Experience from many facets of a cut-and-cover tunnel construction

Une multitude d'expérience de la construction d'une tranchée couverte

W.Steiner – *B+S Ingenieur AG, Bern, Switzerland*

ABSTRACT: A 340-meter long cut-and-cover tunnel was constructed in soft and sensitive postglacial clays and glacial sands with a high water table in a suburban area of Switzerland maintaining traffic with local detours only. The roof slab method with diaphragm walls was used. The thickness of the diaphragm wall and the excavation sequence were adapted to the geotechnical conditions. Detailed site investigations with borehole tests (DMT), in-situ tests (CPTU) and pore pressure monitoring were carried out. Test panels were constructed and monitored. The construction was followed by a detailed monitoring program, which included piezometers, inclinometers, settlements, and strut forces. Measurements were evaluated and corrective action taken, if necessary. Effects on neighbouring structures were limited even with the inter- and counteracting geotechnical and hydrogeological constraints.

RÉSUMÉ: Une tranchée couverte de 340 m a été construite dans la banlieue de Berne dans un terrain d'argiles postglaciaires instables et de sables glaciaires, en présence d'une nappe phréatique peu profonde. Pendant les travaux, le trafic a pu être maintenu au prix néanmoins de quelques déviations par endroits. La technique de construction utilisée est celle du toit avec des parois moulées. L'épaisseur des parois de même que la succession des travaux ont été conditionnées par la nature géotechnique du sol. Des analyses détaillées ont été effectuées à l'aide de forages, d'essais en forage (DMT), d'essais in-situ (CPTU) et d'un monitoring du niveau de la nappe. Des panneaux expérimentaux de la paroi moulée ont été construits et ont fait l'objet d'un suivi. La construction du tunnel a également été l'objet d'un suivi détaillé à l'aide de mesures piézométriques et inclinométriques ainsi que de mesures des tassements et des forces d'étayage. En fonction des résultats de ces mesures, des corrections ont pu être apportées là où cela était nécessaire.

1 INTRODUCTION

Large parts of Switzerland are formed by mountain ranges and the remaining areas are densely populated. The increased demand for individual and public transportation together with environmental and aesthetic requirements lead to underground construction in poorer ground within urban areas. Adequate site investigations combined with in-situ tests and carefully monitored construction become necessary. The integration of all steps and aspects within a major project is crucial to its success. Communication between different planners, contractors and the population is essential. The geotechnical aspects of a recently completed project that was initiated some 15 years ago are described.

2 URBAN SETTING AND HISTORIC DEVELOPMENT

The project is located in the northern suburb of Schönbühl-Urtenen, which lies 10 kilometers north of the city center of Berne, Switzerland's capital. During the age of horse drawn transportation, the first exchange station north of Berne was located there with stables. A town formed over the centuries. The restaurant and hotel were built 150 years ago on good ground. The early roads followed the better ground as long as possible and crossed poor ground the shortest way possible, which led to an alignment that is not always related to topography but linked to geology. The first railways also followed the limits of poor lacustrine sediments with a sinuous alignment like the mainline of the Swiss Federal Railways (Steiner, 1988) and were supplemented some years ago with the new high-speed line through the Grauholztunnel (Scheidegger et al. 1993).

The urban situation, the historic development, the traffic detours necessary and the construction sequences were described by Steiner (1998). The project was the elimination of a grade crossing between a single-track suburban, narrow gauge railway and the former main east-west road (Fig. 1). Also the capacity of the railway was to be increased with a second track. An overpass of the road was rejected due to aesthetic and environmental reasons. A 340 meter long cut-and-cover tunnel partly in poor ground was then designed and selected (Fig. 2)

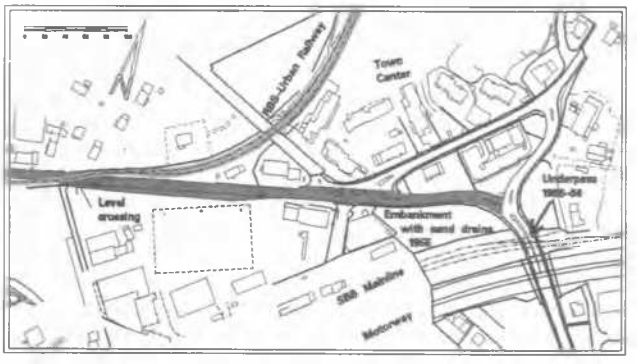

Figure 1. Situation prior to tunnel construction with grade crossing of urban railway with road, showing location of main railway and motorway.

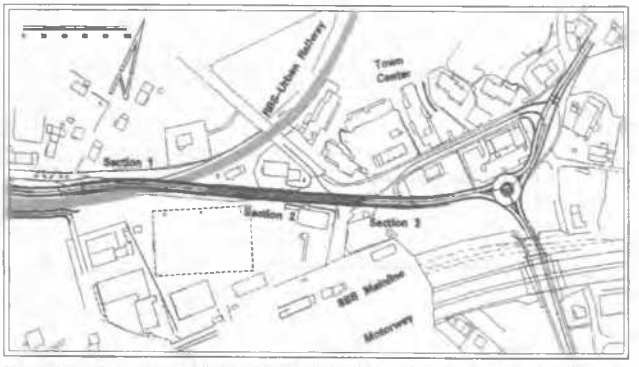

Figure 2. Situation with tunnel and double track urban railway, showing main sections of the tunnel and new road alignment

Two diaphragm walls were constructed with the slurry trench method and a roof slab and/or bracing were adapted to the ground conditions. The walls and the roof of the 340 meter long tunnel had to be constructed in several phases with associated traffic deviations (Steiner, 1998). The constraints on traffic detours resulted also in the necessity of using short temporary sheet piles for the construction of the roof slab.

3 SUBSOIL CONDITIONS

Quaternary glacial deposits overlay tertiary bedrock with variable thickness and heterogeneous quality and a high groundwater table. A detailed subsoil investigation was necessary since the new tunnel reached deeper than existing structures and previous shallower subsoil investigations proved insufficient.

3.1 *Geological framework*

During the ice-ages, the eastern branch of the Rhone glacier, originating in the south-central part of the Alps, flowed in from the west and the Aare glacier from the Bernese Alps to the south. Glacial deposits were created by one or the other glacier or together. Glacial deposits per se are complex and the interaction of these two confluent glaciers left very complex and heterogeneous glacial and postglacial deposits.

3.2 *Local conditions*

Local field names indicate the presence of soft, very compressible soils. The village to the west is called "Moosseedorf" which translates into "Swamp lake village". Many fields carry the part "swamp" in their name. Thus, such names may tell much about possible geotechnical and foundation problems.

The zone includes both glacial and postglacial deposits. The central part, formed by a flat sand ridge was deposited at the margins of the glacier (Fig.3). The lateral parts of the tunnel are in lacustrine deposits formed after the glaciations. Only below 23 to 30 meters depth are coarse-grained and stiff deposits (moraine) present (Fig. 4). The eastern part of the postglacial deposits is formed by sensitive clay and peat. The western part is less sensitive. In the eastern zone some 45 years ago (1954 – 56), a new underpass under the main railway and the motorway was constructed. A small fill for the roadway had to be placed on the soft sensitive clay of variable (a few to nearly ten meters) thickness. A temporary surcharge of 2 to 3 meters was applied and the resulting settlements between 1 and 1.8 meters illustrate the nature of this soil. For this fill, sand drains had been used which was one of the first applications of this technique in Switzerland.

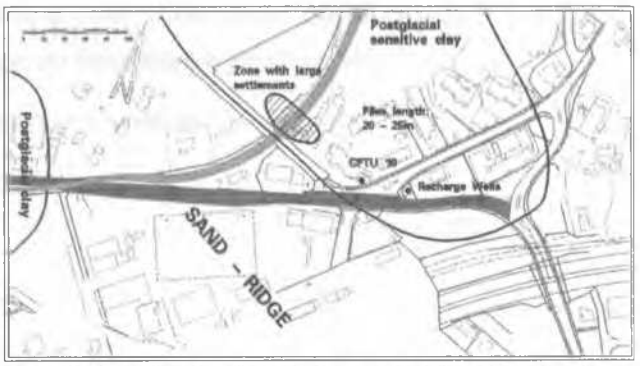

Figure 3. Geologic situation with ice-marginal deposits and recharge wells and zone of large settlement due to localized effect of dewatering.

The new town centre north of the eastern end of the tunnel, built in 1986, required 20 to 25 meter long piles driven through clay into the underlying bearing stratum (moraine).

3.3 *Site investigations*

The site investigations had to address geotechnical and hydrogeologic issues (Steiner & Rieder, 1997) and reach sufficiently beneath the 17 meter deep diaphragm walls, necessary to create a safe excavation. The walls might also form a hydrogeologic barrier in the underground. In 1990 eleven boreholes 15 to 30 meters deep were drilled. In-situ tests (DMT, SPT) were carried out on the boreholes and piezometers installed. Simultaneously, 12 cone penetration tests with pore-pressure measurements (CPTU) were carried out at 12 and 30 meters depth. A delay in construction occurred. An additional five boreholes were carried out in 1994 mainly for monitoring groundwater levels during construction.

The soil strata, even within homogeneous bodies were still relatively heterogeneous. The ice-marginal sand also included layers of silts, the result of the seasonal variable conditions for deposition near the glacier. The evaluation of CPT-U was relatively easy (Steiner & Togliani, 1998) in the sand layer but relatively difficult in the silt layers, since the soil behaved partly drained during cone penetration. The determination of behaviour (drained vs. undrained) was difficult. The friction angle of sand was estimated with the method developed by Durgunoglou & Mitchell (1973).

In the postglacial clay the use of the CPT-U proved very beneficial. The interpretation (SGI, 1994) of a profile of undrained shear strength is shown in Figure 5. The penetration test was carried out adjacent to the town center, constructed four years earlier on driven piles down to a bearing stratum. The local minimum of undrained shear strength between 10 to 15 meters

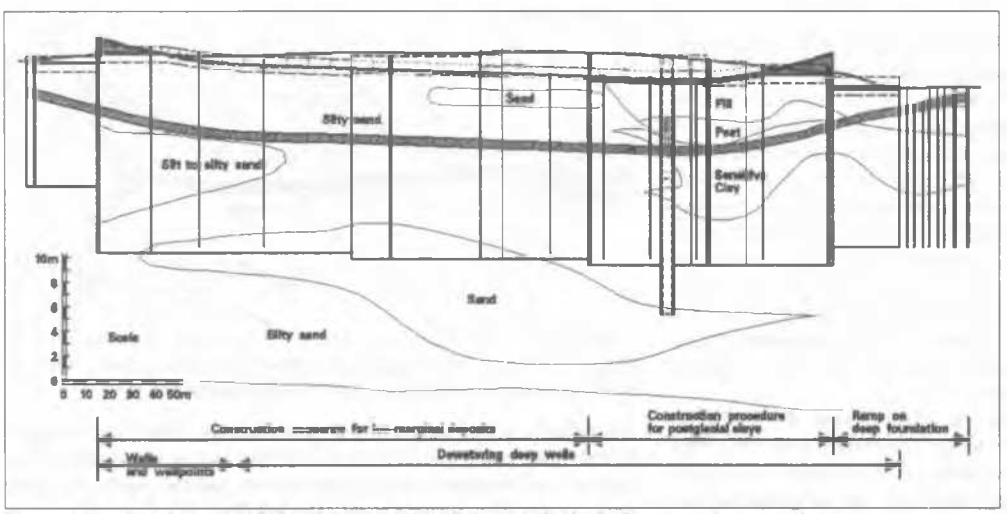

Figure 4. Section along tunnel showing ground conditions, and principal construction procedure.

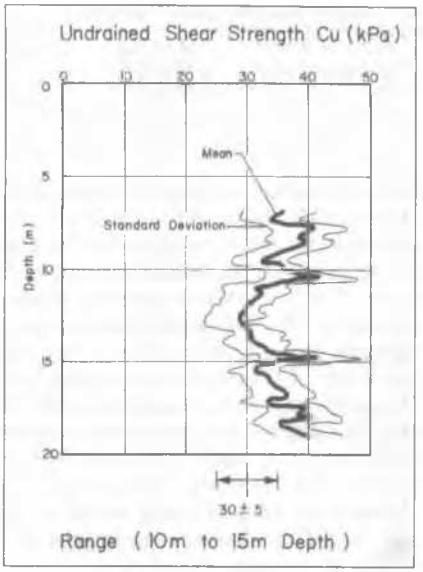

Figure 5: Undrained strength along CPTU 10 adjacent to town center.

Table 1: Summarized geotechnical properties of main soil strata

Soil layer	Unit weight KN/m3	Strength ° or kPa	Modulus E (MPa)
Sensitive clay	16-18	c_u = 20–40	1–3
Postglacial clay	18-20	c_u= 40–60	2–6
Fine to medium sand	19-20	Φ'=36–39	40–50
Silty sand	18-21	Φ'=34-36	35-45
Silt	19-21	Φ'=32-35	20-30

depth in the profile of is the remaining effect of pile driving a few years earlier. The peaks at 10 and 15 meters depth are probably associated with a sandy layer. The clay has an average strength Su = 30 kPa between 10 to 15 meters depth. In an adjacent borehole DMT tests indicated the same undrained shear strength (Steiner & Togliani, 1998). Special samples were taken and consolidated undrained tests were carried out giving similar strength values.

The water table is close to the surface, some one to two meters below the ground surface. The water pressure in the lower, slightly more pervious, sand is slightly above the surface water table by a few decimeters.

4 CIVIL WORKS

The site in the centre of a town required a construction procedure that had a small environmental impact. Sheet piles for the sidewalls had to be excluded due to noise and vibrations. Overlapping pile walls (Secant piles) were excluded due to the numerous joints. The remaining alternative scheme was diaphragm walls constructed in slurry trenches. A roof slab was placed on the diaphragm walls and the remaining construction was carried out underground.

4.1 *Construction in ice-marginal soils*

Principal phases of the construction are shown in Figure 6. Guide walls were constructed followed by lateral diaphragm walls 0.6 meter thick and 4.65 meters long. A preliminary excavation down to the groundwater table followed. On this level dewatering wells at 20 meters longitudinal spacing were constructed and several piezometers were installed in gaps to monitor the effect of dewatering. A dewatering test was carried out to verify that the installed wells could lower the water table to the desired level. In case this test was not satisfactory additional wells were built and the test repeated. The roof slab made from prefabricated elements was placed and completed with cast-in-place concrete. When excavation underneath could be carried out the water table was lowered and the soil excavated. A layer of lean or porous concrete was poured after each working day, which served as bracing of the walls and as protection of the soil. Before placing the bottom slab the piezometers were capped and the wells passed the slab through openings in the concrete slab with flanged closings. These were sealed off at the end and the water table was allowed to rise again.

4.2 *Construction sequence in postglacial soils*

The postglacial deposits were so soft that they could not carry any construction machinery. The material had to be excavated from the surface. The construction phases are shown in Figure 7. Guide walls followed by the diaphragm walls, thickness = 0.8m, length = 3.0 m, were constructed. A small pre-excavation, allowing the placement of the wells in the existing fill followed. After placement of the wells and piezometers a drawdown test was carried out to verify the feasibility of dewatering. The diaphragm wall was connected with a top beam, and every second roof slab was placed as strut. Excavation of the soil was carried out

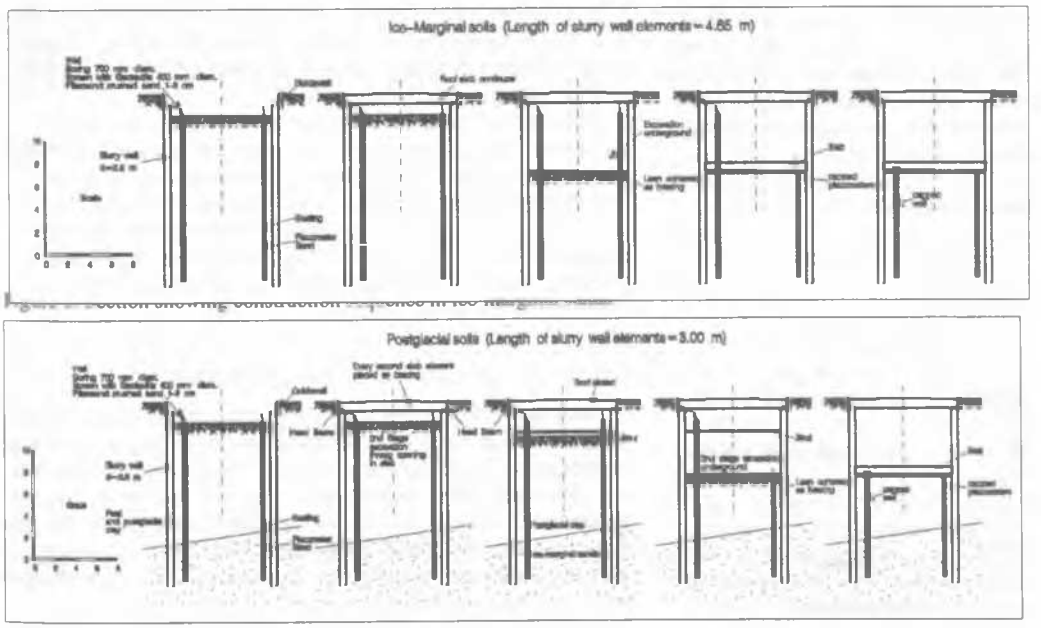

Figure 7. Section showing construction sequence in postglacial clay.

through the openings in the roof and temporary intermediate struts made from steel beams were installed. In a third underground excavation stage the final depth was reached. A protective layer of concrete followed by a slab was poured. The development of the strut forces in the steel beams was monitored and compared to computations.

Not all uncertainty could be eliminated with the extensive site investigations in the section with postglacial clay. It was decided to build three test panels in the postglacial clays ahead of the construction. The test panels formed part of the final construction and their scope was to verify the feasibility of construction of the diaphragm walls.

4.3 *Influence on neighbouring structures*

Deep excavation can influence neighbouring structures. The closest structures are mainly on spread footings on ice-marginal soils. Other effects can be from drawdown of the groundwater table outside the excavation. Along the oldest building made of quarried sandstone the 0.8-meter thick slurry wall was continued in the ice-marginal soils.

5 SLURRY WALL

For the construction of the slurry wall sufficient experience existed, indicating the stability of trenches of approx. 5 meters length, resulting in concrete elements of 4.65 meters. The length of the trenches was a more critical issue in the postglacial clays and little precedent existed in Switzerland. In Norway extensive tests and experience existed (DiBiagio & Myrvoll, 1972) and theoretical models were developed for the stability of slurry trenches (Aas, 1976). Based on a comparison an analysis of the stability of trenches was carried out and test sections built.

5.1 *Probabilistic analysis*

Different methods for the analysis of slurry trenches exist. The Norwegian model (Aas, 1976) was developed for undrained stability in soft clays, and considers anisotropic shear strength, the width and the depth of the trench. The model developed by Huder (1972) does not depend on the depth of the trench and considers isotropic strength only. Models developed in Germany are applicable for use in effective stress analyses (Walz & Pulsfort, 1983) for analysing the stress state on each level. The deterministic model by Aas (1976) was chosen as best suited for the boundary conditions. A probabilistic analysis was carried out using a spreadsheet and a probabilistic add-in program (Palisade, 1997). The undrained shear strength from cone penetration tests CPTU was used (Fig. 5). The layout presented in Figure 8 was used for studying a trench of 3 meters versus 5 meters length, both 14 meters deep. The shear strength was assumed isotropic. The principal results are summarised in Table 2.

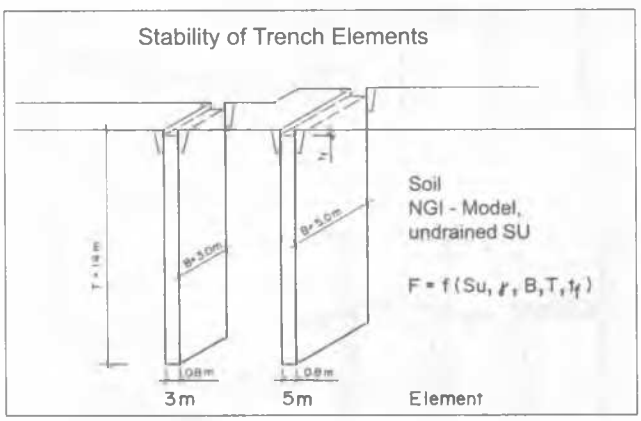

Figure 8. Assumption for stability analysis of slurry trench

Table 2. Results of stability analysis of slurry trenches

Length of trench	Factor of Safety Computed Mean	Probability of failure	Relative Risk
5 meters	1.32	1.9 %	76
3 meters	1.82	0.02%	1

The computed mean factors of safety are relatively close and one might argue that a factor of safety of 1.32 for a five-meter long trench is sufficiently stable based on deterministic judgement. However, the computed probabilities of failure lead to different conclusions. The computed probability of failure for a single trench is close to 2%. The tunnel required over fifty panels of 5-meter lengths in postglacial clay or eighty 3meter long panels. The probability of failure of a single panel during construction of all panels could be calculated precisely with probabilistic methods. This probability is much larger and approaches certainty for the 5-meter long panels. For the 3 meter long panel the probability of failure is nearly two orders of magnitude smaller. The probabilistic analysis clearly indicated that panels of 3 meters length should be constructed in postglacial clay. This length was specified in the tender documents.

5.2 *Layout of Test sections*

The site investigation in 1990 indicated, that the pile driving for the town centre in 1986 had disturbed the postglacial clay. Construction was carried out 1995 and 1996, which led to a further consolidation of the postglacial clay. In order to avoid any surprises test panels were constructed first in the postglacial clays, forming part of the final wall. In case the test panels would not perform satisfactorily, incidental measures could be taken such as using heavier slurry with added barite.

The test panels were located where the deepest layers of postglacial clay were expected. At 1.5 meters from the panel outside the excavation an inclinometer was placed in a 22 meter deep borehole reaching five meters below the 17 meter deep panel. Two pore pressure transducers were placed in a second borehole, 1.5 meters away from inclinometer and panel, in 8 and 11 meters depth in a sand bed into the postglacial clay and sealed. A second inclinometer was placed into a steel casing in the concrete panel and drilling extended five meters below the panel. Five surface settlement points were placed perpendicular to the panel at one meter spacing.

5.3 *Horizontal displacements from inclinometers*

The inclinometer was read twice prior to starting the excavation. The excavation of the panel proceeded in steps of 3 meters. At each stage the excavation was stopped and a reading of the inclinometer was taken (Figure 9).

The displacements from excavation were in the order of 2 millimetres only and occurred a few meters above and below the corresponding level of excavation. After the first stage the soil moved outwards probably due to the presence of a layer of peat, which was compacted from the grab of the excavator. The effect on stability by varying the slurry (bentonite) level in the trench was barely noticeable. Displacements approaching 5 mm were observed after pouring concrete.

5.4 *Movements after construction of test panel*

Monitoring ceased until the excavation of the tunnel continued and was resumed once the inclinometer inside the wall was in operation. The diaphragm wall moved inwards by 10 millimetres (Figure 9) after pre-excavation and the dewatering test, the inclinometer outside in the soil indicated movements for the same time approaching 20 millimetres. Thus the soil outside underwent displacements from the construction of the diaphragm wall panel adjacent to the test panel.

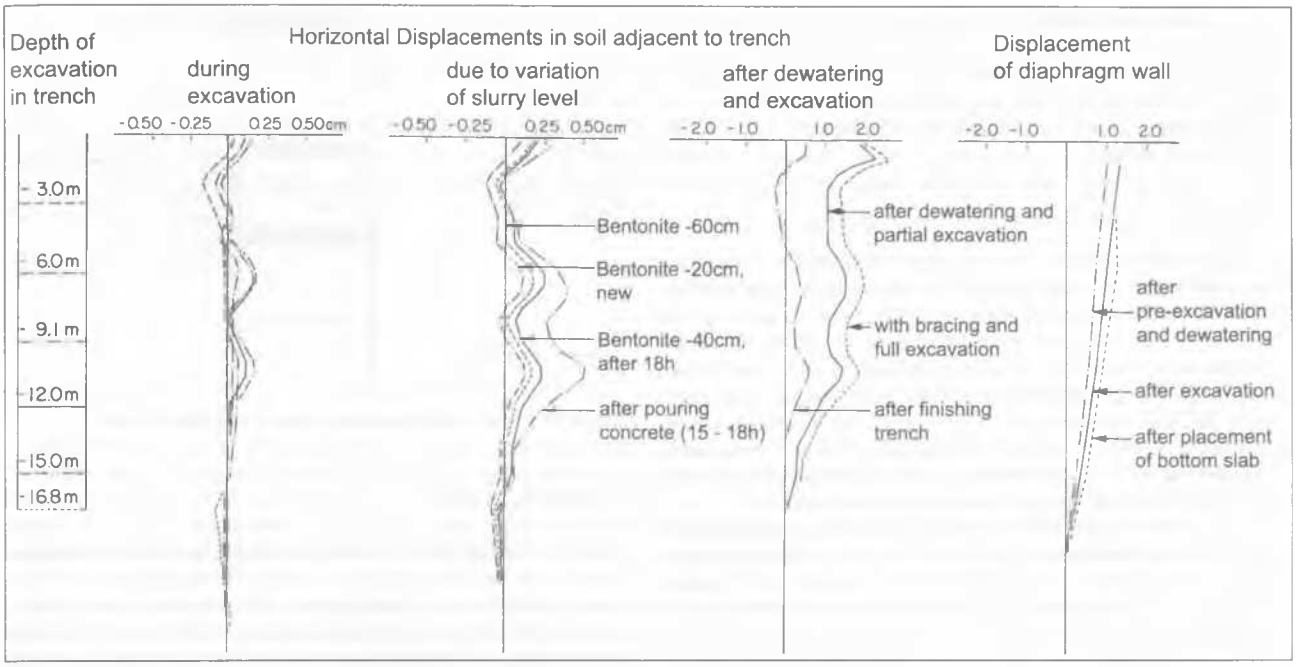

Figure 9. Displacements observed near test panel

5.5 *Variation of pore pressures during trench excavation*

The two pressure transducers were monitored with an automated data acquisition during trench excavation and pouring of the concrete (Fig. 10).

During excavation of the trench one to two meters of additional pore pressures were monitored some 1.5 meters from the trench. During pouring of the concrete additional pore pressures of five meters were induced lifting the pressure head above the ground surface. After the concrete was completely poured the casing pipes were pulled up. This led to a drop in pore pressure by 3 meters below the original level. The total variation of pore pressure was nearly 10 meters. These measurements indicate that soil may undergo substantial changes in pore pressure adjacent to a slurry wall in sensitive clay.

5.6 *Strut forces*

Diaphragm walls and struts in the postglacial clay were designed with the rules of Terzaghi & Peck (1968) for soft clay. Structural capacity of the struts was designed for a worst case (1000 kN per strut). The struts were pre-stressed to the expected (average) load (400kN). Strut loads between 400 and 800 kN were monitored.

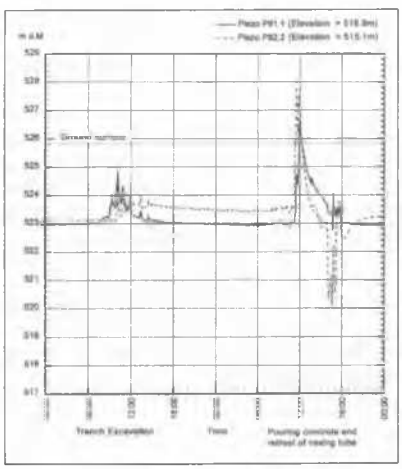

Figure 10. Pore pressure variation in postglacial clay during construction

These loads depended on the actual depth of the sensitive clay. The highest loads were reached at the end of an excavation phase, which had progressed further than given in the drawings, which was accommodated by the design.

6 GROUNDWATER CONTROL AND MANAGEMENT

Groundwater had different effects on this construction. The groundwater had to be lowered within the diaphragm walls to avoid hydraulic uplift. In order to achieve positive control of seepage wells were placed within the deep excavation. The wells were terminated 0.5 meter above the bottom of the panel, in order not to catch water directly from outside the excavation. The depth of the walls was determined by this hydraulic requirement. On the other hand the walls would form a barrier in the ground to natural seepage. On the upstream side water levels may rise and flood basements. On the downstream side water pressures and pore pressures may drop resulting in settlements, particularly in soft soils.

6.1 *Dewatering between walls*

The initially planned dewatering scheme foresaw wells, spaced 20 meters in the eastern part and 10 meters in the western part. Wells of 700 millimeters were bored and equipped with a casing of 400 mm. Filter thickness was 150 mm and crushed sand 1-3 mm was used. The spacing of the wells in the eastern section proved sufficient and groundwater could be lowered sufficiently.

6.1.1 *Effect of vibrations on desanding of wells*

The wells proved to be sensitive to small vibrations. When wells were de-sanded together with the driving by vibrator of short auxiliary sheet piles, inducing vibration velocities of 0.5 mm/s, the wells carried substantial quantities of sand; and de-sanding had to be stopped until the sheet piles had been driven.

6.1.2 *Dewatering ice-marginal silts and sands*

In the western section (Fig. 4) with ice-marginal silts and sands that had probably been pushed by the glacier (so-called glacial tectonics), the wells proved to be insufficient. The monitoring in the piezometers indicated unpredictable flow in the complex ground within the excavation. Based on these observa-

tions it was judged that even wells spaced at 10 meters would be insufficient. Dewatering was then carried out with well points, spaced 1.5 meters, along each wall and drilled to 11 meters depth. The header pipe and the pump was lowered with each excavation stage, thus groundwater could be influenced to the limit depth of the vacuum.

6.2 *Effects of dewatering outside the excavation*

Piezometers indicated that the dewatering inside the excavation influenced the hydrogeologic regime outside more than anticipated. At the bottom of the walls the fine to medium sand was relatively clean and pervious, leading to a drawdown of the groundwater. Recharge wells were placed close to the eastern end. These proved unsuccessful. Water flowed away in the upper layers and the recharge in the lower layer was insufficient to increase the water pressure. The recharged water presumably flowed back to the wells within the excavation and was only recirculated. The recharge wells were then abandoned.

In a localised area on the border between ice-marginal and postglacial sediments, the platform of the urban railway showed settlements exceeding 120 millimetres in an area of 25 meters width (Fig. 3). The settlement stopped once the dewatering was terminated.

7 EFFECT OF VIBRATIONS FROM SHEET PILING

In the central section short sheet piles were necessary in order to guarantee the access to local business and to allow passage of a single roadway. Outside the diaphragm wall six meter long sheet piles were driven with a vibrator. The barely noticeable vibrations (v_p (mean) = 0.5 mm/s) were monitored. The settlements of the adjacent buildings were monitored monthly. No other excavation had occurred in this period. However, widespread settlements were observed on the far side of the buildings. The vibrations did not allow a contemporary desanding of the wells with the driving of the sheet piles.

Table 3: Settlement due to vibration of sheet piles

Building	Side	Distance to sheet piles	Observed settlements
South	Near	7 m	5 mm
	Far	20 m	3 mm
North	Near	10 m	3 mm
	Far	25 m	1-2 mm

8 SETTLEMENTS

Settlements of the adjacent buildings had been monitored and can be attributed to different sources. Observed settlements in the section with sand between the buildings are summarised in Figure 11.

Settlements had been predicted with the empirical method of Clough & O'Rourke (1990) corresponding to a settlement trough of 0.1%. For the near wall of the inn on the southern side of the tunnel, 5 mm settlements were predicted at 6 meters distance and none for the far wall. The actual observed settlements could be attributed to another source other than the tunnel construction as shown in Figure 11. The settlement due to tunnel excavation is 10 mm, which may be mainly due to the horizontal deformations that occurred during the dewatering test when the slab had not yet been placed as a bracing.

The near side of the eastern part of the inn on the south side settled 25 to 34 mm in total. A crack over the entire height of the building existed, indicating that the soft clay was probably present underneath or very close to the northeast corner of this building. The larger settlements can be explained as being due to this soft layer.

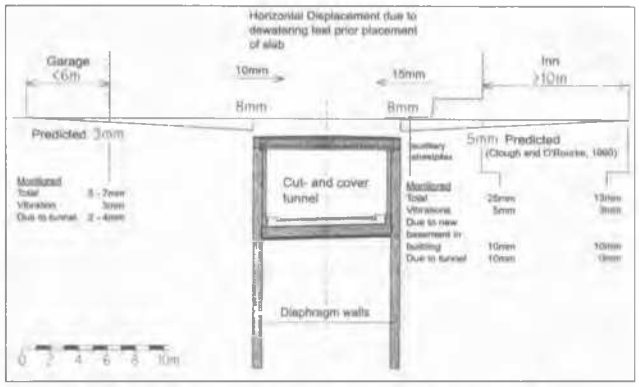

Figure 11. Observed settlements of adjacent buildings in sand

9 CONCLUSIONS

The construction of a cut-and-cover tunnel in complex heterogeneous soils was successful due to detailed underground investigations with in-situ investigations and laboratory tests. In-situ tests and monitoring were also applied. Construction procedures were adapted to the geotechnical risks. The diaphragm wall with top bracing proved to be the solution with the least risks.

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