Effects of Change in the Support System on Temporary Secant Pile Wall

C. Anburaj and A. Srinivas

Abstract Deep excavation with a support system is required to construct various parts of underground structures like shafts, stations, entry structures, etc. These structures have to be constructed using permanent or temporary embedded retaining walls with a support system. The selection of a suitable type of retaining wall will depend upon the geological condition present in the particular location, time, cost, available equipment, etc. In Bangalore Metro Rail Project, a secant pile wall was adopted as the temporary earth retaining system with the depth of excavation of about 20 m. It was designed initially based on bottom-up construction methodology with 3 levels of struts and 3 levels of anchors, but at a later stage, due to time and other construction-related issues, it was decided to change the configuration to 6 levels of struts. Generally, in deep excavations, all the underground structures should be designed and checked for the critical forces from both permanent stage and construction stages. Since secant piles are used as a temporary retaining wall, only construction stage analysis is carried out to get the governing forces and deformation. In construction stage analysis, soil layers are defined with boundary conditions, and the surcharge during construction and surcharge from the actual building near the secant pile are considered. This paper discusses the effect of change in the support system from anchors to struts and how these changes in the support system affect the behaviour of the secant pile and subsequently adjacent buildings present in the influence zone of the excavation. As a result, in changing from anchors to struts, the wall displacement, strut forces and ground movement on the adjacent ground and buildings increase. The increase in deflection, ground movements and strut forces occurs during backfilling sequence of the underground station excavation. During backfilling, struts have to be removed and this imparts a higher magnitude in deflection, strut forces and ground movements.

Keywords Support system · Anchors · Struts · Ground movement · Settlement

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1 Introduction

Construction of underground metro stations in most cities is growing at a faster rate in our country. The general methodology adopted in metro stations seems to be familiar, but a lot of risks are involved due to variable geological conditions. Furthermore, in the case of structures below ground, the close contact of the surrounding ground with the structure, soil–structure interaction plays a prominent role in design. As a result, there is a close interrelationship between the method and sequence of construction of the structure and its design. In other words, the design of underground metro station (cut-and-cover structures) cannot be treated separately from their construction. Therefore, the design and construction of such structures, particularly in a highly constrained urban environment and difficult ground and groundwater conditions, will present some of the greatest challenges to both the designer as well as the constructor. Therefore, for the construction of the underground metro station, a temporary retaining structure is adopted. Considering ground conditions, groundwater conditions, excavation depth, construction equipment, time and cost of construction, secant piles are proposed as temporary earth retaining structures. On observing the site conditions, the Bottom-up method of construction is followed in one of the underground metro stations in the Bangalore Metro Rail project. To retain the soil, struts and anchors are used to provide lateral support to the secant pile walls initially. Over time, due to construction issues, anchors are replaced with struts. Due to the change in the support system, the behaviour of the secant piles and the ground response will change because of the fact that the stiffness of the whole system wall will change while replacing it. Therefore, to analyze the behaviour of the secant pile wall, two support systems are considered. They are (i) 3 strut 3 anchor configuration and (ii) 6 strut configuration. This paper presents the effects on the behaviour of the secant pile wall and also the ground response when anchors are replaced by struts. The response of the adjacent buildings when anchors are changed to struts is also discussed. The changes in strut forces after changing the support system are examined and the results are presented as a part of this study.

2 Site Geology

The reduced level (RL) of the ground varies between +904.500 m and +907.838 m. The main rock formation in the Bangalore area is granitic gneiss. The granitic gneisses are mainly of migmatitic type, highly banded varying in composition from granite to diorite. Grade IV rock is encountered at about 26 m below the ground level. The type of rock generally observed is Granitic Gneiss. Rock is encountered at some locations and is not uniform throughout the station. The water table varies from 1.2 to 18 m below the ground surface based on the investigation. As per the CGWB survey, most of the locations in Bangalore have deeper water level ranging from 10 to 20 m below ground level. Hence, the fluctuation in the water table is due to the effect of the

Table 1 Geotechnical properties of the sub-soil profile

Depth (m)	Strata type	γ (kN/m^3)	E' (MPa)	c' (kPa)	ϕ'
0–2.5	Filled up	18	10	–	28°
2.5–7.5	Clay & silt of low plasticity	18.5	11 + 3.8z	26	24°
7.5–16.5		18.5	30 + 3.8z	46	25°
16.5–21	SM	18.5	64.2 + 3.8z	–	38°
21–26	SC	18.5	81.3 + 3.8z	15	23°
>26	Soft rock	18.5	136	–	40°

perched ground water table and frequent rainfall noticed during the investigation. From the ground profile, a 2 m thick layer of aquiclude retains the water and behaves as an impermeable layer. The generalized profile of the station considered is shown in Table 1.

3 Construction Methodology

It is planned to construct the underground metro station by the Bottom-up method of construction. The bottom-up method is ideally suited for the particular location because of the following reasons:

- There are no major services or utilities in the vicinity of the construction site
- Very less disturbance to the on-road traffic
- Ground movements anticipated are low and groundwater pressures are not high
- Adjacent buildings and structures close to excavation are very less in number.

In the Bottom-up method, as a first step, a capping beam is constructed which helps in connecting the secant pile wall intact from the top to the bottom of the excavation and also prevents the out of plane and lateral movement of the wall when exposed during excavation. After the capping beam acquires the required strength, a proper dewatering system is adopted to lower the water table within the station so that excavation can be carried out in a dry state. This is usually done for any excavation activity because the presence of groundwater will make excavation tedious. Hence, before progressing to every level or stage of excavation, the groundwater table has to be lowered within the perimeter of the excavation. As the excavation progresses downwards, struts or anchors are used to provide lateral support to the walls. After reaching the final or desired excavation level, construction of the structure is then carried out. The bottom slab is constructed first, which progresses upwards in a conventional manner. As the construction of the structure advances upwards and when the elements of the structure which form the permanent lateral supports are in place and operational, each corresponding stage of temporary horizontal bracing is sequentially released and removed ensuring adequate safety measures. The back-filling and surface reinstatement are finally carried out only after the roof slab and the associated support structure are in place and have attained the requisite strength to sustain the imposed loads.

3.1 Supports System

For the construction of the metro station, the depth of excavation is about 20 m below the ground level. Hence, to provide lateral support to the secant pile wall, three levels of struts and three levels of anchors are used during design. The details of the strut and anchor used during the excavation are mentioned in Table 2.

The strut and anchor levels used to retain the soil for 20 m depth are presented in Table 3. The combination of struts and anchors is chosen in such a way that it causes less hindrance to construction activity and requires optimum reinforcement for the secant piles.

However, in a later stage, due to construction issues, the last three levels of the anchor are replaced by struts and the levels are slightly altered. The fourth, fifth and sixth level of struts is raised by 1.58 m, 2 m and 1 m, respectively, from their initial anchor position.

3.2 Construction Sequence

As mentioned earlier, the bottom-up method is adopted for construction. The typical stages of the construction sequence are as follows:

Table 2 Details of strut and anchors

Type of support	Sectional details	Young's modulus in kPa
Struts	2UB 610 × 229 × 125.1	2×10^8
Anchors	7-strand of 15.7 mm diameter	2×10^8

Table 3 Levels of struts and anchors

Levels of support	Depth from existing ground level (m)
First level strut	2.5
Second level strut	6
Third level strut	9
Fourth level anchor	12.5
Fifth level anchor	15.5
Sixth level anchor	17.5

- Installation of secant piles after capping beam gains enough strength.
- Dewatering system is installed prior to excavation. Water table is maintained 1 m below excavation.
- Excavate 0.75 m below the first level strut and install the first level of strut.
- Excavation is carried out in a similar manner up to the installation of sixth level strut.
- After reaching the final excavation level, the base slab is constructed against secant pile wall with proper waterproofing layers.
- After casting the base slab, the station wall is constructed with internal elements after the removal of corresponding struts.
- Construct the concourse and roof slab in a similar way after removing the respective struts and anchors. Backfilling is done between the sides of the secant pile and permanent walls.
- Reinstating the ground to the original level by backfilling.

4 Finite Element Analysis

In general, finite element analysis is used to predict the behaviour and response of the temporary or permanent retaining structures in a robust manner. Even though the secant piles are temporary structures, the analysis is carried out for the construction stage and permanent stage, i.e., undrained parameters of soil for the temporary stage and drained parameters for the permanent stage. Therefore, Secant piles, struts and anchors are designed for the critical case scenario. The finite element analyses were carried out using WALLAP software to predict the deflection of secant pile wall, strut and anchor forces. The properties of secant piles, struts and anchors used in the analysis are given in Table 4.

The construction sequence as said above is exactly modelled in WALLAP and the distribution of bending moments, shear forces and deflection of the secant pile is obtained for both the cases, i.e., with anchors and anchors replaced by struts. The change in the strut forces in the first three levels of struts, due to the replacement of anchors, is also observed. Similarly, the ground movements due to changes in the support systems are also predicted using Clough and O'Rourke (1990) method. The forces on struts, ground movements and tensile strain induced in the adjacent buildings due to changes in the support system are presented in the subsequent sections.

Table 4 Material properties used in WALLAP

Element type	I (m^4/m)	Cross-sectional area (m^2)	E (kPa)	Spacing (m)
Secant pile	0.014893	0.5026	2.74×10^7	1.35
Struts	–	0.03186	2×10^8	9.0
Anchors	–	0.000973	2×10^8	2.7

5 Estimation of Ground Movements and Tensile Strain of Adjacent Buildings

Ground movements can be computed by the principles given by Bowles (1990) and Clough and O'Rourke (1990) depending on the type of soil. Based on several case histories, Clough and O'Rourke (1990) suggested that the settlement profile is triangular for excavation in sandy soil or stiff clay. The maximum ground surface settlement will occur just behind the wall. The non-dimensional profiles are given in Fig. 1. It shows that the corresponding settlement extends to about $2H_e$ and $3H_e$ for sandy soil and stiff to very hard clays, respectively, where H_e is the influence depth.

As the excavation progresses, the lateral pressures imposed by the ground behind the wall would induce wall deflections into the excavation. This would result in vertical and lateral displacements of the ground adjacent to the retaining wall. In principle, the magnitude and extent of this ground movement are a function of the retention system type, the adopted construction methodology and the properties of the soil and/or rock materials. The depth of influence (H_e) is considered depending on the depth of the secant pile wall and the depth of the excavation. The geology of Bangalore is predominantly mixed soil condition comprises mainly of mixtures of silty sand and clays with low to high plasticity and compressibility. The maximum deflection on the ground adjacent to the secant pile due to excavation at the launching shaft can be estimated with the deflection profile of the secant pile. As the strata are generally mixed, the typical settlement profile just behind the secant pile wall as shown in Fig. 2.

The ground settlement curve is taken as "second degree exponential curve" as suggested by Bowles (1990) where the maximum ground settlement occurs just behind the wall. Bowles (1990) suggested a procedure to estimate the excavation-induced ground surface settlements using the following relations:

$$\delta_v = \delta_{vm}\left(\frac{l_x}{D}\right)^2 \tag{1}$$

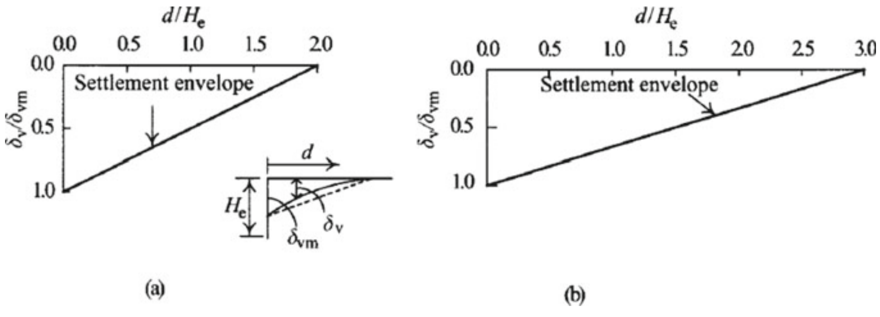

Fig. 1 Dimensionless Settlement profiles adjacent to Excavation **a** sandy soil **b** stiff clays (Clough and O'Rourke 1990)

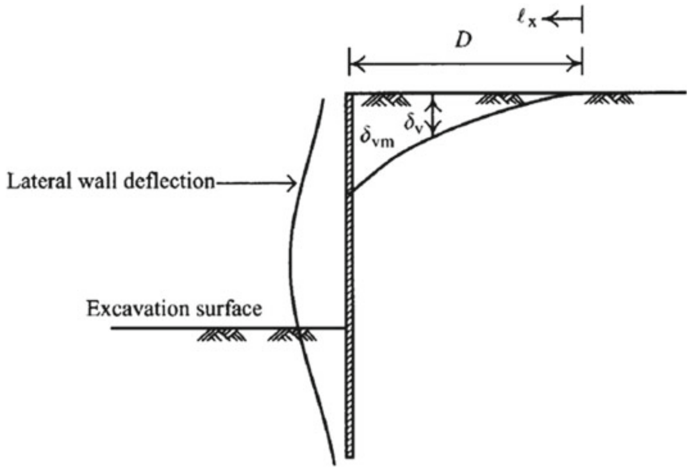

Fig. 2 Estimation of ground settlement (Bowles 1990)

where δ_v = settlement at a distance of $D - l_x$, δ_{vm} = maximum ground surface settlement, l_x = Distance from a point at a distance D from the wall and D is the influence range of ground surface settlement.

The maximum ground surface settlement δ_{vm}, is estimated from the following equation:

$$\delta_{vm} = \frac{2A}{D} \tag{2}$$

where A is the area of the lateral wall deflection. The lateral deflection of the secant pile wall is obtained using finite element software, WALLAP.

5.1 Limiting Tensile Strain Method

Burland and Wroth (1974) and Burland et al. (1977) applied the concept of limiting tensile strain to elastic beam theory to study the relation between building deformation and the onset of cracking. Although modelling a building as an elastic beam is a simplification, it was found that predictions from this model were in good agreement with case records of damaged and undamaged buildings. Furthermore, this simple approach demonstrates the mechanisms which control the onset of cracking within a structure.

The elastic beam in their model is described by a width, B and a height, H (see Fig. 3). The figure shows two extreme modes of deformation: In bending, cracking is caused by direct tensile strain, while in shear diagonal, cracks appear, caused by diagonal tensile strains. For a centrally loaded beam subjected to both shear and bending deformation, the total central deflection is given by Timoshenko (1955)

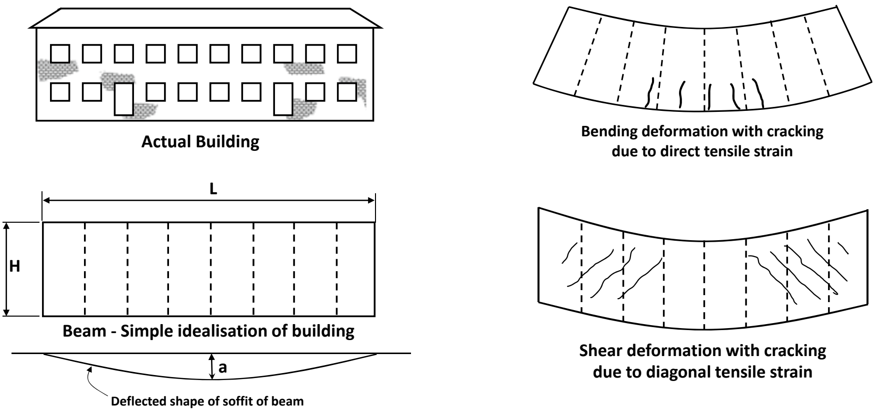

Fig. 3 Cracking of a simple beam in different modes of deformation (Burland and Wroth 1974)

$$\Delta = \frac{PB^3}{48EI}\left(1 + \frac{18EI}{B^2HG}\right) \tag{3}$$

where E—Young's modulus

G—Shear modulus

P—Point load, which is applied at the centre of the beam.

For an isotropic elastic material, E/G = 2(1 + ν). Assuming a Poisson's ratio of ν = 0.3, one obtains E/G = 2.6. In the case where the neutral axis is in the middle of the beam, Burland and Wroth (1974) expressed the above equation in terms of deflection ratio and the maximum extreme fibre strain ε_b

$$\frac{\Delta}{L} = \frac{\varepsilon_v}{3}\frac{\left(1 + 6\frac{E}{G}\frac{H^2}{L^2}\right)}{\left(1 + 4\frac{E}{G}\frac{H^2}{L^2}\right)} \tag{4}$$

$I = H^3/12$ for sagging zone ($t = H/2$ in sagging zone).

$I = H^3/3$ for hogging zone ($t = H$ in hogging zone).

$$\varepsilon_b = \frac{(\Delta/L)}{\left(\frac{L}{12t} + \frac{3EI}{2tLGH}\right)} \tag{5}$$

$$\text{Diagonal strain } \varepsilon_d = \frac{(\Delta/L)}{\left(1 + \frac{GHL^2}{18EI}\right)} \tag{6}$$

The horizontal strain ε_h is calculated as mentioned in Fig. 4 and is given by the following expression:

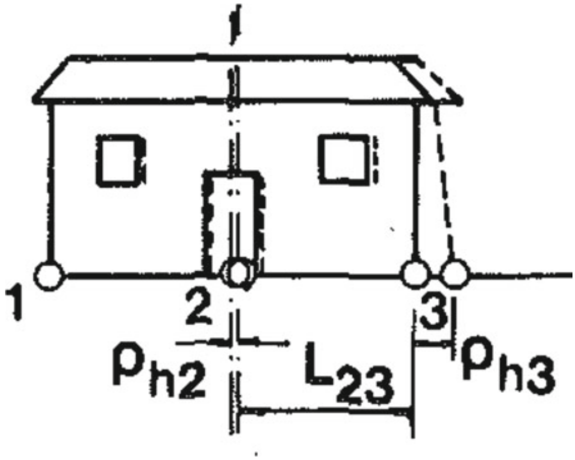

Fig. 4 Typical lateral movement of the buildings near excavation (Boscardin and Cording 1989)

$$\varepsilon_h = \frac{\Delta_{h1} - \Delta_{h2}}{L} \tag{7}$$

where Δ_{h1} and Δ_{h2} are the lateral movement of the building at the ground surface and L is the length of the building in the influence zone of excavation. The limiting tensile strain in the building (ε_t) is the maximum combined horizontal bending strain or combined horizontal and diagonal shear strain.

$$\varepsilon_t = Maximum\ of \left\{ (\varepsilon_h + \varepsilon_b), \frac{1-\nu}{2}\varepsilon_h + \sqrt{\left(\frac{\varepsilon_h(1+\nu)}{2}\right)^2 + \varepsilon_d^2} \right\} \tag{8}$$

In the above equation, when Poisson's ratio (ν) is taken as 0.3, then the resulting equation is given below

$$\varepsilon_t = Maximum\ of \left\{ (\varepsilon_h + \varepsilon_b), 0.35\varepsilon_h + \sqrt{(0.65\varepsilon_h)^2 + \varepsilon_d^2} \right\} \tag{9}$$

6 Results and Discussion

Results of numerical analysis from WALLAP considering the three levels of struts and three levels of anchors with six levels of struts show that the deflection of the secant pile is high when six levels of struts are used even though the stiffness of strut is higher than that of anchors. This is due to the fact that, while removing or backfilling sequence, i.e., after construction of the base slab, the internal horizontal struts have to be removed to proceed with the construction further. In this scenario, the secant pile wall is subjected to a maximum cantilever span. However, when anchors are there, these anchors will not hinder the construction of internal elements. Hence, these anchors need not be removed during backfilling sequence, but de-stressing is required, i.e., load at the anchor head will be removed during backfilling and not the entire anchor. The deflection profile of the secant pile for both the cases is presented in Fig. 5.

The maximum deflection of the secant pile is only in six strut configuration which yields 7 mm more than the deflection in strut and anchor configuration. However, the deflection pattern of the secant pile wall seems to be identical. The increase in deflection due to the change in the support system is about 13% from the original configuration.

Due to the replacement of anchors by struts, the forces estimated on the first three levels of struts increase. This is because during backfilling, the unsupported length of secant piles would have caused the struts at the top to consume more force. In addition, the struts are internal compressive members, and when secant piles move inwards, the forces on the struts will increase. The variation in strut forces due to change in the support system is mentioned in Table 5.

From Table 5, it is observed that the first level strut is subject to very less increment when compared to the remaining two levels. From this, we can conclude that due to the change in supports at lower levels, there will not be much variation for the supports at the top level. Nevertheless, the struts, which are above the anchors, are experiencing a significant increase in the strut forces. As suggested by Bowles (1990),

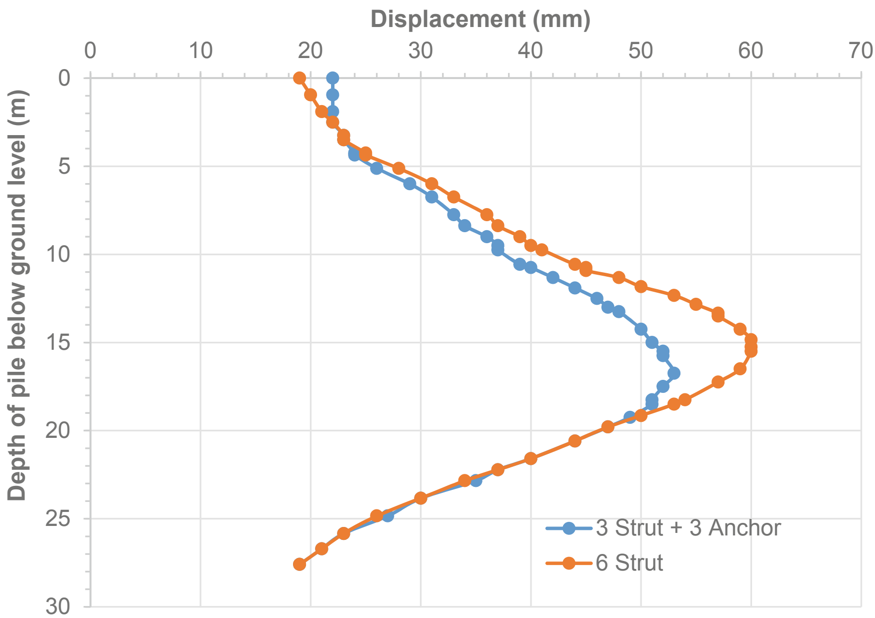

Fig. 5 Deflection profile of secant pile wall

Table 5 Increase in strut forces due to change in support system

Strut levels	Increase in strut forces (%)
1st level strut	5
2nd level strut	10
3rd level strut	17

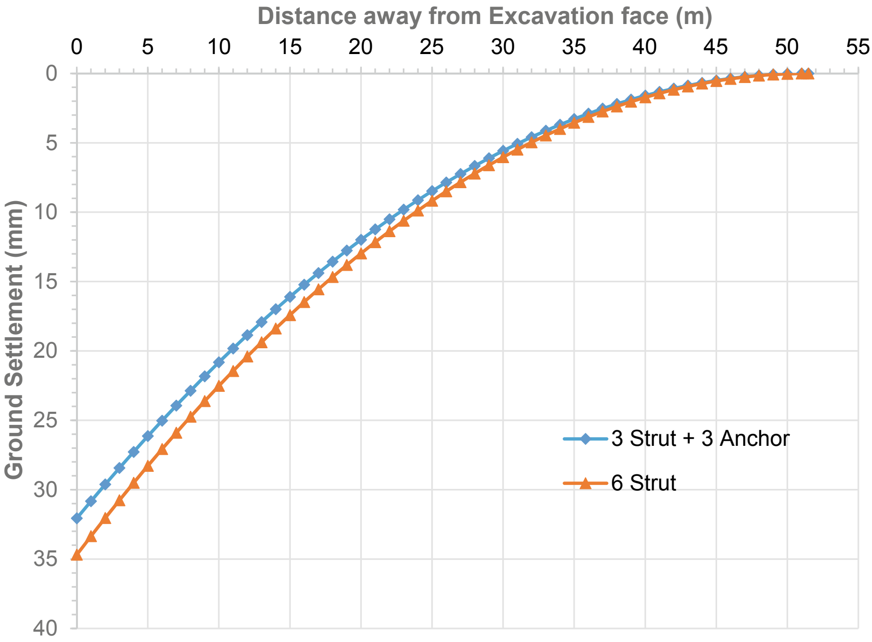

Fig. 6 Ground settlement profile behind the secant pile wall

the ground settlement depends on the lateral movement of the retaining structure. When the deflection of secant piles is high, the maximum ground settlement behind the secant piles will also be high. Figure 6 represents the ground settlement profile behind the secant pile wall.

From Fig. 6, the ground settlement behind the secant pile wall shows that there is an increase in settlement by 8% when struts replace anchors. Similarly, the buildings, which fall within the influence zone of the excavation, will experience differential settlement due to variation in the ground settlement. Due to differential settlement, rotation of the building may occur. Theoretically, the rotation of the buildings can be calculated with the help of the ground settlement profile. There are two buildings which are located at a distance of 3 m and 13 m from the excavation boundary. The details of the building in the zone of influence are mentioned in Table 6.

As the ground settlement follows parabolic distribution, the settlement of the building varies along the length of the building. The rotation of the buildings for

Table 6 Building details near the underground station

Building ID	Dimension	Type of building	Distance from excavation boundary (m)
B1	38 m × 58 m	RCC Framed structure	3
B2	32 m × 34 m	RCC Framed structure	13

Table 7 Summary of building rotation

Building	Building rotation	
	Three struts and three anchor	Six struts
B1	1 in 1960	1 in 1810
B2	1 in 1695	1 in 1565

both the cases is presented in Table 7. The buildings tend to rotate more in a six strut configuration as the distribution of ground settlement is slightly higher than in strut and anchor configuration.

7 Summary

Numerical results from WALLAP shows that the increase in deflection of the secant pile wall is due to a change in the support system from three levels of struts and anchors to six level of struts. The increase in secant pile deflection is about 13% but the deflection pattern looks the same for both the cases. Similarly, the increase in strut forces for the first three levels of struts is about 5%, 10% and 17%, respectively. The increase in the strut force is due to a change in the stiffness of secant piles and the supports. Having the cross-sectional area of the struts very high when compared to anchors, the increase in the deflection and strut forces is because of the restraint provided by the struts. The moment when anchors are replaced by struts, it imparts too much resistance and support to the secant piles. During backfilling sequence, the deflection of secant piles increases when every level of the strut is removed, which in turn increases the forces on the other struts. In a similar fashion, the ground settlement behind the secant piles also increases when the deflection of the secant piles increases. When anchors are replaced by struts, an 8% increase in the maximum ground settlement is observed due to the movement of secant piles. On the other hand, if the buildings are near the influence zone, the rotation of those buildings is high when struts replace anchors. If the struts are placed at the same level where anchors are there, then the response of secant piles may vary and the ground response may be the same for both the cases. However, in the present study, there are certain limitations like stresses induced, potential cracks developed and tensile strains induced in the building during excavation are not accounted in this study.

References

Boscardin MD, Cording EJ (1989) Building response to excavation-induced settlement. J Geotech Eng, ASCE 115(1):1–15

Bowles JE (1990) Foundation analysis and design, 4th edn. McGraw-Hill book company, New York, USA

Burland JB, Wroth CP (1974) Settlement of buildings and associated damage. Proceedings of Conference on settlement of structures. Pentech Press, London, England, pp 611–654

Burland JB, Broms BB, de Mello VFB (1977) Behaviour of foundations and structures. Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, vol 2. Tokyo, Japan, pp 495–546

Clough GW, O'Rourke TD (1990) Construction induced movements of in-situ walls. Specialty conference on design and performance of earth retaining structures, ASCE special publication No. 25:439–470

Timoshenko SP (1955) Strength of Materials, Part I, Elementary Theory and Problems, D. Van Nostrand Company, 3rd Ed.