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Assessment of field measurements of three deep excavations from Copenhagen's Sydhavn metro line.

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Abstract

Although several authors have investigated the performance of deep excavations over the past decades, most of the case studies analysed flexible retaining systems and/or soft ground conditions. Nevertheless, strict deformation requirements in constructions in urban environments frequently impose stiff design solutions for excavations. This study expands the existing data in the literature by evaluating the performance of three deep metro station excavations, located in the vicinity of buildings, and constructed by the bottom-up method in stiff over-consolidated deposits as well as soft rocks in Copenhagen. The data from the long-term comprehensive instrumentation program was employed for this purpose. Lateral wall deflections, axial forces of prestressed anchors and struts and ground water levels were analysed and discussed in accordance with the construction plan. The measured lateral wall deflections at the three case studies indicate a typical cantilever deflection shape until the installation of the first anchor, while the wall starts bending towards the excavation after the second excavation stage. The ratio between the maximum wall deflection and the excavation depth is at most equal to 0.25%, which is close to the lower boundary of reported ranges for available case studies. Corner effects were observed at one case study (Sluseholmen) where the width over length ratio of the underground excavation was relatively small. A linear correlation between the maximum wall deflection and the excavation depth was observed for wall deflections measured after the installation of the first support.

Keywords: metro station excavations, field measurements, wall deflections, limestone, Copenhagen

1. Introduction

The limited space in urban environments has increased the exploitation of underground space, especially for hosting utilities and transportation networks (subway, tunnels). The construction of deep excavations in densely populated areas is nowadays common. For this reason, design and construction of deep excavations is governed by the wall response and ground movements, since excessive soil deformations may lead to the damage of adjacent structures and utilities, even resulting in huge economic and/or human loss. One means of verifying the performance requirements during construction is field monitoring during the process of the excavation. It is widely employed to collect continuous measurements of ground movements and wall deflections as reported in several case studies (e.g Long, 2001).

In the past decades several studies (Peck 1969; Mana & Clough 1981; O' Rourke 1981; Clough & O' Rourke 1990) extensively investigated the performance of excavations and proposed empirical approaches for evaluating the magnitude of wall deflections and ground movements behind the wall on the basis of field observations in different types of soil. The same studies evaluated the effect of the wall stiffness, support stiffness and spacing, depth to an underlying firm layer, excavation width and strut preload on the wall deformations and settlements. Subsequently, empirical charts were derived, and it was shown that the measured performance was governed by the stiffness of the retaining system and of the soil (sheet or soldier pile walls with an excavation depth less than 20 m and/or soft ground conditions).

On the other hand, excavation-induced wall deflections and ground movements were found to be overestimated by these empirical approaches when a stiff retaining system was used (Ou et al. 1993; Wang et al. 2005). In particular, Ou et al. (1993) analysed and discussed the performance of ten deep excavations supported by diaphragm or contiguous pile walls with different construction methods (bottom-up, top-down, berm method) and excavation or wall characteristics in layered soft ground conditions (clay and silty clay). It was concluded that the maximum lateral deflection often occurred close to the excavation surface and typically varied from 0.2% to 0.5% of the excavation depth, which is larger than that reported by Clough & O'Rourke (1990), yet smaller than that reported by Peck (1969). The former refers to cases in stiff clays and sands, while the latter to cases with mainly sheet or soldier piles as lateral supporting systems in different ground types (soft to stiff clay and sand). Similar ranges were reported by Wang et al. (2005), who discussed the field measurements of six multi-strutted excavations constructed for the metro line in Shanghai. The retaining walls were diaphragm walls with different stiffnesses and characteristics in soft clay. Long (2001) concluded that the deformations of deep excavations in non-cohesive soils and stiff clays are largely independent of the system stiffness, as the average maximum deflection was found equal to 0.17%, 0.19% and 0.16% of the excavation depth for propped, anchored, and top-down construction, respectively. The same observation was reported by Bryson & Zapata (2012), while in the same study system stiffness was found to have a significant role in the excavation performance in soft-to-medium clays. Bryson & Zapata (2012) suggested a semiempirical approach for selecting the stiffness of the excavation support system, by introducing a new parameter called relative stiffness ratio that considers the effects of soil-structure interaction, relative bending resistance (contribution of wall and support elements) and soil conditions.

This study aims to investigate the performance of deep excavations that were carried out in urban environment and therefore require a stiff retaining system. The case studies presented in this paper are the Enghave Brygge (EBR) station, the Ørstedværket (ØVK) shaft and the Sluseholmen (SLU) station which were constructed as a part of the extension of Copenhagen's metro line to Sydhavn. Field measurements of lateral wall deflections, axial forces of struts and anchors as well as ground water levels inside and outside the excavation are analysed and described. The lateral wall deflections in the stiff over-consolidated glacial deposits found in Copenhagen are established at different construction stages and compared with similar cases reported in the literature.

2. Geological and Geotechnical conditions of the case studies

The geological conditions of the Copenhagen area consist of Quaternary deposits overlaying the Paleogene deposits, mainly consisting of Danian Limestone and locally of Selandian greensand (South Copenhagen). The Quaternary deposits are composed of some transitional deposits covered by true glacial deposits which are covered by different types of post and late glacial sediments and fill layers (Frederiksen et al., 2002). The fill layer is heterogeneous and consists primarily of sand, gravel and clay and occasionally organic material or building debris. The glacial deposits are divided into groups of low permeability tills (upper and lower clay till/sand till) with intermediate high permeability meltwater deposits (upper, middle, and lower sand and gravel). The bedrock in the Copenhagen area is divided into three lithostratigraphical units (Knudsen et al., 1995): the Selandian Greensand, the Copenhagen Limestone, and the Bryozoan Limestone. The Bryozoan Limestone forms an approximately 60 m thick unit resting on the Maastrichtian white chalk. The Copenhagen Limestone overlies the bryozoan Limestone, and it is characterized by alternating indurated and less indurated beds and can be generally described as a weak rock with very hard layers or nodules of flint (Paulatto & Carstensen, 2017). The Copenhagen Limestone formation is divided into three stratigraphical units: the Upper Copenhagen Limestone (UCL), the Middle Copenhagen Limestone (MCL) and the Lower Copenhagen Limestone (LCL). The upper part of the Limestone is typically disturbed or highly fractured by glacial processes, with the thickness of the disturbed zone varying from 0.3 m to 3.6 m.

Extensive field borehole investigations defined the site stratigraphy of the case studies, which is presented in Figure 1 together with the excavation's typical cross sections. In particular, 9 new boreholes were drilled at EBR and ØVK stations and information from 5 available boreholes from GEUS were derived so as the stratigraphy to be obtained. The stratigraphy at SLU was obtained by 16 new boreholes and data from 4 existing boreholes. It can be seen in Figure 1 that the ground conditions at EBR and ØVK are similar, however the thickness of the sand till layer decreases towards the North side of the excavation and consequently a different design profile was considered (see Figure 1). On the other hand, the fill layer at SLU is thicker than in EBR and ØVK station, and its thickness varies along the excavation. It should be mentioned that glacial deposits in SLU station were found to overly directly the Bryozoan Limestone, with the first 2 m of the formation to be disturbed. In addition to the boreholes, field and laboratory geotechnical tests were performed. The physical and mechanical properties of the soil and rock units according to the Geotechnical Investigation Report (Metroselskabet, 2018) are presented in Table 1.

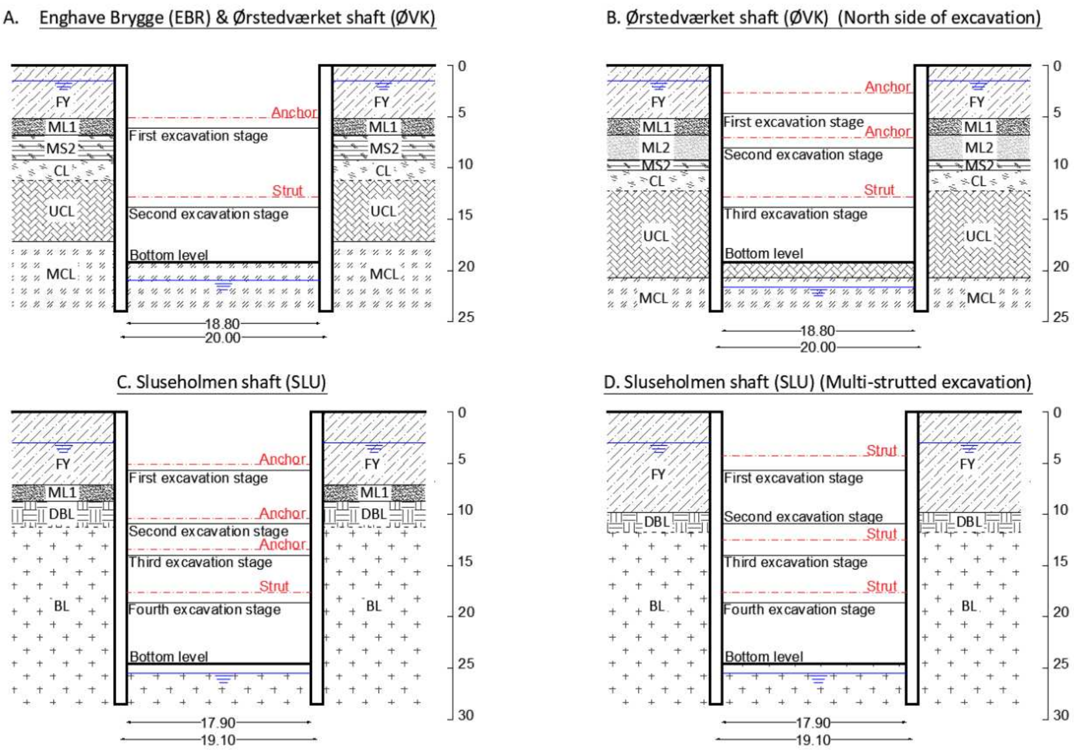

Figure 1: Typical cross sections of the excavations carried out at Enghave Brygge (EBR), Ørstedværket (ØVK) and Sluseholmen (SLU). The soil design profiles for each case are also displayed.

Abbreviation	Bulk unit weight γ_{bulk} [kN/m^3]	Poisson's ratio v / v_u [-]	Young's/ Rock mass modulus E' / E_m [MPa]	Angle of Shearing resistance φ' [°]	Cohesion c' [kPa]	Pre-consolidation stress σ'_p [kPa]
FY	18.0	0.2 / 0.5	8	30	0	σ'_{v0}
ML1	21.5	0.2 / 0.5	50	33	10	600
ML2	21.5	0.2 / 0.5	65	33	10	2400
MS2	21.0	0.2 / -	70	38	0	2400
CL	21.0	0.2 / -	600-1000	40	70	-
UCL	21.0	0.2 / -	750-4000	45	100	-
MCL	21.0	0.2 / -	750-4000	45	100	-
DBL	21.0	0.22 / -	1000-3500	46	110	-
BL	22.0	0.22 / -	1000-3500	47	130	-

Table 1: Mechanical properties of soil and rock units found at EBR, ØVK and SLU excavation pits (Metroselskabet, 2018). FY: Fill layer, ML1: Upper clay till, ML2: Lower clay till, MS2: Glacial sand till, CL: Disturbed Upper Copenhagen Limestone, UCL: Upper Copenhagen Limestone, MCL: Middle Copenhagen Limestone, DBL: Disturbed Bryozoan Limestone and BL: Bryozoan Limestone.

3. Construction sequence

The excavations reported in this paper are rectangular with a width of approximately 20 m and length of 60-120 m; a bottom-up construction method has been used in all cases. The excavations are retained by secant pile walls with 1.2 m diameter piles, supported by ground anchors and struts in the temporary case, as shown in Figure 1 for each case. The length of the piles is 24.5 m at EBR and ØVK and 28.5 m at SLU, the primary pile spacing is 1.8 m and the bending stiffness of the wall is 1745 MNm^2. The free length of the anchors is 11 m while their fixed length is 8.5 m. The anchors have been installed at an inclination of 30° at EBR shaft and 35° at ØVK shaft and pre-stressed at a force of 1000 kN. The horizontal spacing of the anchors is 1.8 m. Circular hollow struts (steel class S355) of 1016 mm diameter and 30 mm wall thickness were installed at all case studies with spacing of 6.5 m. On the other hand, a different wall supporting system was adopted for the SLU station due to the larger excavation depth and thicker fill layer. The construction of the staircases in the southeast side of the excavation (see Figure 2, blue box) blocked the use of anchors for supporting the excavation on this side and therefore, struts were installed at 3 different levels in the east part of the excavation. The ground water control inside the excavation consists of pumping wells, where the subtracted water is recharged back to aquifer through re-infiltration wells that are situated outside the excavation. At SLU shaft, 95% to 100% of the pumped water was re-infiltrated back to the aquifer while the re-infiltration rate was eventually decreased at EBR-ØVK shafts in order to not exceed the design assumptions and the exceeding pumped water was discharged to the harbour. Bottom-up excavations started with the construction of the secant pile walls. As excavations proceeded to a lower level, new anchors or struts were installed until the final excavation depth was reached. Excavations were carried out in 3 to 4 stages. Following the completion of the excavation, the base slab was casted. It should be mentioned that the removal of the bottom struts and the casting of the base slab were carried out in sequence and in parallel. The bottom struts removal was completed few days after the completion of the base slab, which thereafter worked as bottom support. Table 2 summarizes the construction activities at EBR; however, it should be mentioned that a similar construction process has been followed in the other case studies.

Stage	Construction Event	Starting Date	End Date
1	Construction of primary secant piles	26-11-2018	14-02-2019
2	Construction of secondary secant piles	05-12-2018	19-02-2019
3	Construction of soft secant piles (between EBR and ØVK)	23-11-2018	21-02-2019
4	Excavation from level +2.20 m until -3.90 m		02-04-2019
5	Activation of the first anchor	22-04-2019	01-05-2019
6	Excavation from level -3.90m until -11.60 m		29-05-2019
7	Activation of strut	20-06-2019	09-07-2019
8	Excavation until -19.21 m		26-07-2019
9	Removal of Bottom strut	14-10-2019	13-11-2019
10	Construction of base level slab		12-11-2019

Table 2: Main construction stages of the excavation at EBR shaft (all levels in m DVR90).

4. Instrumentation

A long-term comprehensive instrumentation program has been implemented for the verification of the design assumptions, for securing the safety of the deep excavations and supervising the impact of the construction activities on the adjacent buildings and structures. Field measurements are stored in the Kronos monitoring database system that was established and used for the Copenhagen metro network (Rabensteiner et al., 2015). The horizontal displacements of the secant pile wall are monitored by inclinometer casings that are fixed to the steel reinforcement cages. The inclinometer casings were installed in both sides (longitudinal and transverse) of the excavation down to a depth of 23 to 26 m and were equipped with In Place Inclinometer (IPI) sensors with a vertical spacing of 3 m. They are recorded with an automatic acquisition unit and a reading interval of 4 hours. The anchor and temporary strut loads are monitored by load cells that are installed on anchor heads and struts respectively. Strut loads are measured by three sets of strain gauges that are installed on each side of the selected struts and recorded with a reading interval of 20 minutes. The instrumentation program also includes monitoring of ground movements which are measured by using levelling points that were installed on the ground surface in the vicinity of the excavation and outside the expected primary influence zone in order to assess the extent of the area that is influenced by excavation activities. Ground water table is also monitored continuously in piezometers inside and outside the excavation pits. In addition, water level measurements and inflow rate are available from pumping and re-infiltration wells. In the present paper, the field measurements from inclinometers, load cells and piezometers are discussed in detail. An illustration of the instrumentation layout at EBR, ØVK and SLU is shown in Figure 2.

5. Observed performance of the secant pile walls

The lateral deflections of the secant pile walls measured at ØVK shaft, from inclinometers situated at the center of the shaft (ØVK02 and ØVK08, considered representative of plane strain conditions) and for various constructions stages are illustrated in Figure 3. The measurements of the inclinometers located at the central section of the excavation on the opposite sides are consistent. As expected, the lateral wall deflections increased as the excavation proceeded. The magnitudes of the measured lateral wall deflections were lower than the 0.25% of the corresponding excavation depth and fell at the lower bound of reported ranges from similar deep excavations (Long, 2001). The relatively small wall deflections observed can be attributed to the stiff ground conditions, the high system stiffness, and the short horizontal span of the excavation.

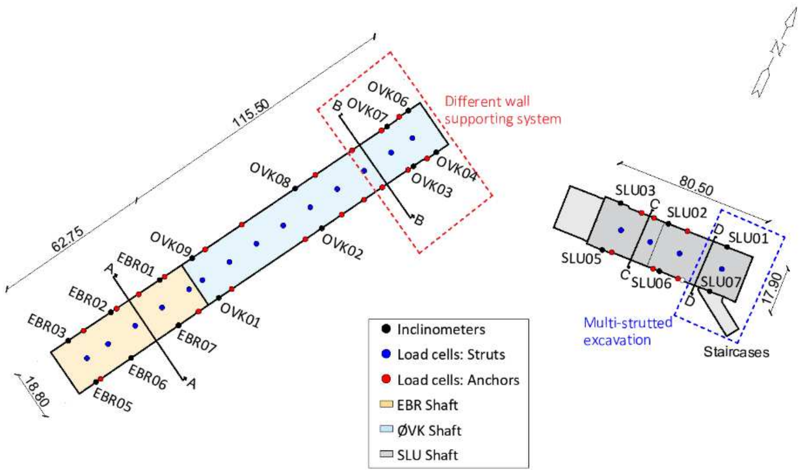

Figure 2: Plan view of EBR, ØVK and SLU excavations with the instrumentation layout, different wall supporting systems and the excavation cross sections shown in Figure 1.

A cantilever-type deflected shape is observed until the installation of the anchors as the wall starts bending after the second excavation stage and prior to the strut activation, consistently with the general pattern of wall movement reported by Clough & O' Rourke (1990). The magnitude of the anchor forces is almost constant throughout the construction stages, while strut forces increase from 1 MN to 4 MN as the excavation progresses from construction stage D to construction stage E, and larger deflections take place.

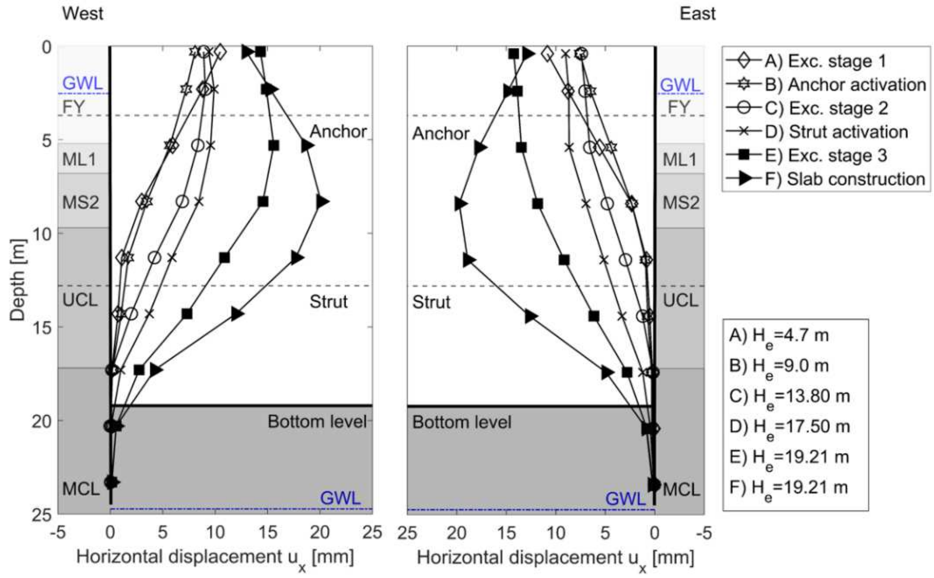

Figure 3: Lateral deflections measured on ØVK shaft by inclinometers ØVK08 (left) and ØVK02 (right) for different construction stages.

The lateral deflections of the secant pile walls at EBR station (EBR02 and EBR06, considered representative of plain strain conditions) are shown in Figure 4. Field measurements showed that the West and East side of the excavation behave differently after the installation of the anchors. A rotation within the MCL unit was observed from the measurements of the inclinometer in the East side of the excavation during the initial construction stages which is not observed in the other side of the wall. A rotation correction was therefore applied to counter this effect. It was found that after correcting the rotation within the Limestone unit the deflections in both sides of the walls are similar. As it was observed at ØVK shaft, the wall has a cantilever-type deformed shape until the installation of the anchors. The activation of the anchors at EBR shaft significantly reduces wall deflections compared to ØVK shaft which might be associated with the different anchor installation levels. The maximum lateral wall deflection is 0.195% of the corresponding excavation depth which is lower than the one observed at ØVK shaft. In general, the lateral wall deflections measured at ØVK were larger than the ones observed at SLU and EBR. The lower wall deflections observed at SLU compared to the ØVK are associated with the different supporting system that was used at SLU shaft and the shallower depth of the limestone.

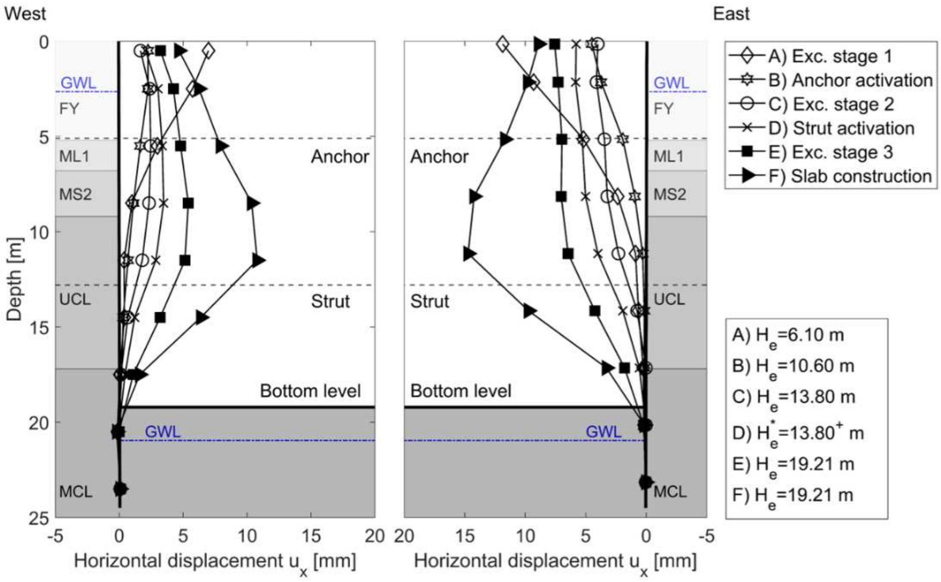

Figure 4: Lateral deflections measured at EBR shaft by inclinometers in plane strain conditions (EBR02-EBR06) for different construction stages. *Excavation depth at construction stage D could not be defined accurately, however, it is higher than 13.80 m and lower than 19.21 m.

On the other hand, the lateral wall deflections measured at EBR are approximately half of those measured at ØVK even though both shafts have similar stratigraphy and system stiffness. This is mainly attributed to the fact that higher pumping occurred as the excavation progressed at ØVK and consequently there is a bigger water head difference between the two sides of the retaining wall.

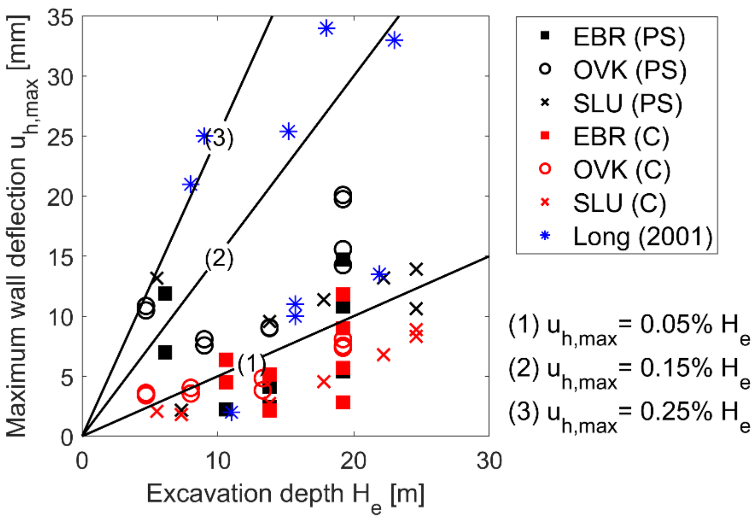

Figure 5: Maximum lateral deflection versus excavation depth for the three case studies. Results display data at different construction stages (various H_e) and from several inclinometers organized according to location in plane strain (PS) condition, and close to the excavation corners (C).

Figure 5 illustrates the maximum wall deflection measured at various construction stages with respect to the excavation depth from the three case studies reported in this paper and from reported case histories (Long, 2001) in stiff clay conditions with similar wall stiffness and support system. It can be seen that the maximum normalized deflection during the first excavation stage (Stage A) is approximately 0.25% for the three case studies. However, maximum lateral deflections decrease after the activation of the anchor (Stage B), as expected. Thereafter, the maximum deflection increases linearly with excavation depth though with a different rate, depending on each case study. The rates of increase of EBR and SLU are 0.36 mm/m excavation and 0.4 mm/m excavation respectively while the increase rate at ØVK was found to be 0.7 mm/m excavation. The measured lateral wall deflections are close to the lower boundary of the ones reported by the selected case histories from Long (2001) which is attributed to the higher wall stiffness and the presence of Limestone for at least 50% of the excavation depth at the three case studies. Smaller deflections were observed in the corners at SLU, and less apparently at EBR compared to those measured in the center of the excavation. Similar observation was made at ØVK shaft; however, this may also be attributed to the different lateral wall supporting system that was considered in the North side of the excavation.

6. Conclusions

According to the analyses of the field measurements from three deep metro station excavations in stiff over-consolidated deposits in Copenhagen, the following main findings were drawn:

i. A cantilever type deflected shape was observed during the first excavation stage and after the anchor activation at all case studies. Nevertheless, wall starts bending towards the excavation after the second excavation stage.
ii. The magnitudes of the maximum lateral deflections were smaller than 0.25% of the excavation depth at the corresponding stage and fell at the lower margin of reported ranges observed in the literature from excavations of similar dimension ratios supported by stiff retaining walls in soft and stiff clay conditions.
iii. The lateral wall deflections were observed to increase linearly with the excavation depth after the installation of the first row of anchors.
iv. Smaller deflections were measured at the corners of the shafts which could be related to arching effects, different local support system or a combination thereof.

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