A Follow-up of Two Different Foundation Principles

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ABSTRACT

In a special lecture held by the senior author at the Int. Conf. on Case Histories in Geotechnical Engineering in St. Louis (Hansbo, 1984) a follow-up was presented of the observations on two nearby buildings founded on deep deposits of soft clay according to different foundation principles. In one case the total load of the building is carried by friction piles with a factor of safety against failure equal to 3 while, in the other case, the load is partly carried by contact pressure at the raft/soil interface and partly by friction piles with a design load equal to creep failure. The presentation included about two years of observations of settlements, pile loads and contact stresses. In this paper a brief recapitulation of the foundation circumstances and the design of the two buildings will be made and the results of another 14 years of observation will be analysed. The observations include, besides the settlement distribution over the building areas, settlement at various depths, the pile loads and contact stress distribution and the excess pore pressure dissipation.

KEYWORDS

Settlement-reducing piles. Friction piles. Piled rafts. Long-term consolidation settlement. Settlement distribution.

INTRODUCTION

In traditional design of piled foundations the load of the building is carried by piles with a high factor of safety against failure—according to the Swedish building code equal to 3. In the new Eurocode (which will replace the national codes of the member countries in the European Union, EU) great importance is attached to the ultimate limit state of piled foundations and, therefore, in practice, the design according to Eurocode will most probably be carried out in very much the same traditional way as described but with other definitions on the safety concepts. In this paper, the longterm behaviour of two buildings designed according to two different design concepts is presented. One of the buildings is founded on piles according to traditional design. The other building is carried partly by contact pressure at the soil/raft interface and partly by creep piles, i.e. piles in a state of creep failure.

PRINCIPLE OF CREEP PILES

In practice, many cases occur where the settlement requirements on a building necessitate installation of piles. The piles are then usually designed to carry the total load of the building. The idea behind the use of creep piles is to benefit by the fact that a certain percentage (Q_1) of the total load (Q) of a building, settlement requirements fulfilled, can be carried without piles. The remaining part of the load ($Q-Q_1$) that would cause unacceptable settlement has to be carried by a pile system arranged in the way that the settlement obtained under full load (Q) is within allowable limits. Let us, for example, assume that the load that can be carried by a pad footing or a raft without leading to excessive settlement represents 80% of the total load. Then the remaining 20% of the load has to be carried by piles. Now the object of the design of the piles is to achieve a load *vs.* settlement behaviour for the total load that is in agreement with the load settlement behaviour of the unpiled footing (raft) itself when subjected to 80% of the total load. Usually, the piles can be designed so as to carry the load in a state of creep failure.

If we have to deal with a raft foundation on soft clay, the stresses induced by the combined action of contact stresses at the soil/raft interface and piles should preferably not exceed the preconsolidation pressure. If this is not possible the imposed stresses exceeding the preconsolidation pressure should be transferred to the soil at a depth which is most advantageous from the settlement point of view. In order to govern the load sharing between raft and piles, the piles should be designed so as to carry their load in a state creep failure. The term 'creep failure' is used in the sense that the load carried by the piles should remain constant in the course of settlement. This is important since a decrease in pile resistance in the course of settlement would impose contact stresses exceeding those assumed in the analysis and, consequently, lead to possible unacceptable consolidation settlements. In the case of a piled raft foundation, the piles should be placed in a way to reduce differential settlement as much as possible. Therefore it is important to analyse carefully the load transfer into the soil. The influence of the rigidity of the superstructure has to be taken into account. This has certainly been made easier by the use of computers and advanced numerical models but it is still quite complicated and the accuracy in the result obtained is very much dependent on the complexity of superstructure and on the subsoil characteristics.

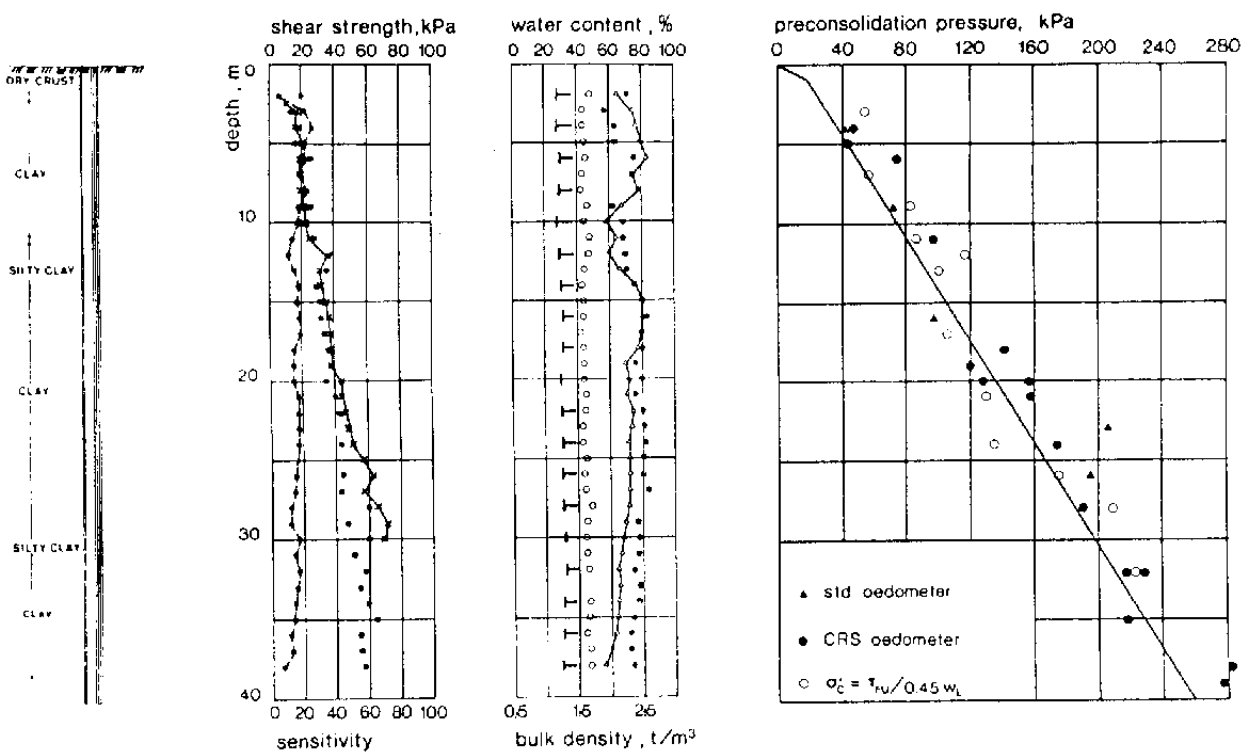
The above design concept has been applied successfully in practice (Hansbo, 1984, 1993; Burland, 1986; Jendeby, 1986, 1996; Svensson, 1991; Randolph & Clancy, 1993). The concept is also strongly backed up for piled footings on granular soil (Phung, 1993; Hansbo, 1993).

CONDITIONS

In Gothenburg, Sweden, in the late seventies, a new residential area was to be constructed. After some persuasive talk with the clients and the authorities, and with financial support from the National Board for Technical Development (STU), it was decided to make a comparative study between the results obtained according to traditional design of piled buildings on clay subsoil and according to the new concept of design utilising creep piles. The results of this comparison was published, as mentioned, at the 1984 International Conference on Case Histories in Geotechnical Engineering in St. Louis by the senior author (Hansbo, 1984).

Soil conditions

The subsoil at the site of the buildings (Fig. 1) consists of soft, high-plasticity clay underlain by sand and gravel on till or on bedrock. The thickness of the clay layer underneath the buildings varies between 35 and 55 m. The undrained shear strength, determined by the field vane test, varies almost linearly from about 15 kPa below the dry crust to about 70 kPa at a depth of 30 m. The sensitivity is 10–20. The liquid limit and the water content vary



Legend:
- undrained shear strength determined by the field vane test
- undrained shear strength determined by the fall-cone test
- sensitivity determined by the fall-cone test
- natural water content
- liquid limit determined by the fall-cone test
- plastic limit
- bulk density

Fig. 1. Soil conditions at the Olskroken site

between about 60% and 80%. The effective overburden pressure varies almost linearly from 20 kPa at a depth of 2 m to 260 kPa at a depth of 40 m. The clay can be considered to be normally consolidated. The virgin compression ratio $CR = C_c/(1 + e_0)$ increases almost linearly with depth from 0.4 to 0.65.

Before clearance, the sector of Gothenburg in which the buildings were to be erected was densely built over with 3-storey houses constructed with the bottom storey of stone and the upper storeys of wood. The lighter parts of these houses were founded on wooden mats, the heavier parts (chimneys, fire walls and stairwells) on short, closely spaced wooden piles.

Building description

Two similar residential buildings with similar loading conditions were selected for this study (for detailed description, see Jendeby, 1986).

The building founded on conventionally designed friction piles, in the following named *building 1*, is a 4-storey concrete building with a bottom area of 50 m by 14 m. Space for water pipes and sewers is left below the bottom floor. The foundation beams are cast *in situ*. As to the rest, the house is constructed of prefabricated concrete elements. The house creates a load of 66 kN/m^2 on the average (a total load of 46.4 MN). The excavation for the basement corresponds to an unloading of 44 kN/m^2 and, hence, the net load is 22 kN/m^2. The house is founded on wooden piles, 18 m in length, spliced on top with concrete piles, 10 m in length and 275 mm in width (square cross section). The number of piles installed, 211 in total, yields a threefold safety against short-term pile failure.

The building founded on a piled raft designed according to the creep pile concept—in the following named *building 2*—situated opposite to building 1, on the other side of the street, is also a 4 storey building but its bottom area is larger, 75 m by 12 m. The building is completely cast *in situ*. In the basement a space is left for sewers and water pipes and also for an air raid shelter. The raft consists of a reinforced, watertight concrete plate, 0.4 m in thickness. The basement is provided with transverse concrete walls with a spacing of 3.6 m whence the basement as a whole can be considered as quite rigid. The house is somewhat lighter than the other one; average load equal to 60 kN/m^2. The excavation for the basement corresponds to an unloading of 51 kN/m^2 and, thus, the net load on the underground is 9 kN/m^2. The piles installed consist of spliced wooden piles, 18 m in length, with concrete piles on top, 8 m in length and 300 mm in diameter. The creep failure of these piles, estimated on the basis of the creep strength of the soil (about 70% of the conventionally determined undrained shear strength) was determined to 330 kN. The total number of piles is 104, having a total carrying capacity of about 62% of the total load of the building. The piles are placed under the basement walls only.

RESULTS OF THE FOLLOW-UP

A follow-up of the behaviour of the two buildings has been undertaken ever since the termination of the building activities. This implies an observation period of about 15 years, long enough to make possible a reasonable prediction of the long-term end result.

Monitoring system

The buildings were monitored with pile load cells (flat jacks, 40 mm in height and 350 mm in diameter, filled with oil) placed on top of the piles and with Glötzl cells to measure the contact stresses. Settlement gauges (brass nails) were installed in the basement in order to study the total and differential settlements of the building. Bellows hoses and piezometers were also installed to measure the settlement and excess pore pressure distribution with depth.

Settlement

The average long-term settlements of the buildings is presented in Fig. 2 and the settlement distribution, 13 years after the termination of the construction, in Fig. 3. As can be seen, the course of settlement s is very nearly the same for the two buildings. Assuming that the continued course of settlement in the s/log$t$ presentation, Fig. 2, is a straight line, the settlement after 100 years would equal around 50–55 mm A regression analysis, based on the assumption of a linear correlation between time (t) and ratio of time to settlement (t/s) yields $s = (0.0204 + 0.0315/t)^{-1}$ for *building 1* (coefficient of correlation 0.998), and $s = (0.0206 + 0.0540/t)^{-1}$ for *building 2* (coefficient of correlation 0.994), where time t in years ($t \geq 2$ years) and settlement s in mm. This yields a long-term average settlement of about 50 mm for both buildings (49 mm for *building 1* and 48 mm for *building 2*.). Also the differential settlements up to now are nearly equal, the total settlements varying between 36 and 50 mm for *building 1* (average 44 mm) and between 32 and 48 mm for *building 2* (average 41 mm).

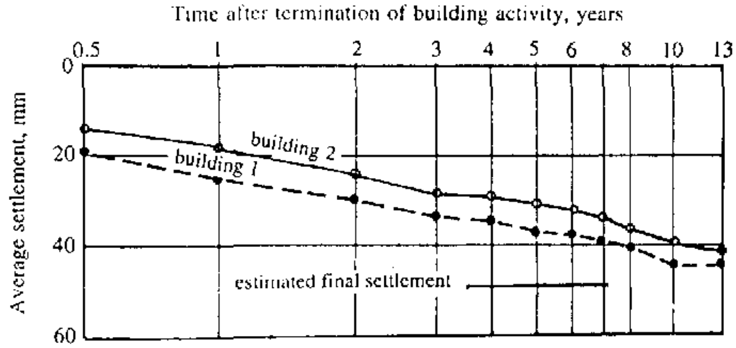


Fig. 2. The course of average settlement of the two buildings

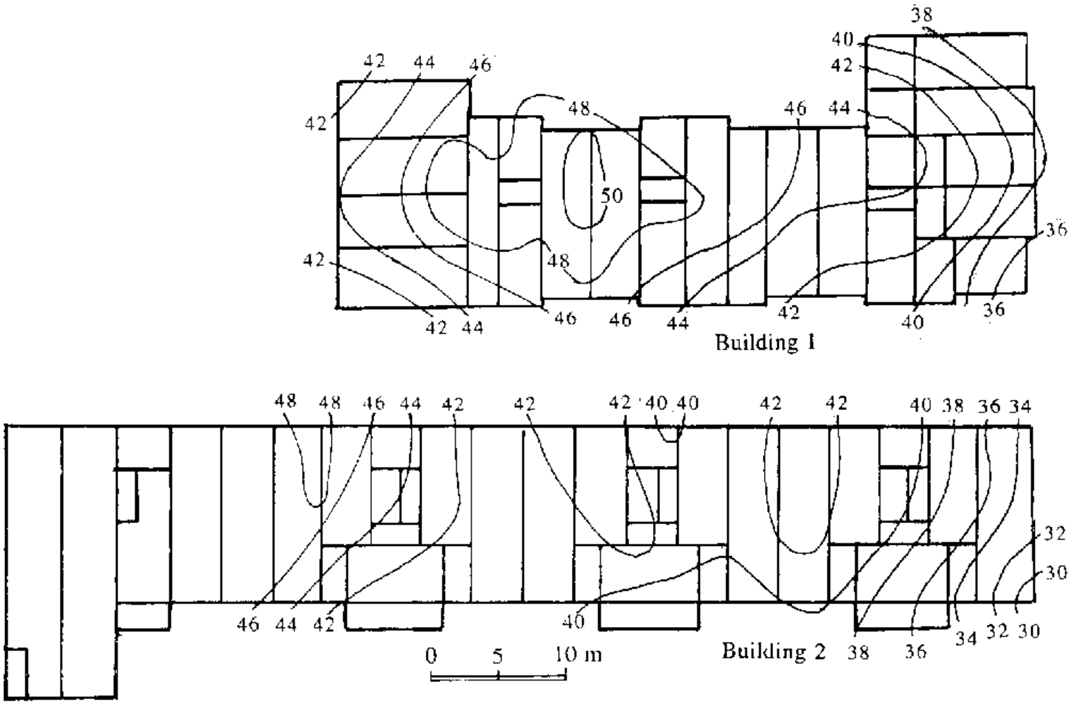


Fig. 3. Settlement distribution (in mm) observed in 1995.

The observations of the settlement distribution with depth show that nearly all settlement below *building 1* is achieved by compression of the soil below a depth of 10–15 m, Fig. 4. From this depth downwards the settlement varies linearly to zero at a depth of 35–40 m. Three quarters of the total settlement has occurred by compression of the soil above the pile tip level. As regards *building 2* nearly all settlement is achieved by compression of the soil below about 8 m, Fig. 5. From this depth downwards the settlement decreases linearly to zero at a depth of 25–30 m. All settlement has taken place by compression of the soil above the pile tip level.

It is interesting to note that the average settlement obtained for the net load 9 kN/m^2 agrees with the estimated final settlement on the basis of observations under the assumption that $CR \approx 0.050$, i.e. 9–10 times lower than the average virgin compression ratio (which seems reasonable for overconsolidated clay).

Pile loads

The average pile load, Fig. 6, was found equal to about 150 kN for *building 1* immediately after the termination of the construction period in 1982 and has then decreased successively with time, with some variations, to about 120 kN in 1995. A clear tendency can be observed towards a redistribution of pile loads. Thus, the piles along the outer walls are subjected to an increase in load while the piles in the internal part of the building show a decrease in load. The average pile load corresponding to the weight of the building is equal to 210 kN, i.e. 90 kN higher than the observed

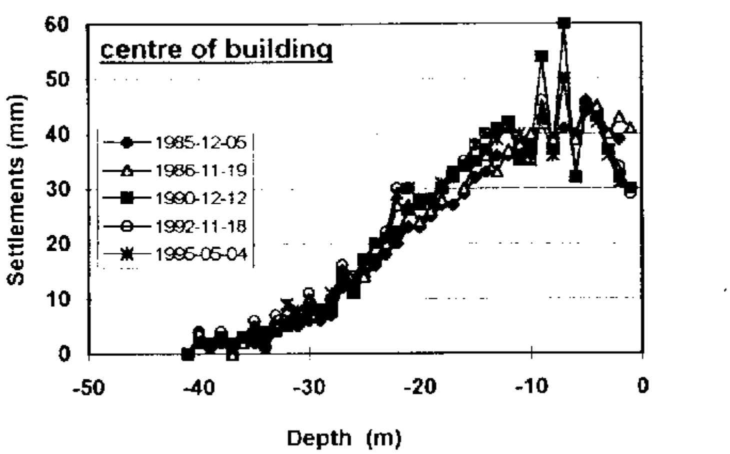


Fig. 4. Vertical settlement distribution with depth in building 1.

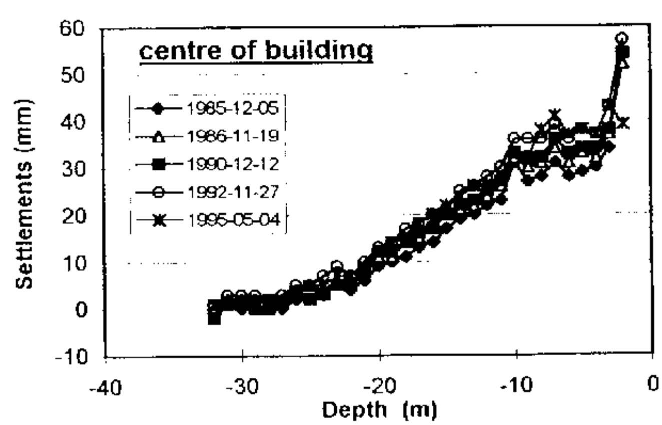


Fig. 5. Vertical settlement distribution with depth in building 2.

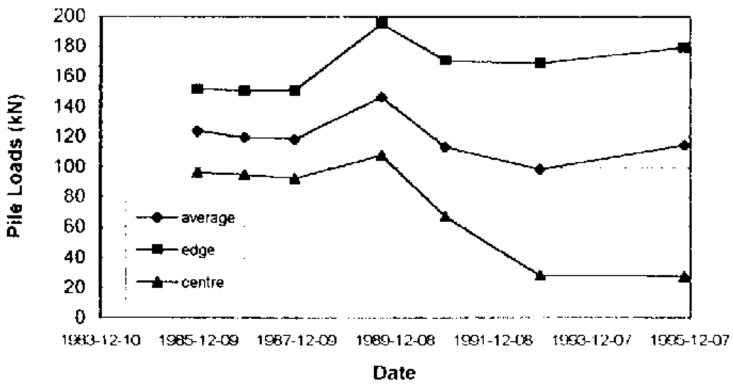


Fig. 6. Pile loads in the centre of building 1

average. As regards *building 2* all the piles observed, along the outer walls as well as in the centre of the building, have nearly the same load, varying with time between 280 kN and 400 kN (average about 320 kN). No tendency towards long-term increase or decrease in pile loads can be noticed.

Contact pressure

As regards contact pressure, a maximum of 10 kPa was observed in *building 1* during casting of the foundation beams (equal to the dead weight of the concrete), decreasing to a maximum of 4 kPa when the concrete had set, and then, during erection of the framework, slowly decreasing to zero. The contact pressure at the soil/raft interface in the centre of *building 2* has remained more or less constant with time and amounts to about 50 kPa, i.e. very nearly equal to the stress decrease due to excavation for the basement (51 kPa). Below the front wall it has decreased from about 35-40 kPa immediately after the termination of the construction period in 1982 to a constant value of about 25 kPa during the last 10 years, Fig. 8.

Excess pore pressure distribution

Unfortunately, the results of the pore pressure observations are dubious. Some of the piezometers are obviously malfunctioning

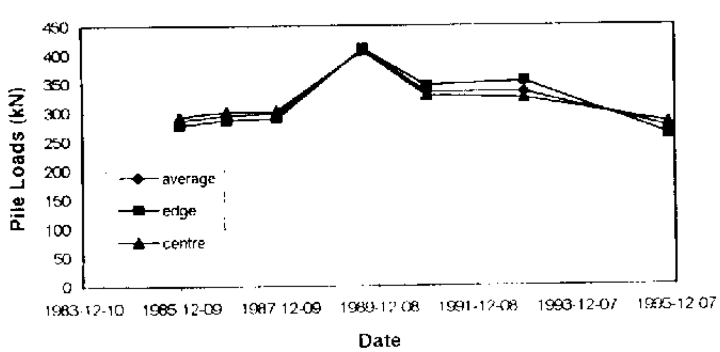


Fig. 7. Pile loads in the centre of building 2

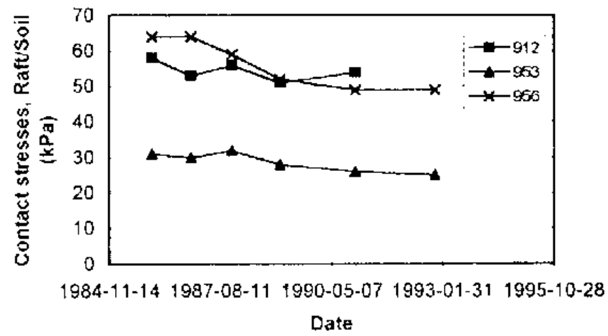


Fig. 8. Contact stresses at the soil/raft interface as a function of time.

and some have seized functioning. The excess pore pressures from depths 5 to 35 m below *building 1* have decrease slightly (about 5 to 15 kPa) during the period 1985–1995 while in the case of *building 2* the excess pore pressure has decreased slightly at depths where the initial excess pore pressure was highest and increased slightly where the initial excess pore pressure was lowest.

The future effective stress increase caused by remaining excess pore pressure dissipation indicates that the additional settlements to be expected ought to be larger than those anticipated from the settlement observations. However, this discrepancy may be due to the fact that the compression modulus of the soil is higher than assumed in the range of effective stresses referred to.

CONCLUSION

In conclusion we find that the result obtained by the new design principle based on load sharing between raft and creep piles is equally good as that obtained by the classical approach where the load is carried merely on piles with a high factor of safety against pile failure. The saving in number of piles (in this case about two thirds, considering the total bottom areas of the two houses) and, accordingly, in foundation costs is considerable. An illustrative factual example of the possibilities of saving foundation costs by applying the design principle described in this paper is demonstrated by Fig. 9.

REFERENCES

Burland, J. [1986]. "The value of field measurements in the design and construction of deep foundations." Proc. Intern. Conf. on Deep Foundations, Beijing, Vol. 2, pp. 177–187.

Hansbo, S. [1984]. "Foundation on friction creep piles in soft clays." Proc. Intern. Conf. on Case Histories in Geot. Engrg., St. Louis, Vol. 2, pp. 913–922.

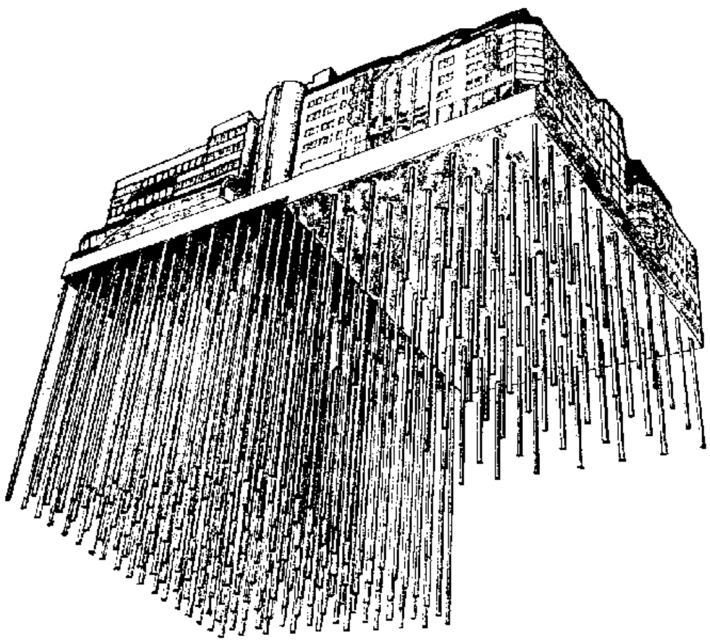


Fig. 9. Two nearby buildings in Gothenburg, one founded in the traditional way, the other according to the new principle of design with creep piles.

Hansbo, S. [1993]. "Interaction problems related to installation of pile groups". In *Deep Foundations on Bored and Auger Piles, BAP II* (Van Impe, ed.), Balkema, pp. 59–66.

Jendeby, L. [1986]. "Friction piled foundations in soft clay. A study of load transfer and settlements." Ph. D. Thesis, Chalmers University of Technology, Gothenburg.

Jendeby, L. [1991]. "Load transfer in friction piled foundations in soft clay". Proc. 10th ECSMFE, Florence, pp.

Jendeby, L. [1996]. "Långtidsuppföljning av fyra byggnader grundlagda med hjälp av kohesionspålar". ("Long-term follow-up of four buildings founded by the use of cohesion piles"). Chalmers Un. of Techn., Dep. of Geot. Engrg., Rapport B 1996:7

Randolph, M. F. & Clancy, P. [1993]. "Efficient design of piled rafts". In *Deep Foundations on Bored and Auger Piles, BAP II* (Van Impe,d.), Balkema, pp. 119–130. e

Phung, D. L. [1993]. "Footings with settlement reducing piles". Ph. D. Thesis, Chalmers University of Technology, Gothenburg.

Svensson, P. L. [1991]. "Soil-structure interaction of foundations on soft clay. Experience during the last ten years". Proc. 10th Europ. Conf. Soil Mech. Found. Engrg., Florence, Vol. II, pp. 583–586.