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Deep basement with permanent sheet pile walls.

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Oslo City: Deep basement with permanent sheet pile walls. High strength concrete piles

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ABSTRACT: The largest commercial building in Norway - the OSLO CITY - comprises a total basement area of 28.000 m^2, including a six floor deep parking garage excavated in soft to medium clay, partly down to rock. This 17-23 m deep excavation is supported by a 6.000 m^2 sheet pile wall which also forms the permanent walls of the basement. Approx. 700 pcs of temporary back-anchors transferred the earth pressures to rock during the excavation phase. The project had strict requirements to limited deformations of adjacent property, which included several large buildings and a subway-station. Particular care was taken to avoid any permanent changes in the ground water conditions, and to limit the temporary effects as much as possible. The basement is undrained with the sheet piling made watertight by welding. The uplift is controlled by permanent rock-anchors in the base plate. The problem of corrosion is mainly taken care of by providing a generous steel thickness.

RESUME: Le plus grande bâtiment commerciel de la Norvège - Oslo City - à un surface total, en plan, de 28.000 m^2 y compris 6 niveaux sonterrains de garage qui ont eté excavée en argiles mols, partiellement reposent sur la roche. L'excavation profonde est sontenue par en rideaux de palplanches, permanent qui jouent ausi le rôle de mur exterieur pour le bâtiment. Environs 700 anchrages temporaires ont transferrait la pousse du terrain à la roche. Le projet est characterise par des exigeauces tres strictes en ce qui concern les deformation des bâtiments voisius, y compris des bâtiments importantes et une station de metró. Des mesures particulières ont ete puis pour eviter/reduir changements dans le regime hydrostatique de la zone. Le radier de soussol n'est pas drainé, et les rideaux des palplanches sont impermeabilisées par sudure des jointes. Les pressions asscendentes de l'eau agissantes sur radier sont contracarrées par auchrages verticaux du radier. La corrosion d'acier est prevenne en choisirent une grosses suffisante pur les palplanches.

1 INTRODUCTION

OSLO CITY was built by Selmer-Furuholmen Oslo a.s for Selmer-Sande Properties a.s. The 81.000 m^2 project was completed in November 1988 after a total construction period of only 24 mths.

The triangular down-town site of 8400 m^2 has depths to rock varying from 14 to 40 m and medium to soft marine clay subsoils.

Proper consideration of the geotechnical possibilities and restraints of the site from the initial feasibility study to the construction stage ensured an optimal utilization of the ground, with a total basement area of 28.000 m^2.

The soil conditions were such that stability problems/bottom heave would occur in conventional excavations deeper than for 1-2 basement levels. It was therefore decided to make a deep basement excavation in the norther part of the site where excavation support to rock would be possible. The southern half of the site which had greater depths to rock was only excavated for one basement level, as a deep excavation here would have required prohibitively costly stabilizing measures.

Two aspects of the project are of particular interest:-

1.1 Steel basement

An area of 4400 m^2 with depths to rock varying from 14 to 25 m was fully excavated to 17 to 23 m depth for 5 to 6 basement floors, providing parking for 520 cars and room for the main technical installations. The excavation was supported by a heavy steel sheet piling, which also forms the permanent walls. During excavation the wall was supported by approx. 700 pcs of inclined back-anchors to rock. In the permanent phase, the walls are now supported by the basement floors. This use of permanent sheet piling was an important time-saving element of the project.

1.2 High strength precast concrete piles

Light temporary sheet piling with Soilex ground-anchors supported the shallow excavation of 5 m depth, the permanent walls being of conventional concrete construction.

This part of the building is founded to rock on 250 pcs driven precast concrete piles, totalling approx. 7500 m. In order to reduce the number of piles and the pile cross-section, a concrete cube strength of 75 N/mm^2 was specified. High strength concrete in piles is a result of our long experience in the use of such concrete in offshore structures.

1.3 Progress. Project administration

The foundation works started in November 1986 and the baseplate of the deep excavation was completed in

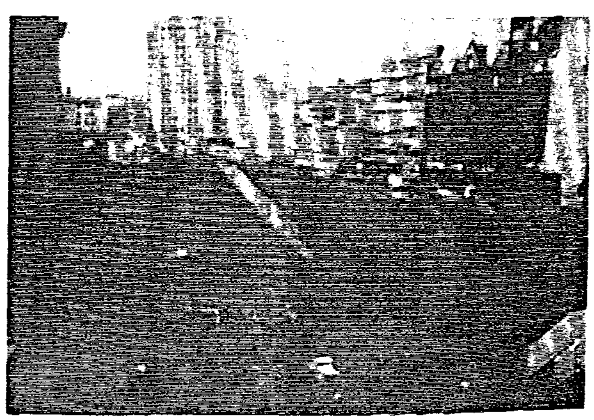
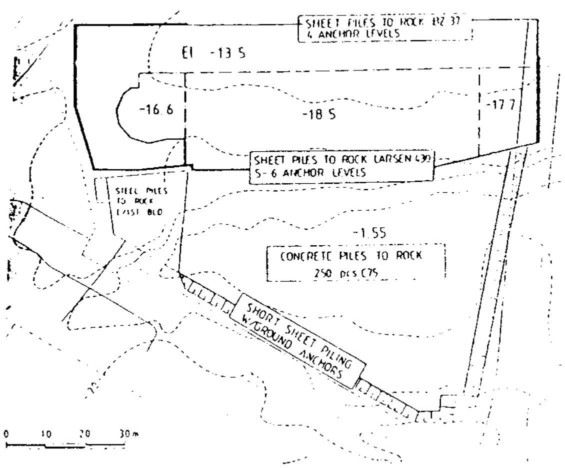


Figure 1.



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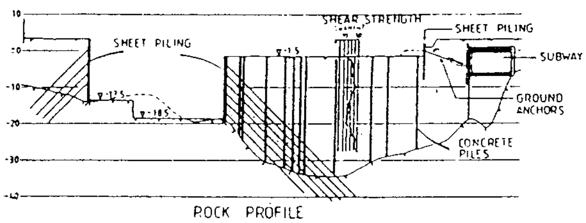


Figure 3.

December 1987. Columns were then erected and the basement top-slab poured. This enabled the above ground construction work of 8-10 floors to proceed at the same time as the deep basement was completed. The owner took possession of the building only 11 months after the completion of the deep base plate. Despite the record breaking excavation depth for the use of back-anchored sheet piling in "Oslo clay", the work advanced with no major problems.

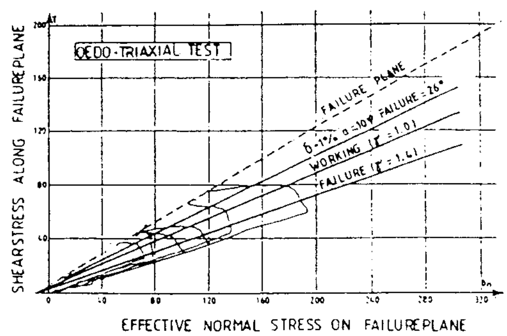
NOTEBY the geotechnical consultant was on the site throughout the construction stage, monitoring the work and advising on whatever problems that arose in co-operation with the project administration and contractor.

Both the permanent sheet piling basement construction and the high strength concrete piles were chosen as a result of close co-operation between the architect, consultants, contractor and owner, and a collective willingness to try new ideas. Several other concepts were considered, e.g. circular and plane diaphragm walls and use of consecutively cast basement floors to support the sheet piling in the excavation phase. The final design was chosen on the grounds of economy and construction progress.

2 SITE AND GROUND CONDITIONS

The site is situated in downtown Oslo. The ground elevation is +3 to +3.6 m. Depths to the shale rock vary from 14 m towards Stenersgata in the north to approx. 40 m in a steep sided depression in the bedrock crossing the site in a NE-SW direction, re. Figs. 2 and 3. Fill of thickness 2-5 m covered the natural ground of normally consolidated marine clay with undrained shear strength varying from 15 to 50 kN/m^2. Fig. 4 shows typical shear strength parameters. The ground water level is approx. 2 m below terrain.

The existing buildings north of the site are mostly founded directly on the clay soils. The southwestern part of the site is bordered by an underground railway tunnel founded on piles. One older building,



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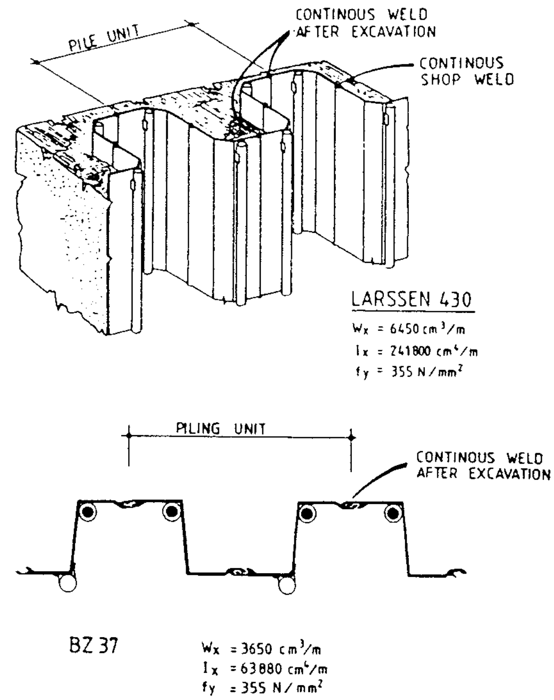

Figure 5.

Nygata 12, founded on steel piles to rock was to be incorporated in the new development.

3 PERMANENT STEEL SHEET PILES

The following pile sections were chosen after close consultations with the piling contractor Entreprenørservice A/S and the suppliers Norsk Stål/Hoesch and Trade Arbed:-

- Bz 37 (Arbed) for depths to rock up to 17 m
- Larssen 430 (Hoesch) for depths exceeding 17 m.

Refer Fig. 5.

The following particulars were considered when choosing pile sections:-

- Pile driving. Tolerances and requirements to verticallity. Required driving equipment, leads and hammers.
- Positioning of temporary back-anchors and ease of sealing
- Water tightness of the clutches. Need for on site welding/shop welding
- Bolting to rock of the pile foot for transfer of shear forces during excavation
- Bearing capacity on rock for the vertical anchor load reaction during excavation and for the permanent foundation loads

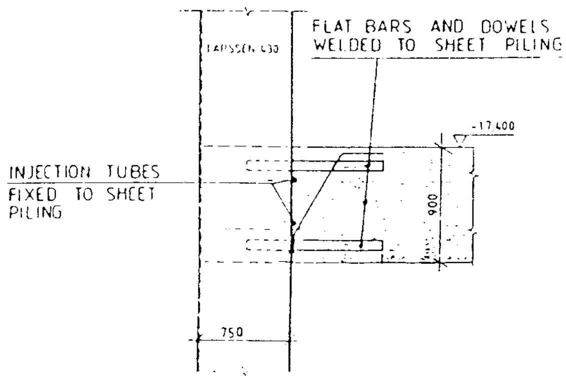

Figure 6.

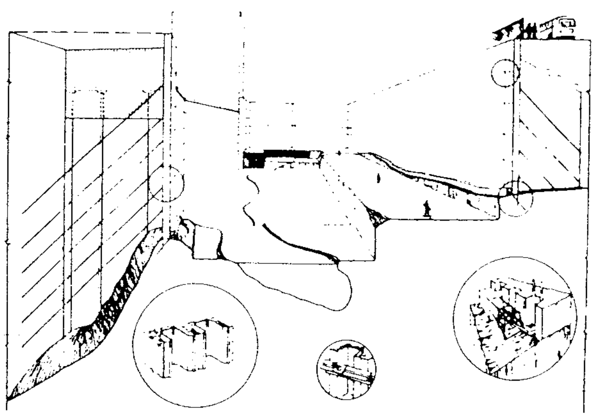

Figure 7.

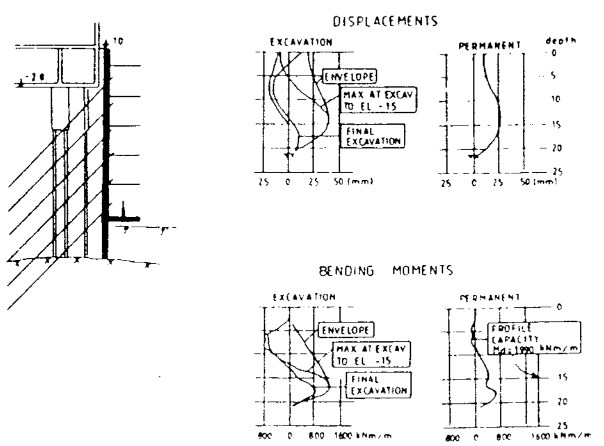

Figure 8.

- Delivery time and cost

The Bz 37 profile was well known for excavation support, whereas the suitability of the Larssen 430 could not be documented. This profile had previously been used mostly in quay and dock-works with moderate driving depths.

3.1 Corrosion and water tightness. Base plate

Corrosion protection is by passive means. Monitored steel piling in the "Oslo clay" show slow corrosion development, 0.01-0.06 mm per year. This was confirmed by inspection of some old sheet piling extracted from the site after 30 years in the ground. The steel thickness was found still to be within the original production tolerance. External corrosion capacity is therefore ensured by use of excessive steel thickness (min. total thickness 12 mm). Inside protection is obtained by an epoxy paint system applied after sand blasting.

The basement is undrained. All clutches and the openings cut for the back-anchors have been sealed by welding.

Sealing of the base plate/steel wall joint is primarily by injection through two horizontal tubes fixed to the steel piling. Refer Fig. 6.

The 21 m head of water pressure on the base plate is balanced by a combination of anchor cables and piles to rock.

3.2 Wall design

The sheet pile wall was designed using the computer program "SLISS-SPUNT" developed by NOTEBY. This program is based on a "beam on elastic supports" concept. Three phases were evaluated:

- Excavation
- Transfer of earth pressures from back-anchors to the floor slabs
- Long term, permanent phase

For all three phases, the calculations were based on assumed working loads with emphasis on limited deformations, and with a check of the ultimate capacity.

Our computer program calculated displacement, rotation, bending moment, shear force and the earth pressures along the full depth of the wall for each of the above defined phases.

Important soil and pile parameters are shown by Figs. 4 and 5. The Norwegian code of practice for geotechnical design is based on the limit state design concept, prescribing a factor of 1.4 on the effective shear strength for projects with very serious consequences of an ultimate failure.

The calculations were also controlled with a finite-element analysis of one section.

3.3 Design of back-anchors

The back-anchors were designed from the calculated required working loads. Safety against the possibility of progressive failure and the ultimate strength requirements were then taken into consideration.

Much effort was spent on "threading" the anchors between the 250 pcs of concrete piles of the southern part of the site, the steel piles of the one existing building in the same block and the concrete piles of the adjacent underground tunnel.

Anchor design capacities varied from 1500 to 3600 kN each, with spacing approx. 2-3 m apart both vertically and horizontally.

3.4 Special considerations. Deformations

The design aimed at limiting permanent wall displacements to max. 50 mm, whereas up to 80 mm had to be tolerated during certain phases of excavation.

Even stricter limitations had to be observed for the 20 m deep sheet piling adjacent to the existing building, Nygata 12, due to the danger of inducing yield in the foundation piles, refer Fig. 8.

The design also had to account for high pore pressures in the clay caused by the pile driving on the southern part of the site.

3.5 Construction

The sheet piling and back-anchors were installed by Entreprenørservice A/S. Vibratory equipment drove the piling to rock followed by a chiselling procedure using a hydraulic drop hammer. The sheet piling was

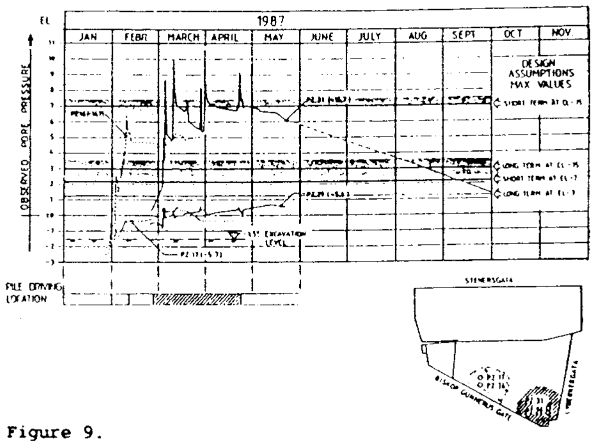

Figure 9.

delivered to the site "tailor made" to the expected rock elevations. The vibro technique proved both efficient and environmentally sound in regards to noise and vibration.

The pile tip was then bolted to rock by drilling from the terrain through pipe sections welded to the sheet piling.

Excavation was in lifts of 3 meters with successive installation and tensioning of the 4 levels of inclined back-anchors to rock. A total of 380 anchors were drilled between the concrete piles of the southern part of the site and adjacent properties, with only 8 incidents of conflicts necessitating change of drilling angle. Some areas with weak rock and locally unfavourable rock topography caused certain problems. Water leakages from the rock both during the anchor installation and later were the most serious complication, as expected from previous experience with this construction method in the area.

4 DRIVEN CONCRETE PILES AND STABILITY OF SHALLOW EXCAVATION

Use of driven, precast concrete foundation piles bearing on rock was favoured by cost and progress considerations. However, pile driving could cause stability problems due to the high pore pressures expected to be developed in the clay soils. Another problem was the question of negative skin friction. The geology of the area is such that the clay is still consolidating at a rate of 3-4 mm/year inducing considerable downdrag on piles. Few and slender piles would therefore be advantageous.

The effect of piling on the pore pressures and the use of high strength concrete was subsequently tested by a full scale trial of 2 piles, one of which had fibre reinforced concrete. We could conclude that pre-drilling for each pile to a depth of 10 m would ensure the stability of the excavation. Bitumen coating would reduce the downdrag load caused by negative skin friction. Fibre mesh concrete offered no apparent advantage.

For the test piles the acceptance criterion was verified by our Pile Driver Analyzer.

The actual foundation piles were of cross-section 756 cm^2, driven to an ultimate bearing capacity of 2800 kN by an hydraulic drop hammer of 6 t dead wt. Only 5 out of 250 piles were rejected and replaced. Drilled steel piles were used at a few locations due to very steep rock.

The pore pressures were registered by 3 pcs electric piezometers at 5 m depth and 5 pcs at 15 m depth. As expected, the piling resulted in considerable pore pressures, and the specified pre-drilling had to be supplemented by sand drains in certain critical areas to expedite the dissipation. The registered pressures were in acceptable agreement with the results of the test piling, refer Fig. 9, but with higher pressures

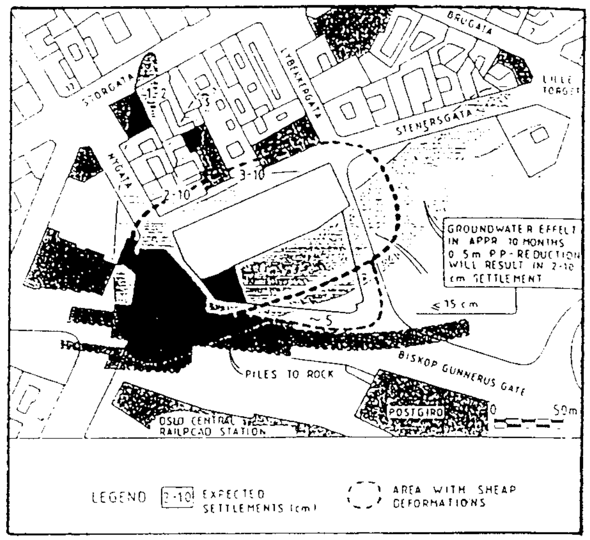

Figure 10.

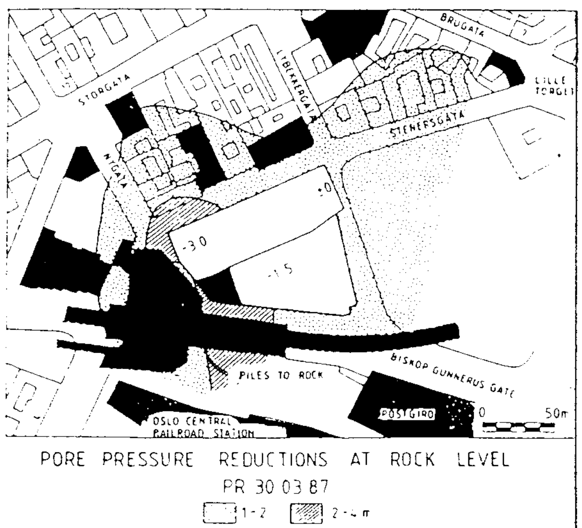

Figure 11.

at depth than anticipated. It took five months for the pore pressures to normalize.

5 DEFORMATIONS/GROUND WATER. PROBLEMS DURING CONSTRUCTION

Such a deep excavation is bound to influence neighbouring properties and installations. Considerable care was taken to minimize such effects to an acceptable level:

- Very stiff sheet piling/back-anchor system, designed to meet strict deformation criteria.
- Extensive temporary water sealing works. Injection sealing of water bearing rock. Injection of the sheet pile/rock joint prior to excavation and sealing with a continuous concrete foot beam immediately after excavation. Anchor detailing to minimize leakages both during installation and throughout the service period. Continuous welding of sheet pile clutches.
- Installation of water injection wells in the rock outside the excavation to counteract leakages.

Prior to construction, expected settlements due to sheet piling/soil shear deformations and ground

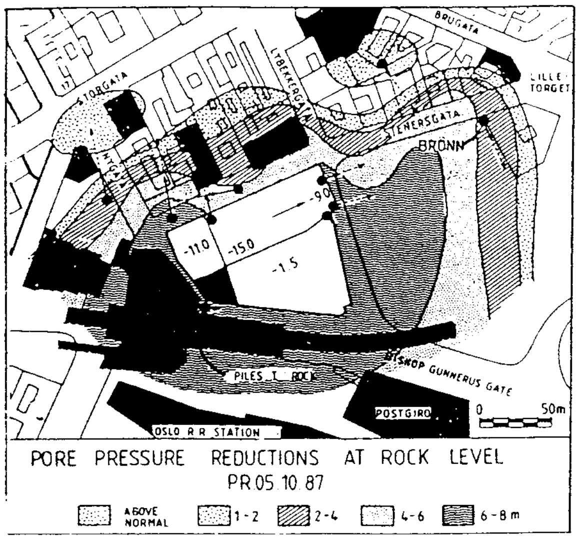

Figure 12.

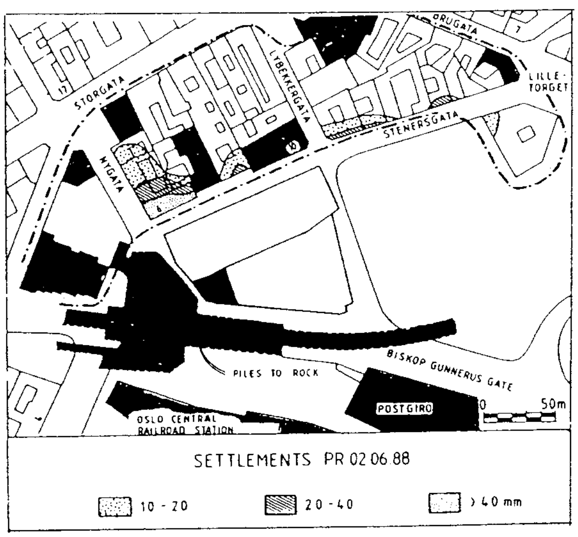

Figure 13.

water/pore pressure effects were calculated for all the surrounding properties, refer Fig. 10.

Settlement and pore pressures were closely monitored. Sheet piling deformations were checked by inclinometer measurements. Unfortunately, these latter measurements started after the initial excavation. The tension in some back-anchors were monitored by jacking.

The lessons learned were as follows:-

5.1 Sheet piling deformations

No fully verifiable conclusions can be drawn since the inclinometer observations don't include the initial excavation phase. However, the relative deformations in the subsequent phases where less than calculated.

Some local areas of the deepest wall showed sign of yielding for a short period.

In some cases the Larssen 430 profile was deformed locally by the reaction from the back-anchor wailing supports. This was caused by an unfavourable cutting of holes for the back-anchors relative to the wailing brackets. The transverse stability of the profile

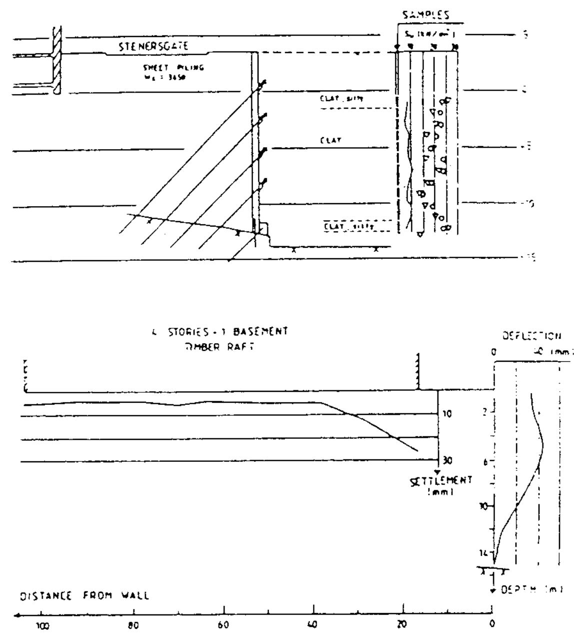

Figure 14.

then had to be ensured by struting against adjacent piles.

5.2 Back-anchor performance

The real back-anchor loads were generally found to be less than expected from the design calculations and the initial tensioning. This supports the results of the inclinometer observations.

There were no problems with anchor failure or excessive deformation during service.

However, there was a problem of water leakages. Some of these may be contributed to certain deviations from the specified injection/sealing procedure. This was done to save construction time, and was found very difficult to correct at a later stage.

5.3 Ground water

As expected, the project caused a temporary decrease in the pore pressures at rock level, resulting in consolidation settlement of the clay soils. The system of injection wells proved essential to control this.

The activities causing this fall in water pressures were mainly the inclined back anchors, rock cuts, vertical sheet pile foot bolts and installation of the permanent uplift anchorages. The leakages were mostly associated with areas of weak, fractured rock, particularly along the major depressions in the rock surface, which contained some permeable gravel cover causing extensive drainage.

Fig. 11 shows the situation after drilling of the second row of back-anchors in the western part of the site. The influence is already considerable. No injection wells were as yet installed.

Fig. 12 is the situation near the completion of the excavation, including all injection wells. At this time approx. 50 and 140 l of water/minute were injected in the western and eastern areas, respectively. To the east, the area of reduced pressure extends up to 150 m from the excavation.

Valves in the base plate were closed in April 1988, and within 2 months the pore pressures at rock level were back to 1 to 1.5 m from the normal. Normal conditions were completely restored 6 months later, except for areas influenced by other projects.

The project had no or insignificant effect on the ground water level as observed in the fill above the clay.

5.4 Summary

- The adjacent properties generally suffered less settlement than expected, refer Figs. 10 and 13. The local area with settlements exceeding 40 mm was caused by irregular excavation. Also note that the undeveloped area to the east of the site (not included by the monitoring) suffered settlements.
- The temporary effect on the ground water pressures was not very different from expected, but required more injection wells to control.
- The pore pressures caused by pile driving were within the limits accepted by the design.
- The sheet piling deformations cannot be accurately documented, but are believed to have been within the design assumptions.
- Fig. 14 gives a reconstructed displacement/-settlement diagram for one section towards the north, clearly showing the extended effect of the temporary ground water disturbance, and the greater deformations within a distance of approx. 2 x the excavation depth.

6 PERMANENT PERFORMANCE

The permanent structure functions well with an attractive, easy to maintain epoxy coating. It is exposed in all areas for easy observation of leakages and yearly ultra sonic thickness tests to monitor any development of external corrosion.

A more extensive chemical injection program than expected were required to obtain a satisfactory seal of the base plate/sheet pile wall joint, which has to be perfect to prevent local corrosion. The design included polyurethane coating of this detail as a tentative measure. So far, this has not been found necessary.

In addition to visual routine inspections and ultrasonic tests, the most highly stressed part of the wall is equipped with permanent inclinometer tubes for long term deformation observations.