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Royal Christianina Hotel: Basement with permanent sheet pile wall, 'up-and-down' method

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ABSTRACT: The extension of the old Viking Hotel in Oslo was carried out by using an unconventional method of construction, so called "up and down" method. This gives the opportunity to construct both above and below ground level simultaneously. It requires, however, that the foundation work is finished before the main excavation starts.

Permanent steel sheet piles forms the outer wall in the basement, the basement floors making the inner bracing. The pile type used is Larssen 430 from Hoesch Stahl, Germany. The sheet pile also forms a part of the foundation of the new building. A total of approx. 3000 m^2 sheet piles were used in the project.

RESUME: L'extension de l'Hotell Viking à Oslo, a eté realisé en utilisent la methode "up and down". Cette methode permet la construction simultanée à partir de rez-de-chaussée des etages de la structure et, en meme temp, des soussols, mais elle requises que les travaux des fondations seront accomplis avant que les travaux d'excavation commencent.

Le mur exterieur de soussols est constitue de rideaux de palplanches metalique permanentes les planchers des soussols jouent le rôle de buttons. Les rideaux des palplanches sont utilisées ausi comme une partie de fondations du bâtiment. À-peu-prêt 3000 cm^2 de rideaux des palplanches ont eté utilisée pour ce project.

1 INTRODUCTION

Viking Hotel is situated in the Central Area of Oslo, the capitol of Norway. The hotel was opened just before the winter Olympic Games in 1952 and is a 13 story building with one basement floor.

Between 1988-90 the hotel was extended with a new building covering 2200 m^2 and renamed Royal Christiania Hotel.

The new part has 9 floors above ground level and 3 floors below. Today, the hotel is an international 5 stars hotel, and the 3 basement floors in the addition part is used for car parking.

2 CONSTRUCTION METHOD

The traditional way of constructing a building is first to carry out the excavation and construct the basement, and then construct the remaining portion of the building above ground level. This method was evaluated together with others in the conceptual study. One of the methods we proposed was to build below and above ground level at the same time. This would result in a shorter construction period and earlier profit on invested capital. Although the construction method itself turned out more expensive than others, a calculation of total cost where financial expenses etc. was included, showed that this was the most feasible solution.

The construction method can be named as the "up-and-down" method, and had at that time not been used in a similar way in Norway. This was quite a challenge both for the design team and the contractor. All participants worked in close cooperation and the following Norwegian companies were involved

Main contractor:	Aker Entreprenør
Structural:	A. L. Høyer
Geotechnical:	NOTEBY

The other main item in the construction method was the decision of using permanent steel sheet pile walls. The sheet piles had two primary tasks.

1. Horizontal support of soil during excavation and in permanent phase.
2. Vertical support of the deck slabs in the basement.

A short description of the implemented "up-and-down" method is given in the following sketches.

Stage 1:

Open excavation down to approx. 2 m below ground level, the same level as the slab in the first basement. From this level the sheet piles were driven and the foundation was created. The foundation of inner columns/walls consisted of cast in place concrete piles with diameter 1.1 m. The piles were concreted from bedrock to the level of the future bottom slab. In the pile casting operation, steel columns were fastened to the top of the concrete piles to support the coming basement floor slabs.

The outer walls are founded on driven steel columns positioned directly behind the sheet pile wall.

The sheet piles were driven to bedrock by vibratory-hammers. To assure the bearing capacity we used a 5 ton hydraulic gravity hammer as the final device to reach a specified criteria. See Fig. 1.

The sheet pile top were cut at defined level, ply wood was placed on levelled ground, then the slab in the first basement was concreted. In the centre of the slab we left out an opening through which the excavated soil from the lower basement were lifted out, see Fig. 2.

Stage 2

The next step was to concrete columns, walls and the slab on ground floor and thus establish a rigid frame. See Fig. 3.

Stage 3:

Excavation of the second basement was done in sequences combined with the concreting of the slab. The slab formed the horizontal brace of the sheet piles,

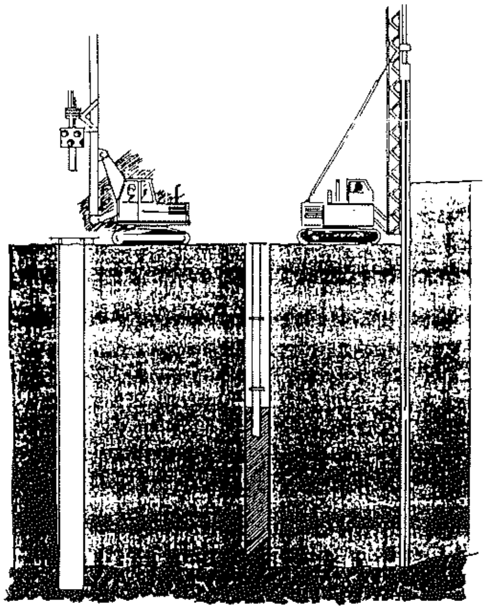

Figure 1. Stage 1. Foundations.

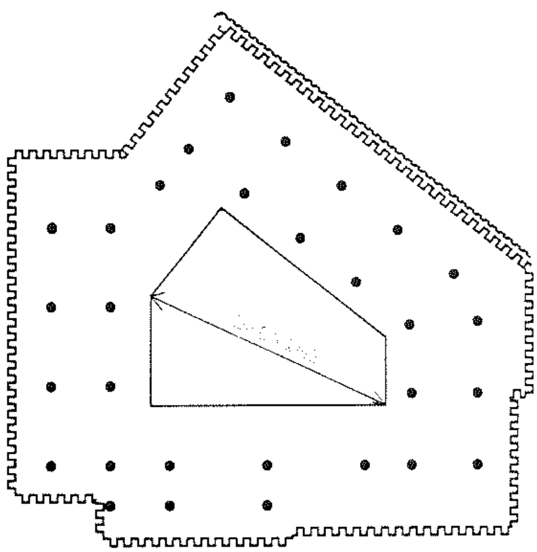

Figure 2. Stage 2. Opening in floor slab.

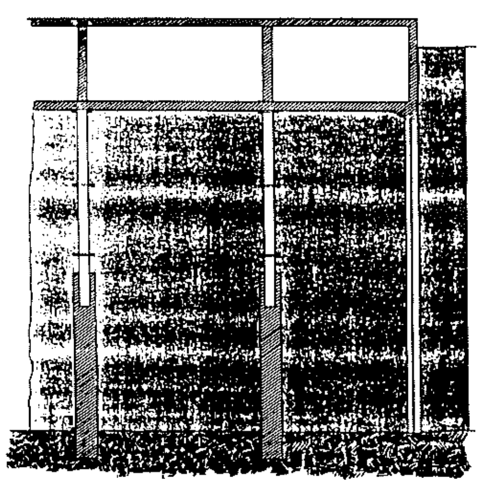

Figure 3. Stage 1

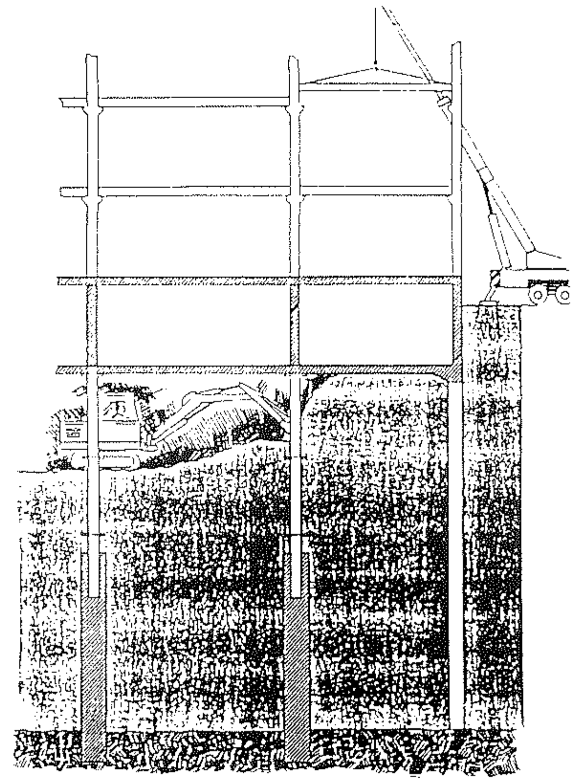

Figure 4. Stage 3. Excavation below floor slab.

and back-anchors could be left out. At this stage erection of the precasted structure above ground level could be initiated. See Fig. 4.

Stage 4:

The third and lowest basement floor was constructed by the same procedures as the one above. See Fig. 5.

The bottom slab was concreted with a thickness of 600 mm and with double injection pipes in all joints.

Finally, the main advantages of the method:

1. Shorter construction time
2. Lower total costs
3. No anchors outside the construction, which could have meant conflict with neighbouring buildings or possible aquifers

3 SOIL CONDITIONS

Several soil investigations had been carried out earlier at the construction site. Together with experience gained from various sites in the neighbouring area information about soil condition were compiled allowing establishment of geotechnical design parameters.

Beneath the usual top layer of fill materials, old foundations etc., the soil consists of medium stiff to soft clay. The clay is normally consolidated and has low sensitivity. Fig. 6 shows undrained shear strength from one bore hole. In general, the undrained shear strength varies between 15-40 kN/m^2. Triaxial tests gave results as follows (drained):

Friction: $\phi_k = 26°$

Attraction: $a = 10$ kN/m^2

The bedrock is covered by a thin layer of sand/-gravel having a much greater permeability than the clay.

The depth from ground surface to bedrock varies between 15 to 23 m. The bedrock consists of shales with areas of alun shales.

Groundwater level is approx. 2.5 m below ground surface.

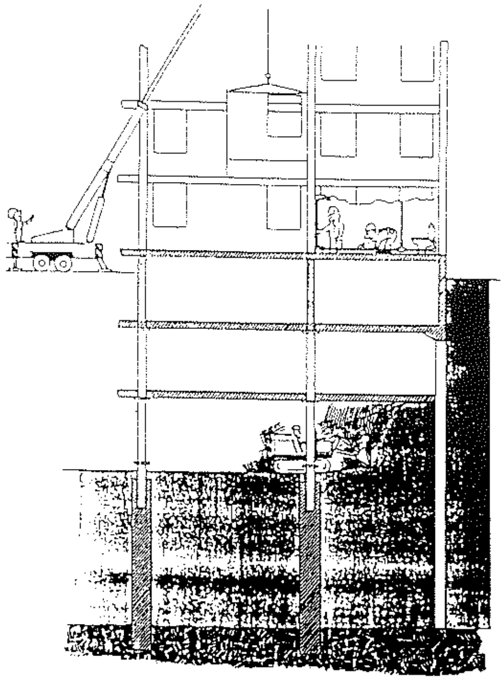

Figure 5. Stage 4. Final excavation.

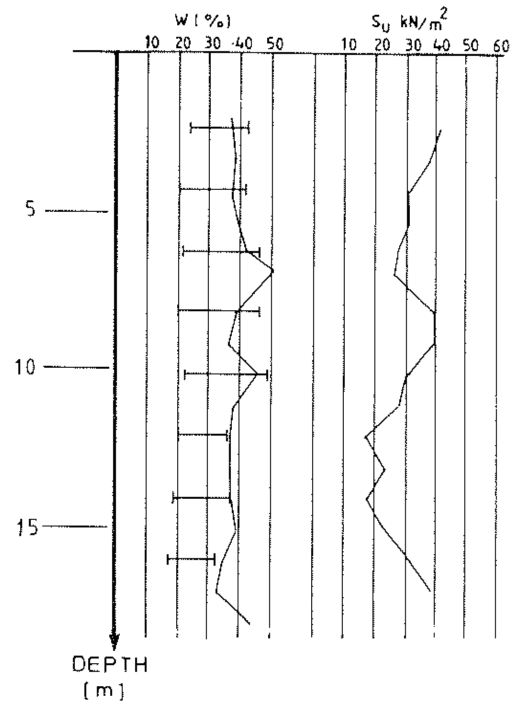

Figure 6. Clay profile.

4 SURROUNDING CONDITIONS

The central area of Oslo is composed of both new and old buildings. The new buildings are normally founded on piles to bedrock, while the old ones are brick buildings founded in clay. Development of an urban site which includes deep excavation (normally taken as deeper than two basement floors) requires expanded responsibilities to the developer concerning the surrounding buildings, streets, conduits, tram rails etc. The responsibility is connected with all sorts of damage due to the foundation- and excavation work, thus making it necessary to be able to evaluate the

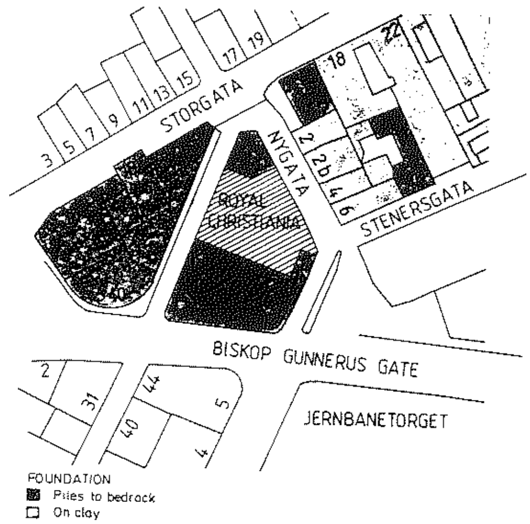

Figure 7. Neighbouring conditions.

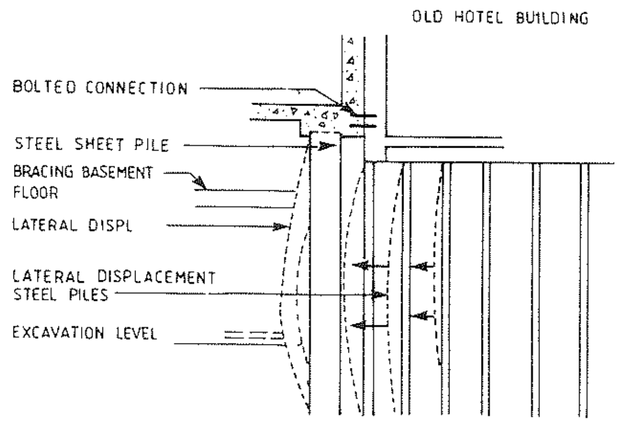

Figure 8. Connection old/new structure.

possible damage prior to the construction period. Most of the potential damage would be connected with settlement.

Fig. 7 illutrates the neighbouring conditions of the Royal Christiania Hotel.

The nearest existing buildings are founded on steel piles on bedrock, piles which have, however, very little reserve capacity. Due to these conditions and the fact that the excavation would lead to horizontal deformation which could cause the piles to buckle, it was necessary to strengthen the existing foundation. This was accomplished by bolting the old structure to a beam concreted on top of the sheet piles. Fig. 8 illustrates the principle against the old hotel.

Other areas that needed special consideration were the surrounding streets and the old buildings founded in clay on the other side of Nygata. Based on the estimated degree of settlement due to the excavation, it was decided that it would not be necessary to strengthen the foundation of the old buildings.

All buildings in the area of influence were registe-red with photos and their condition described before the construction commenced. Furthermore, a programme of registrating settlements was established several years before hand. In order to follow up ground water conditions, piezometers were installed.

The follow-up programme was coordinated with several other projects in the area which were carried out at the same time. This could also, to some degree, make it difficult to determined which project was the reason for a certain damage due to common influence in the area.

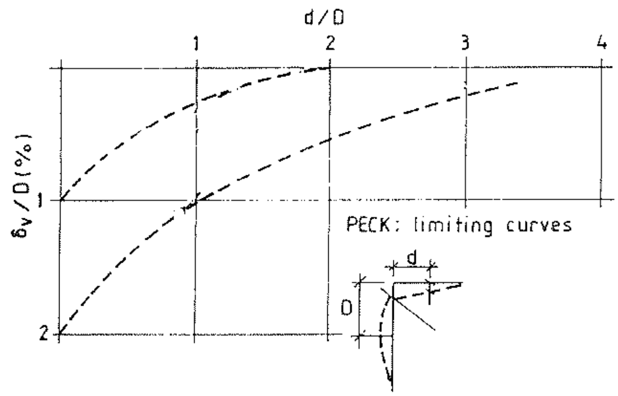

Figure 9. Peck Diagr. 1969.

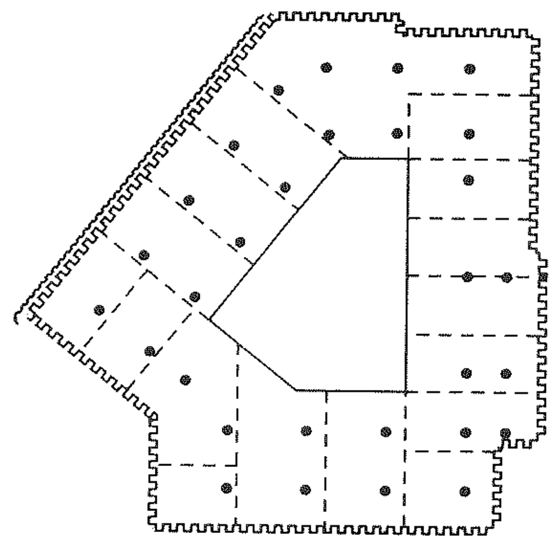

Figure 10. Excavation sequences - the lowest basement floor.

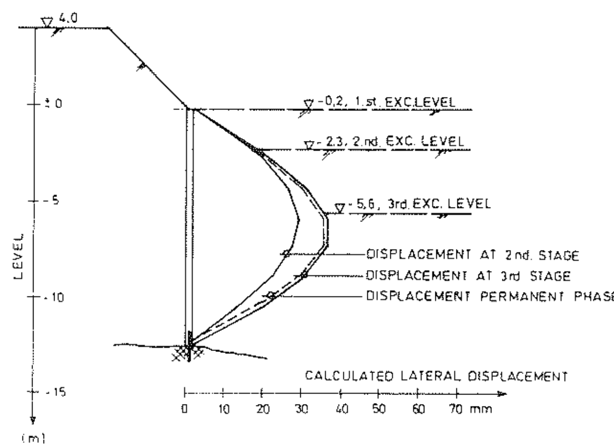

Figure 11. Calculated lateral displacement.

5 PERMANENT STEEL SHEET PILE WALL

5.1 Design

One of the main problem with retaining structures for deep excavations in soft clays, particularly in urban areas, is not only to adequately design the retaining structure itself to resist forces and bending moments induced by earth pressures, but also - equally important - to restrict lateral deformations and settlements of the ground behind it so that the surrounding buildings and facilities should not be damaged by these movements.

Settlements of the ground around excavations, both magnitude and extent, are affected by the following main factors:

1. Soil stiffness and strength characteristics
2. Influence of pore pressure changes and consolidation
3. Lateral displacement of retaining structures i.e. stiffness of retaining structure and interaction soil/structure

The classical collection of experienced data concerning retaining walls with inner bracing is the diagram of Peck, 1969. See Fig. 9. The experiences in Oslo-clay has shown that expected settlement next to the excavation would be of magnitude 1-2 % of excavation depth. The magnitude and extent of settlement increases with increasing depth to bedrock, duration of the excavation, stiffness of the retaining system etc.

The calculation of the retaining wall was done with a computer programme developed by NOTEBY. The programme is based on elastic behaviour of the soil and interaction soil/structure. The programme computates lateral displacement, rotations, movement and shear forces together with earth pressure on both sides of the wall for each step of the excavation.

The calculations were performed in two main phases:
1. Excavation phase
2. Permanent phase

Both phases have been calculated in service stability limit state where the main interest was horizontal displacement. Then the structure was checked in ultimate limit state to compute dimensions, forces acting on bracing basements slabs etc.

The calculations revealed not surprisingly, that the intermediate phase (during excavation) was the period which resulted in largest displacements and stresses in the structure.

The calculations and choice of method/structure had as a main option to reduce lateral movement to an acceptable level. In our case, this was decided to be between 25 and 65 mm accumulated in the final state. The different values are due to different surrounding conditions. Using the basement slabs as bracing, made it difficult and expensive to do any pre-stressing of the retaining wall. Furthermore, shrinkage of the concrete slabs had to be taken into consideration.

Optimizing the retaining wall structure also included calculation of the resulting effect of excavating in sequences. Fig. 10 shows the sequences on the lowest basement floor. As soon as one sequence had been excavated, a 20 cm thick concrete slab was established. The first stage was to connect two opposite sides of retaining walls. Then sections with approx. 6-8 m width was excavated and the slab continuously concreted. By using this method, we calculated the reduction in displacement to be 20-25 mm. Another profit of the method was that the concrete sequences and joints of the constructive-bottom-slab could be performed independent of the excavation.

Typical results from the calculations are shown in Fig. 11.

As shown in the figure, the designing phase for the sheet pile wall is during excavation. The permanent phase has a design situation more or less equal to excavation phase. However, permanent phase includes other design criterias such as corrosion. Experiences both abroad and in Norway imply rate of corrosion in clay to be low, especially below ground water level. The final solution in this project included the top of the sheet piles being cut below ground water level, and hence the rate of corrosion was evaluated to have a maximum of 50 µm/year.

Different kinds of corosion protection methods were looked into. However, we found that the most feasible solution was using the steel itself in combination with sandblasting and painting on the exposed surface in the basements. This should ensure the entire life period of the building (100 years). The remaining

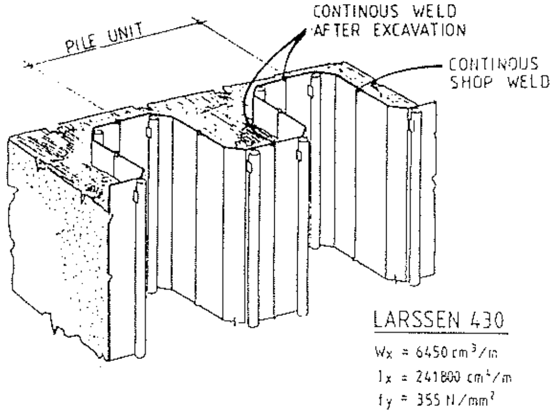

Figure 12. Larssen 430-profile.

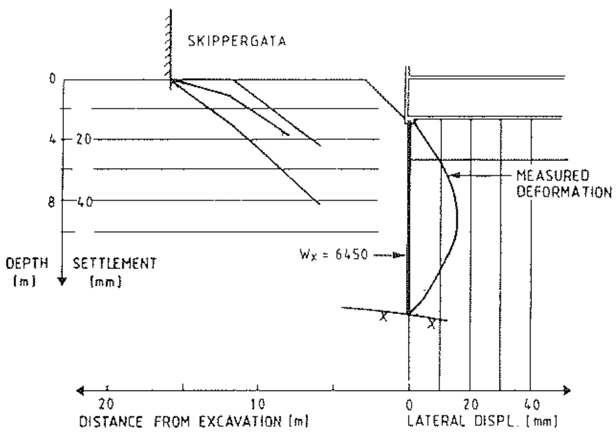

Figure 13. Observed deformations.

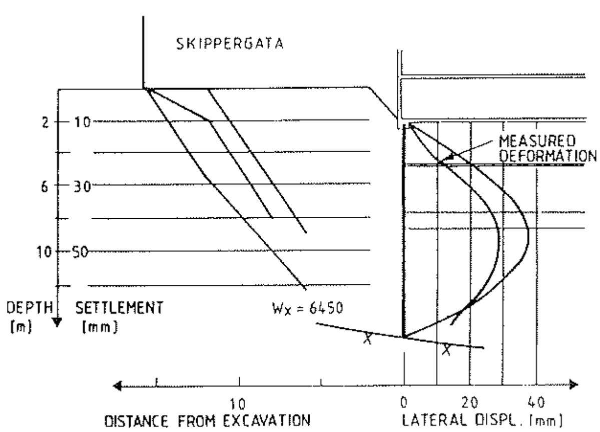

Figure 14. Observed deformations.

steel thickness can, however, at any time be checked by ultrasound devices. Strengthening measures can include concreting inner walls if found necessary.

5.2 Type of sheet piles

The use of permanent steel sheet pile walls in ordinary buildings has just recently been adopted in Norway, although it has been a well known construction method, for instance in quays.

Several profiles and types of sheet piles where considered for the Royal Christiania Hotel project. The evaluation was based on

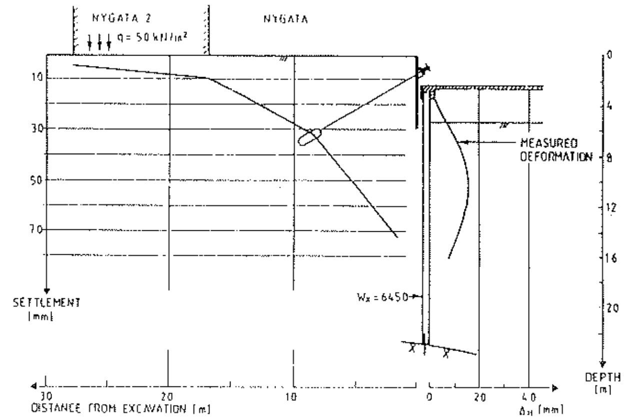

Figure 15. Observed deformations.

- experience of the contractor
- equipment needed and available
- cost and delivery time
- tolerances
- drivability
- stiffness
- water tightness in joints
- possibilities to place rock anchor bolts at pile toe

A total of approx. 2900 m^2 of sheet piles were needed, and the calculations suggested a section modulus greater than or equal to 6500 cm^3/m, and moment of inertia of 240,000 cm^4/m. Pile length up to 23 m was needed.

The evaluations resulted in a decision to implement sheet piles of type "Larssen 430" from Hoesch Stahl AG. (See Fig. 12). A detailed sheet pile plan was laid forth by Hoesch, showing all numbered elements, pile length, driving direction etc. The plan was based on coordinates and theoretical measurements of the building site. All corners were designed in detail in order to ensure a continuous jointed wall.

The schedule of delivery and shipment was based on the pile lay-out and the progress on the site.

Larssen 430 are put together by placing two and two Larssen 43 profiles in "Z" and "S"-configurations. The joint in the "Z" and the "S" are welded at the plant. This results in a continuous seem on the inside with a length a bit longer than the future excavation depth, and in certain intervals on the rest of the pile length. On site, the piles are put together four and four. Steel pipes for later bolting of the piles to bedrock are welded to the pile sides as shown in Fig. 12.

The piles are then driven to bedrock by using vibratorhammers and each of the "Z" and "S" piles are afterwards driven separately by a gravity hammer to ensure good contact with bedrock.

5.3 Water tightness

Excavation below ground water level results in temporary lowered water level. One main goal in this project was to influence the water level as little as possible. In permanent phase the basement has to be watertight since permanently lowered ground water level is not allowed.

The joints of the Larssen 430 are in general reasonably tight. However, to ensure as little leakage as possible during excavation phases, a bitumen based material (Eurolan) was put in the unvelded joints. Primarily this was done to prevent leakage up from the sand/gravel layer covering the bedrock.

The method in itself is favourable concerning leakage possibilities due to using inner bracings instead of back-anchors. After completed excavation all the joints in the sheet pile wall have been continuously welded on the exposed length. Between

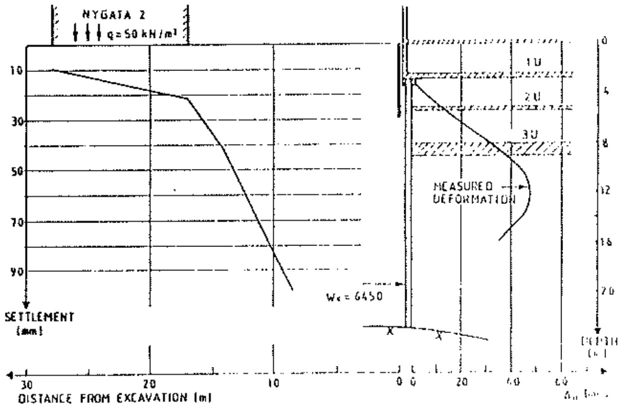

Figure 16. Observed deformations.

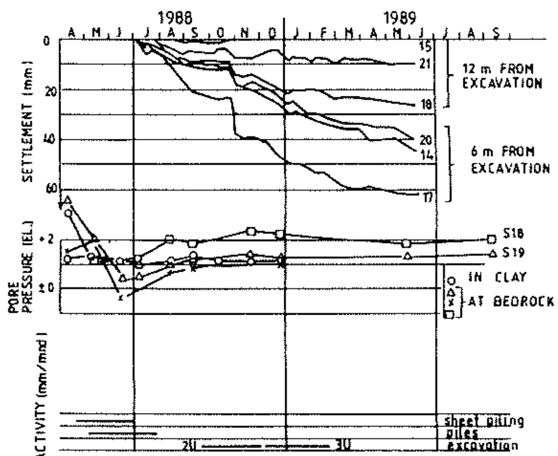

Figure 17. Observed settlement and pore pressure during excavation period.

the sheet piles and the bottom slab, the seal is ensured by double injection tubes.

6 FOLLOW UP/EXPERIENCES

Some vital data are given in the following table:

Site area	2200 m^2
Excavated volume	20000 m^3
Sheet piles	3000 m^2
In-situ casted piles	48
Steel piles	30

The construction work started in March 1988 and the basement was completed one year later. The precasted structure above ground level started 15. Nov. 1988 and was finished 1. July 1989.

To monitor and ensure safety and quality, a specified programme was established. This included

- pore pressure surveillance
- settlement registration of surrounding buildings and street
- registration of vibrations during driving of sheet piles and other piles
- core drilling in in-situ casted piles
- leakage measurements after completed basement structure
- pore pressure registration underneath bottom slab to ensure sufficient dead load from the building before pore pressure was allowed to rise
- inclinometer registrations in tubes welded to the sheet piles

Inclinometer registrations together with settlement development gave important information in determining the dimensions of sections. In Fig. 13, 14, 15 and 16 the calculated and measured displacement in two cross-sections are shown. Registered settlement in street level are also shown in the figures.

Lateral displacement was less than those calculated, with maximum displacement of approx. 50 mm. The displacement varied to a large extent with depth to bedrock. Experience showed that displacement in the sheet piles near the corners were approx. 50 % of the maximum displacement.

The settlements turned out to be largest in Nygata. This was, however, also a result of a temporary sheet pile wall along this street. This sheet pile wall had back-anchors of Expander Body system soil anchors and was established as a temporary retaining wall against the street. The anchors were released and the sheet pile wall drawn up.

There was a measured decrease in pore pressure of about 0.5-1.0 m during the construction period. This was a very satisfying result and was together with watertight sheet piles and inner bracing, probably also influenced by water injection system established in connection with an other project in the area.

Fig. 17 shows pore pressure development, settlement and construction activity.

Finally, Fig. 18 shows the measured settlement in the Peck diagram of 1969.

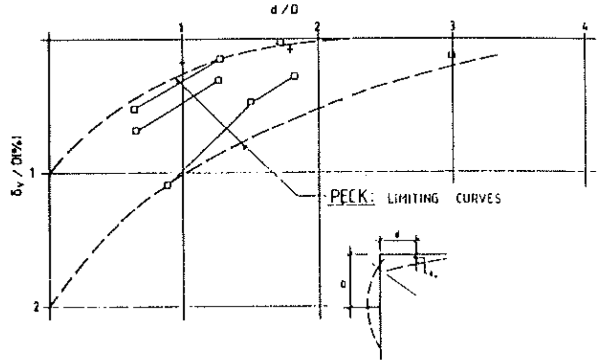

Figure 18. Observed settlements.