The Oslo Plaza Hotel - Scandinavias tallest building, founded on slurry wall concrete panels on an extremely steep rock surface

L'Hôtel Plaza à Oslo - Le plus haut édifice de Scandinavie fondé sur des parois moulées sur une superficie rocheuse très inclinée

E.THORN, NOTEBY A/S, Geotechnical Consultants, Oslo, Norway

SYNOPSIS: The tallest building in Scandinavia, the Oslo Plaza Hotel under construction in Oslo, Norway, is scheduled to be opened by 1990. The building, a slender tower 33 stories high, will reach a height of 110 metres. The tower is situated in the new commercial district in Oslo where the soil conditions consist of soft marine clays overlying an extremely steep bedrock surface. The hotel will straddle a subway tunnel that crosses the site.

Large horizontal wind loads require that special attention be paid to the foundations of the tower. Several alternative conoepts for transmitting the foundation loads from the tower to the underlying rock were investigated. The chosen solution consists of slurry wall concrete panels with stressed vertical rock-anchors within the panels.

Due to the extremely steep rock surface the necessary foundation level in the rock was determined by an analysis of the rock slope stability along potential planar slip surfaces. Finite element methods were used to study the final stress distribution in the rock mass. The stability analysis required the adjoining barrette panels to be stepped down an inclined plane of about 45°. The panels vary in depth from 12 m to 30 m, and are chiselled a maximum of 8 m into the relatively weak shale rock. A total of 150 vertical anchors within the concrete panels provide additional forces to balance the overturning moments caused by wind.

1 PROJECT. SITE AND GROUND CONDITIONS

The hotel complex consist of the 33 floor tower block and a 4 storeyed service building, of plan area ca. 1000 sqm and 2300 sqm, respectively. Both buildings also have one basement floor, with the existing piled subway tunnel incorporated in the basement of the service building.

In-situ cast reinforced concrete is used for the main structural frame, the tower being stabilized by shear walls and a central lift core.

The foundations of the tower are designed for a horizontal windload of 65 MN and a total vertical load of 635 MN.

The site in the area of Vaterland in Oslo is reclaimed land at elevation +3, near the present harbour. Below 3 m of fill, the natural ground consists of soft to medium clay over bedrock.

The marine clay of the area is generally normally consolidated. However, at this site the clay is partly reconsolidated material after former slides, resulting in considerable variations in shear strength and the presence of wood particles etc. at depth. At the level of the former shore line, elevation ±0 to -2, the clay is particularly soft with an undrained shear strength C_u of 10 kN/m². Typically the clay of this area is subjected to a secondary consolidation process causing surface settlements of approx. 2-4 mm/yr. The bedrock consists of dark shale and limestone. The rock surface undulates very steeply with depths to rock varying from 12 to 30 m.

The tower block is situated with one front along a 10 to 15 m high drop in the bedrock surface, **Fig. 1 & 2.** Ground investigations showed this gorge in the bedrock to have partly

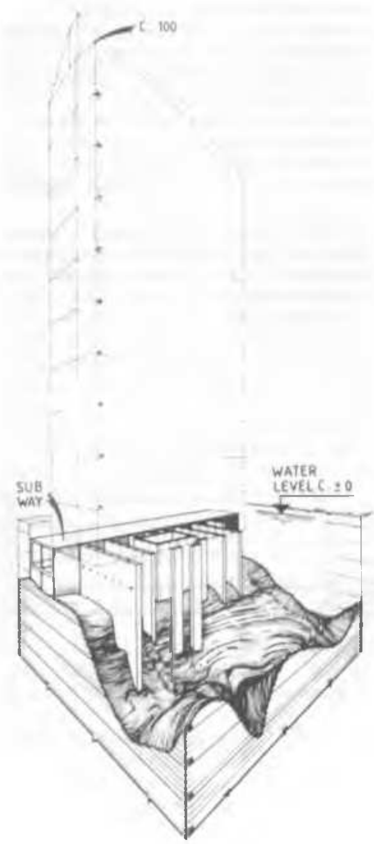

Fig. 1 Isometric view. Tower block foundations

vertical or overhanging sides, shaped by glacial activity which has rendered a smooth rock surface.

This position of the tower was fixed by the restricted site, bordered by the Akerselva river, an elevated motorway and otherwise other developments.

The ground-water level is near the ground surface approximately equal to the level of the river.

2 FOUNDATION DESIGN

2.1 Service building

The four storeyed service building is founded to rock on 700 pcs. high capacity driven, precast concrete piles with concrete quality C75 (75 MPa cube strength) and cross-section, 275x275 mm. The design capacity is 2860 kN. The total length is 12.000 m which is about 5 % of the yearly Norwegian consumption of piles.

2.2 Tower block foundations

The alternative foundation concepts considered for transmitting the heavy horizontal wind forces, moments and vertical tower loads to the rock were as follows:

Alt. 1 Concrete pillars, Ø1.5 m, anchored to the rock by inclined steel piles (Ø190 mm) or prestressed cables (12 pcs. 0.6") as shown to the left in **Fig. 2**. This concept transfers the wind loads to the low service building where the rock conditions are more favourable.

Alt. 2 Slurry wall concrete panels with stressed vertical rock anchors within the panels, in principle a direct extension of the tower structural frame. **Fig. 2**.

Alt. 3 Excavation to bedrock inside a cellular sheet pile structure (Ø12 m) for construction of a massive concrete pillar and direct transfer of forces. Extensive blasting and anchoring is necessary.

Alt. 4 Pillars and inclined steel pipes (Ø180 mm) anchored to the bedrock with prestressed cables (12 pcs. 0.5") within the pipes.

Of these alternatives No. 1 and 2 were investigated in detail.

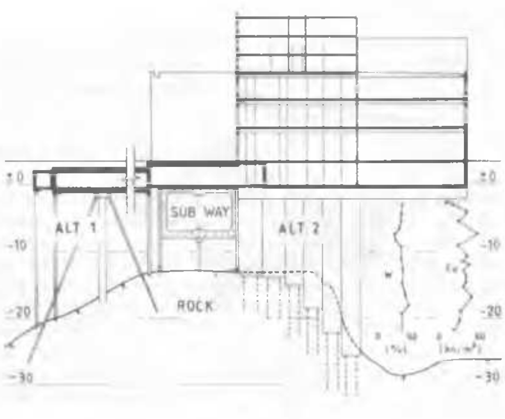

Fig. 2 Alternative foundation systems

2.3 Alternative foundation solution. Alt. 1

An alternative foundation system which should carry the heavy horizontal wind forces and moments was investigated. It consisted, as mentioned, of concrete pillars, Ø=1.5 m, anchored to the rock by inclined steel piles (Ø=190 mm) or prestressed cables (12 pcs. 0.6") as shown to the left in **Fig. 2.**

The ongoing secondary consolidation of the clay will affect such a foundation by increasingly loading and bending the inclined members with time. This effect was studied by the 3 D finite element program **SPACEFRAME (PIRCHER, 1986).**

The lateral and axial displacements of the inclined pile were calculated assuming that the pile is a beam on elastic foundation.

The free displacement components of the soil were determined from vertical secondary compression strains and the equivalent nodal forces, $k_n \cdot y_n$ and $k_s \cdot y_s$ (k_n and k_s are the modulus of subgrade and shear reaction respectively and y_n, y_s are the free soil displacements in the same directions) were applied to the system, along with the external applied loads.

The analysis showed that both the pile and tension cable alternatives were lacking in compression capacity of the inclined members, overstressing the vertical pillars and the tension legs. Regarding use of tensioned cables, the required capacity in compression was found to be so high that it could not be balanced by initial prestress without causing plastic yielding when the cable acted in tension. In the case of piles being used for the inclined members, the analysis confirmed that the bending caused by some years of secondary settlements would cause failure in buckling at very low loads.

Consequently it was decided to use the alternative with reinforced concrete diaphragm walls, anchored to the bedrock, **Alt. 2.**

3. STABILITY ANALYSIS OF BEDROCK. ALT. 2 CHOSEN DESIGN

Two types of stability analysis of the bedrock under the loads applied by the diaphragm walls and the anchorage zone were performed:

i) a global limit equilibrium analysis to determine the depths of embedment for the diaphragm walls and
ii) a finite element analysis of stresses in the bedrock mass to check the local stability condition.

3.1 Material properties

Unconfined (uniaxial) compressive strength in the shale is 60-80 MPa. The bedding has an orientation almost parallel to the rock slope with bedding planes dipping 40-50° N, which is a favourable situation with regards to the total stability of the slope. However, core drillings revealed secondary joints more or less perpendicular to the bedding. Some of the joints also have an orientation parallel to the slope, which could have caused local instabilities. The friction angle of the secondary joints was therefore the governing parameter when the stability of the rock cliff was evaluated. The joints are frequently developed as slickensides.

The shear capacity of the joints were measured on 46 mm rock cores mounted in a shear box. The shear strength was measured for both 0.5 and 2.5 MPa normal stress, and the results were used for calculating the active friction angle, Φ_a, according to the relation:

$$\tau = \sigma_n \mathrm{tg}\Phi_a \qquad (1)$$

$$\text{where} \quad \mathrm{tg}\Phi_a = \frac{c}{\sigma_n} + \mathrm{tg}\Phi_b$$

$$\Phi_b = \text{basic friction angle}$$

The active friction angle varies with applied normal stress as shown in eq. (1). As the average normal stress on the slip plane is about 1 MPa, a characteristic value of **44°** was chosen for the stability analysis.

A material factor of γ_m = 1.3 was used, resulting in a design active friction angle of 34°

3.2 Global stability analysis

The entire rock slope was divided into three parts, and each part treated separately in the stability analysis because of the geometry differences and the load variations along the slope.

The model used in the analysis is shown in **Fig. 3**. The stabilizing effect from the clay pressure acting on the rock slope was taken into account, but not the shear strength of the clay as no displacements were allowed.

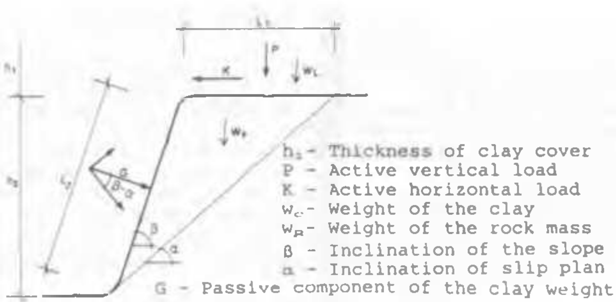

Fig. 3 Global analysis model

The calculations gave a limiting angle for an assumed planar slip surface for which sliding would not take place. This angle varies between 42-44° in the different sections. All wall panels had to be chiselled down to below this slip surface.

3.3 Finite element analysis. Stress distribution

A static analysis of the diaphragm wall was first performed to determine the panel loads, assuming that no shear force can be transmitted from panel to panel along the wall.

A computer program, **SPLATE**, developed at NOTEBY was then used to determine the contact stresses between the base of each panel and the bedrock, assuming that the panel is a foundation resting on elastic half space and tied down to the rock by the anchors.

A finite element analysis was performed to determine the stress and displacement fields in the bedrock.

The element mesh was then loaded with the contact stresses at the surface of the bedrock and with the anchor forces in the rock mass, at the anchorage zone.

Both plane stress and plane strain conditions were investigated to account for stress distribution normal to the diaphragm wall plane.

The results of the finite element analysis using the program **PCFEAP** are shown in **Fig. 4**.

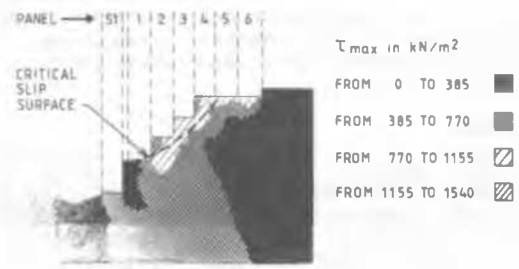

Fig. 4 Finite element maximum shear stress contours and critical slip surface.
Diaphragm wall - axis 4

The contours of maximum shear stress enables the critical failure surface to be detected. Using the state of stresses in each element along the potential slip surface, it was possible to determine both normal and tangential stress along this surface. The safety factor was then calculated as the ratio between the sums of available shear strengths along the slip surface and the mobilized shear stresses.

For the most critical slip surface a factor of safety of **1.44** was obtained.

4. STRUCTURAL DESIGN

In principle the chosen foundation concept acts as an extention to the rock of the four shear walls and central core of the tower, made up by a total of 47 cast in place barrette panels, each 1.2 m wide and 2.8 m long. Panel depths range from 12 to 30 m, including the rock penetration which varies from 0.3 m at the top of the slope to a maximum of 8 m in the steep rock slope.

The single panels of each wall are joined together through a top beam (1.5x1.5 m). This system forms a frame with the top beam providing shear force capacity as there is no shear reinforcement between adjoining panels.

In addition to the vertical tower loads, stressed rock anchors in each panel provides sufficient total forces to balance the tensile stresses from the overturning moment caused by the wind.

The rock anchor tension cables extend to the top of the base slab. Post tensioned Macaloy bars are used to stress the shear walls through the next four floors.

The rock anchor cables, 4 in each panel, consists of 19 pcs. 0.6" Dyform strands greased and coverd by plastic sheathing, each cable having a service load of 3400 kN. The cables have been given double corrosion protection by corrugated PVC-tubes outside the strand bundles.

All cables have a fixed anchorage length in rock of 4.0 m, while the drilled lengths in rock vary from 5.6-10.9 m. They all met the specifications for acceptance during the stressing procedures.

The concrete stresses in the diaphragm walls are high, necessitating a min. concrete design cube strength of 45 MPa.

5. CONSTRUCTION

The foundation works were characterized by a very tight time schedule, the deep and extensive rock excavation, the specified concrete quality and stringent requirements regarding the concrete/rock contact zone below each panel.

5.1 Schedule

Preparatory works, clearing the site and concreting of guide-walls etc. were carried out by the main contractor Berntsen & Boe within 3 weeks. Meanwhile Bachy (UK) mobilized and shipped their construction equipment from England to Oslo.

The barrette construction program was extremely tight to enable the main contractor to complete as much of the groundwork as possible before the worst of the winter weather. The schedule production rate required the finishing of one panel per day.

The diaphragm walls were completed in 9 weeks, working 6 days a week. The rock chiselling operation was on the critical path of the schedule. Work proceeded both day and night during the critical phase. This made great demands on the maintenance of the equipment to avoid break downs.

5.2 Equipment. Procedures

The main rig consisted of 2 pcs 60 tonn cranes working wire grabbs with a capacity of 25 m^3 clay per hour or rock chisels of max. 15 tonn dead wt and 3.0 m drop height, excavation between 0.3 and 1.5 m^3 solid rock per hour.

Panel joints were formed by the CWS principle of Bachy using a cassion beam panel end-form, which may remain in place and guide the excavation of adjacent panels, and which gives a 0.25 m deep key in the panel joint.

A 80 tonn crane was used for general handling of equipment, reinforcement cages, tremie pipes, etc.

The efficient slurry plant with reconditioning facilities ensured that the content of sand and rock contaminants was reduced to less than 1 % by weight before concreting. Specified slurry density was 10.5 to 11 kN/m^3, obtained by 7 % barytt and 3 % bentonite. The guide walls kept the slurry level 0.5-1 m above ground.

5.3 Concreting

The specified concrete class C45 (cube strength 45 MPa) has actually proven to yield 28 day strengths from 55 to 70 MPa. The mix of slump 20 cm contained 390 kg cement per m^3, 20 kg silica and various plasticisers and retarding agents. Max aggregate size was 27 mm.

Drilled cores from 4 panels show a high quality dense and uniform concrete with no signs of segregation or cold joints.

5.4 Concrete/rock interface. Grouting

Experience has shown that residual pockets of slurry may be expected below the concrete at the bottom of the panels. This was checked and repaired prior to the installation of the anchor cables by a procedure of systematic drilling, water jetting and cement grouting through steel pipes attached to the reinforcement cage.

Most panels had voids from 50 to 400 mm against the rock. Clean return water when jetting was required before grouting.

Water jetting and grouting pressures were not allowed to exceed 5 bars due to the danger of initiating channels to the ground surface alongside the panels. However, grout frequently escaped along the panel sides and to the surface at even lower pressures.

In such cases grouting was aborted after a consumption of approx. 1 m^3 grout. After hardening of the grout a new round of drilling, cleaning and grouting was executed. On one occation the conditions were so difficult that the ordinary cement grout was replaced with micro-concrete, delivered through a concrete pump. A satisfactory footing was obtained after a consumption of 9 m^3 of concrete.

6. COSTS

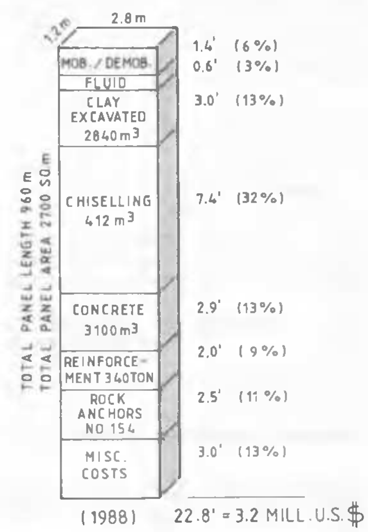

Fig. 5 Slurry wall concrete panels
Total costs mill. NOK, 47 panels

REFERENCES

Athanasiu, C.M (1987). **SPLATE** - A computer program for analysis of contact stresses and settelements of rigid plates on non-linear soil. User's Manual. (In Norwegian). NOTEBY Report 6928/55.

Pircher, H. (1986). **RM - SPACEFRAME**. Static analysis of 3-D frame structures. User's Manual. Teknisk Data A/S, Oslo.

Taylor, R.L. (1986) **PCFEAP** - A personal computer finite element analysis program. User's Manual. University of California, Berkeley.