

BUCKLING OF PILES DUE TO LATERAL SOIL MOVEMENTS



Fellenius, B.H., 1972. Buckling of piles due to lateral soil movements. Proceedings of the 5th European Conference on Soil Mechanics and Foundation Engineering, Madrid, April 10-13, Vol. 2, pp. 282-284.

BUCKLING OF PILES DUE TO LATERAL SOIL MOVEMENTS

Bengt H. Fellenius, Swedish Geotechnical Institute
Bengt Johansson, Sven Tyren AB

The following is a contribution with reference to Fig. 7.2 in the State-of-the-Art Report on Stability of flexible Structures, Piles and Pile Groups by B. Broms. It deals with the consequence of surcharging the ground outside a building founded on piles in soft soil.

Ten years ago a two storey building in Huddinge near Stockholm in Sweden was founded on 20 m long piles driven through a soft clay undergoing consolidation. The piles were square, 250 mm by 250 mm, precast concrete piles with four longitudinal reinforcement bars 12 mm in diameter. Generally, the soil in the area consists of an upper layer of mud and peat on 20 m thick layers of soft clay. The undrained shear strength of the clay is about 1 ton/m² and that the water content varies considerably. A representative soil profile is shown in Fig. 1.

The shear strength is unreduced. In Swedish practice the obtained shear strength values are reduced when the liquid limit exceeds 80 % as shown in Table 1.

Table 1. Reduction of undrained shear strength according to the liquid limit. (The liquid limit is determined by the Swedish fall cone test, Karlsson 1961).

Liquid limit (%)	Reduction factor
< 80	1.0
80 - 100	0.9
100 - 120	0.8
120 - 150	0.7
150 - 180	0.6
> 180	0.5

A rough estimate of the maximum fill height which is stable using an undrained shear strength value of $\tau_{fu} = 1.2 \text{ tons/m}^2$ (12 KPa) for

the underlying clay and a reduction factor of 0.7 gives the following relation

$$\text{Eq. 1} \quad h = \frac{5.5 \times \tau_{fu}}{\gamma} = \frac{5.5 \times 0.7 \times 12}{18} = 2.5 \text{ m}$$

A 2.5 m fill height results in a stress of about 4.5 tons/m² (45 KPa) on the ground

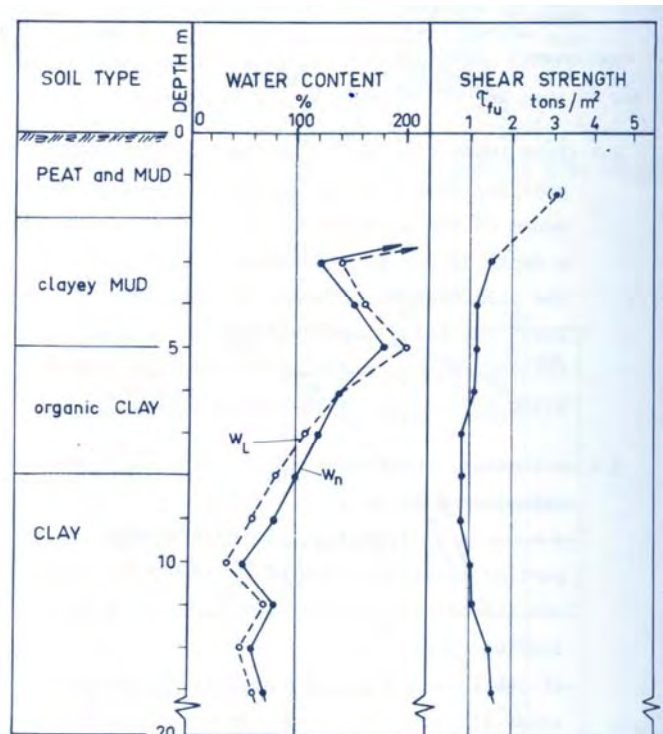


Fig. 1 Soil profile

The shear strength is determined by the Swedish fall cone method and is unreduced.

The consequence of surcharging the ground outside a pile supported building is illustrated in Fig. 2. The surcharge has been increased continuously to compensate for settlements caused by a groundwater lowering in the area and by the previous surcharge.

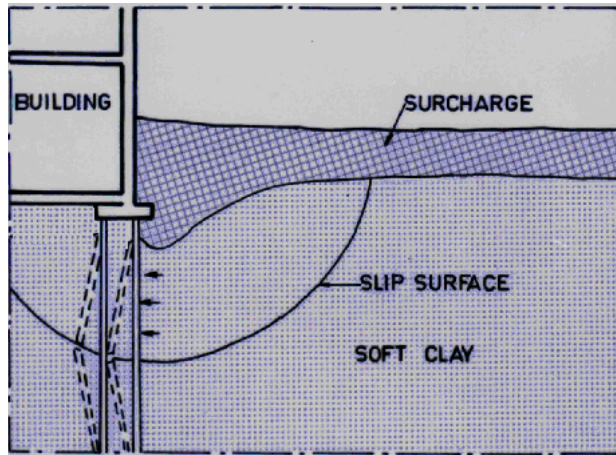


Fig. 2 Problem Illustration

The pile caps were placed approximately 2 m below the original ground surface. Over a wide area around the building, the upper peat layer (Fig. 1) was removed and replaced by sand. An additional surcharge of sand was then placed on the first fill elevating the ground surface by 1 m. Considering the removal of the peat this surcharge gave an increased vertical stress in the soil outside the building of about 2.6 tons/m^2 (26 KPa). Due to excavation for the basement, the soil underneath the building was at the same time unloaded by about 1 ton/m^2 (10 KPa). Together, this resulted in an initial stress imbalance of 3.6 tons/m^2 (36 KPa).

Due to the combined effect of the sand surcharge and a lowering of the groundwater level by about 1 m, the ground surface settled. This settlement was continuously compensated, or reclaimed, by adding more surcharge to the ground outside the building, adding stress on the clay of about 2 tons/m^2 (20 KPa) to a combined stress imbalance of 5.6 tons/m^2 (56 KPa). Naturally, the settlements increased. Today (1972), the magnitude of settlement ranges from 1 m to 2 m. Moreover, as indicated by Eq. 1, this load is more than the soil can carry and troubles have appeared as illustrated in Fig. 2.

Cracks and settlement appeared about four years ago, and the building was underpinned by Mega-piles as it was falsely assumed that the original piles were badly installed and had

settled. However, the underpinning did not arrest the settlement. Fig. 3 shows a pile cap and one of the Mega-piles. Settlements and tilting are obvious. For further investigations, a few caps were removed and the piles underneath them were excavated inside large steel-tubes, enabling the piles to be studied in situ.



Fig. 3 The settled pile cap and Mega Pile

Fig. 4 shows a photo of a pile that was inclined toward the building. After exposing the pile, it was found broken some 4 m below the cap.



Fig. 4 A ruptured pile excavated in-situ

Fig. 5 indicates the mode of rupture: failure in a combination of bending and shearing due to the lateral thrust from the soil. After the rupture, the upper part of the pile was forced down by the weight of the cap causing the reinforcement bars to buckle.



Fig. 5 Detail of the rupture

The remedy in this case has been to remove all the fill and to restore the ground floor by a deck supported on slender steel piles. Thereafter, the building will again be underpinned by Mega-piles.

References

Karlsson, R., 1961: Suggested improvements in the liquid limit test with reference to flow properties of remolded clays. Fifth Int. Conf. on Soil Mechanics and Foundation. Engineering., Paris Vol. 1, pp 171 - 184.

