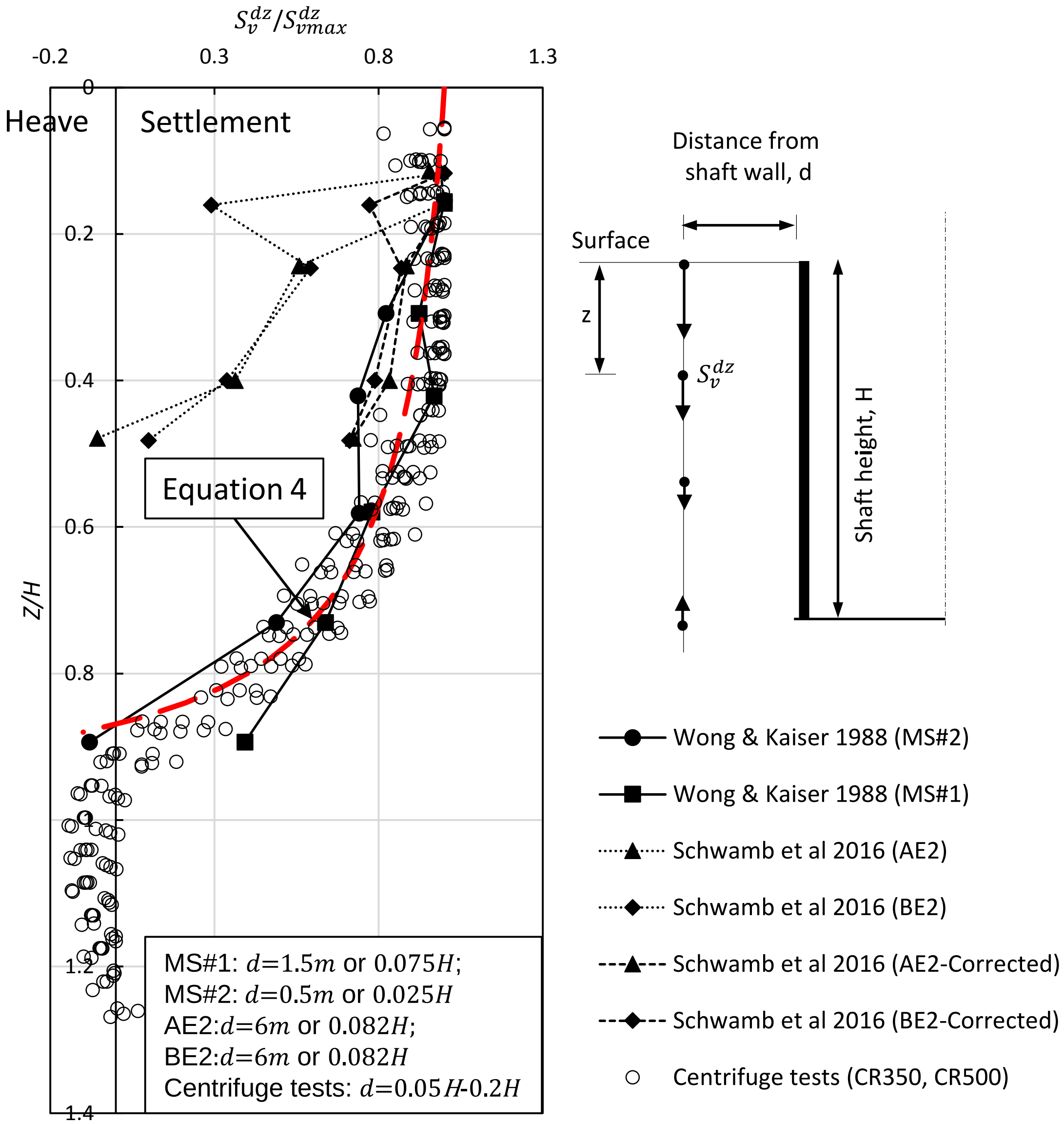


Fig. 7: Subsurface vertical movements.

Figure 8

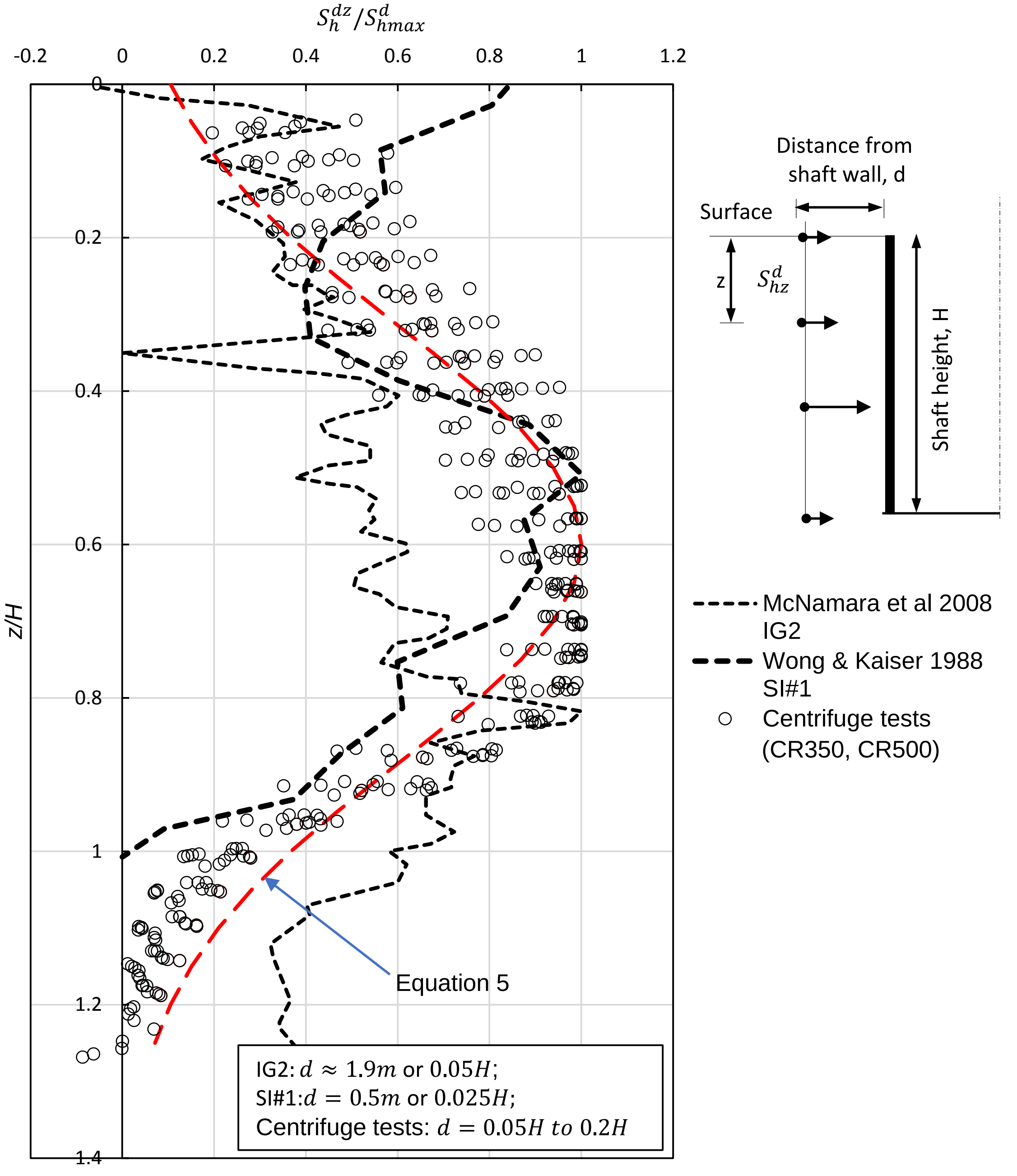


Fig. 8: Subsurface horizontal movements.

Figure 9

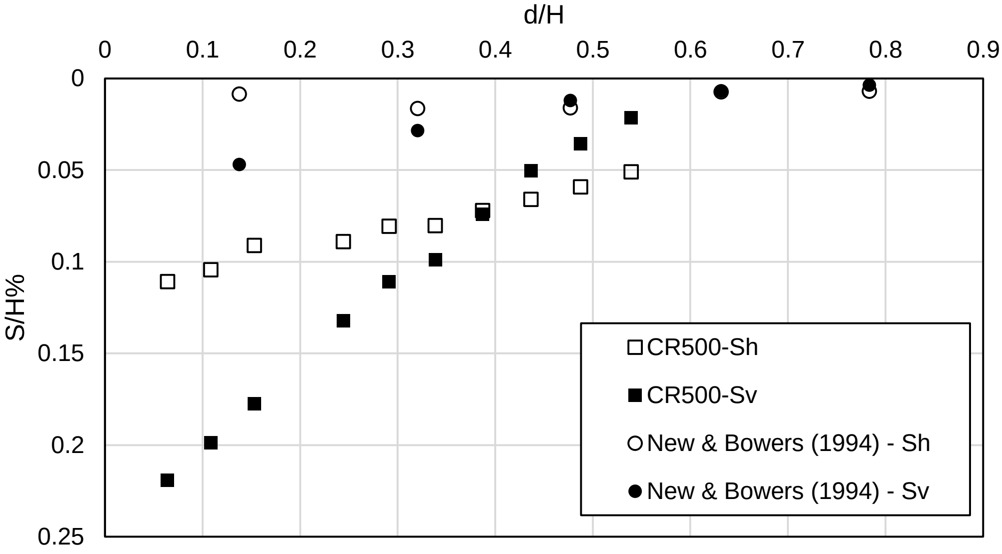


Fig. 9: Displacements at surface in centrifuge test and New & Bowers (1994).

Figure 10

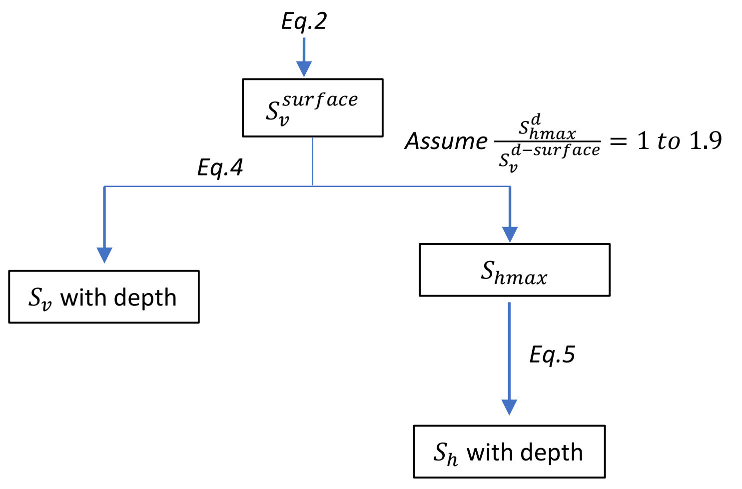


Fig. 10: Suggested flow chart on the usage of the proposed equations.

Figure 11

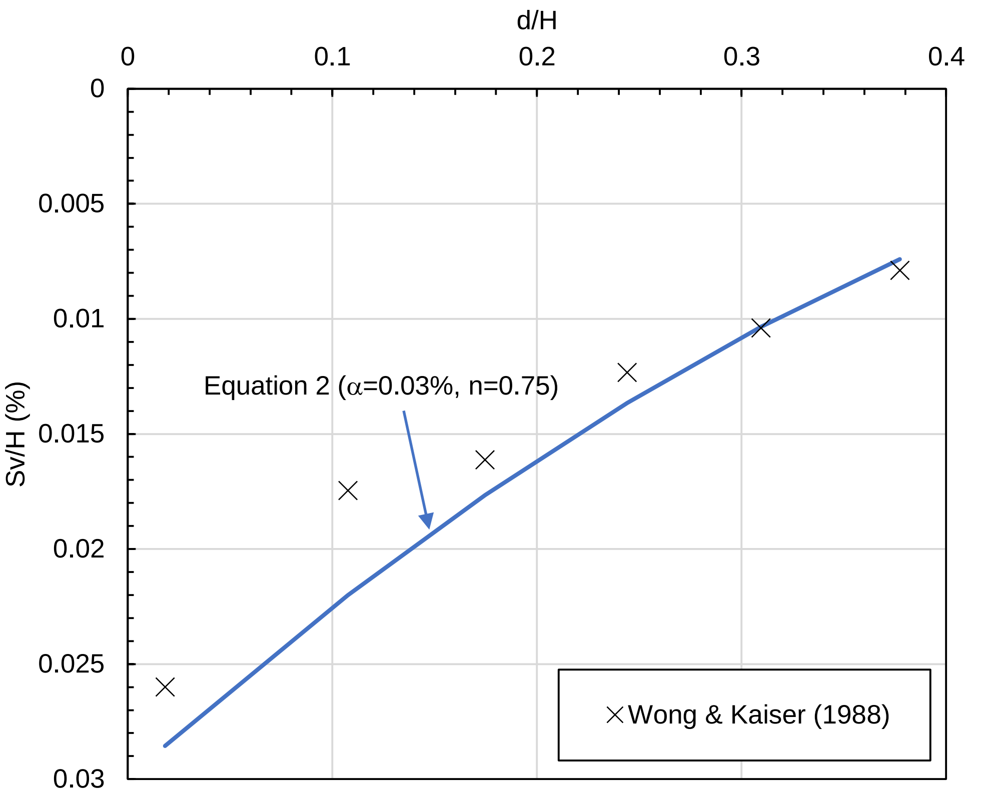


Fig. 11: Comparison on Surface settlement in Wong & Kaiser (1988) and back analysis using Equation 2.

Figure 12

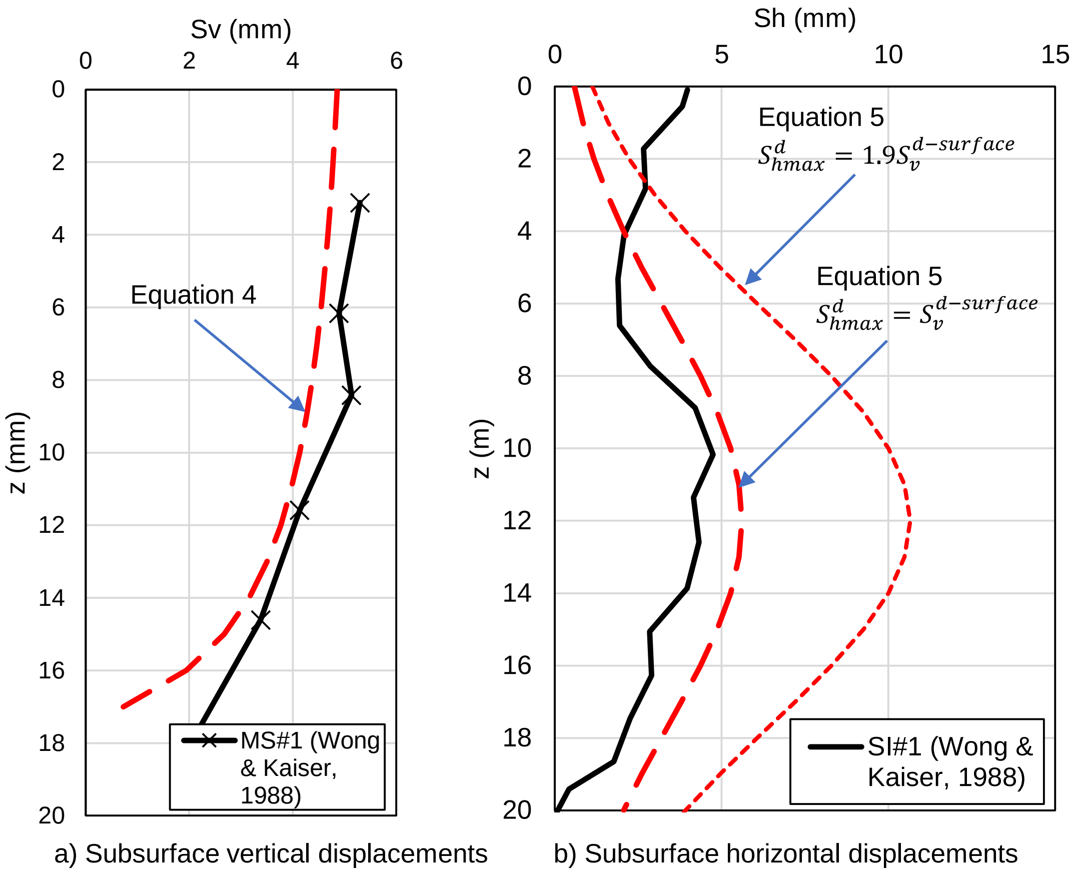


Fig. 12: Comparison on subsurface soil displacements in Wong & Kaiser (1988) and back analysis using Equations 4 & 5.