PROJECT PEGASUS

PFIZER STRÄNGNÄS

GEOTECHNICAL INVESTIGATIONS

TECHNICAL REPORT

FINAL REPORT

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10067436	9G03001	16.Dec.05	Lars Johansson

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APPENDICES

1.1 AIM OF THE REPORT

This report has been prepared in relation to proposed sub-structure foundations and ground preparation works at the Pfizer plant in Strängnäs, Sweden, which are to be carried out as part of the named Project Pegasus. This report summarises the results of geotechnical investigations, describes the geotechnical conditions, and provides information and recommendations to be used in the design and construction of the above proposed work.

The report focuses on the construction of a new building directly south of the head building (Building B200). The extension of the planned building is approximately 50 x 50 m. The report also focuses on the construction of a pipe rack from the northeastern part of the planned building to the purification plant (Building B203) in the eastern part of the area. The total length of the pipe rack is approximately 150 m.

1.2 VALIDITY OF THE REPORT

This report is based on information presented in document No 1G03001 "Report – Geotechnical investigations and topographical survey", elaborated by WSP, and information achieved at site. This technical report shall be read together with the "Report – Geotechnical investigations and topographical survey" and deal with the constructions according to chapter 1.1.

This report contains some examples of calculations, which are marked with italics to separate from advisory text.

2.1 PERFORMED INVESTIGATIONS

WSP has carried out geotechnical investigations at the site during November 2005. Those are;

- CPT (Cone Penetration Test) in 6 points
- HfA (Ram sounding) in 6 points
- Vim (Weight sounding) in 4 points
- Skr (Auger sampling) in 4 points
- Kv (Piston sampling) in 2 points

- Vb (Vane test) in 4 points)
- Rf (Standpipe with filter tip) in 2 points
- Pp (Standpipe with piezometer) in 2 points

2.2 PRESENTATION OF RESULTS FROM INVESTIGATIONS

The results from the geotechnical investigations are presented by WSP in a separate report, Document No 1G03001. The report contains e.g. primary result drawings and laboratory appendices.

2.3 INTERPRETATION OF THE RESULTS

Experienced geotechnical engineers at WSP have interpreted the results from the investigations.

The investigations have included several different methods to find out the characteristics of the soil related to construction activity.

Stability analyses have been carried out with the Swedish programme PostoGRAF version 3.1 containing the calculation software programme Beast.

3.1 DESCRIPTION OF THE AREA

The site of investigation is e.g. presented on drawing G10-01-001 in document No 1G03001.

The site is an area of land immediately south of the existing Strängnäs Plant south of the city of Strängnäs. The new building is to be built on the site of the existing IT block, which is to be demolished and will extend southwards across an existing drainage ditch. The Strängnäs Plant, which is a production facility for medicines, has been in existence since the 1950s.

East from the planned building the drainage ditch is covered, probably since the beginning of the 2000s. The planned pipe rack will pass this area.

Lawns cover the ground and the ground level varies from +5 in the western part to +3 in the eastern. In the western part there are some trees, but the eastern part is open space. The ditch passing the western part of the area has a depth of approximately 3 m.

The site is bounded to the east by the new Svealand high-speed railway line at the shore of the bay Ulvhällsfjärden of the Lake Mälaren, to the north by the existing Strängnäs site, and by an area of undeveloped land to the south beyond which is the Skanska prefab plant. The access to the site is from west and the street Mariefredsvägen.

3.2 GEOTECHNICAL CONDITIONS, IN BRIEF

The geological map (SGU ser. Ae nr 60, 10 H Strängnäs NV 1984) shows that the geological conditions at the area are dominated by glacial clay. This clay probably originates from sedimentation during lacustrine conditions about 10000 years ago. To the north and south from the site there are hills where the upper soil consists of till.

According to geological map (SGU berggrundskarta 1946) the rock at site consists of gneiss, typical for the province of Södermanland where the site is situated, with veins of garnet and limestone.

3.3 GEOTECHNICAL CONDITIONS, IN DETAIL

In this report the soil condition at the site is described through the information achieved in 6 investigation points. The actual conditions may differ from this description, which shall be used as information. The positions of the investigation points are shown on drawing G10-01-001 in the Document No 1G03001. Geotechnical investigations have earlier been carried out close to the extension of the pipe rack and concluded in a report, established by the consultants Bo Orre Markråd AB in January 24, 2000. This information has also been considered in the descriptions above.

In general the following soil layers are found from the ground surface:

Stratum	*Depth (mbgl)*	*Thickness (m)*
Made ground / Artificial fill	0 – appr. 3	0 – appr. 3
Organic topsoil	0 – 0.1	0 – 0.1
Clay	0.1 - 7	2 – 6
Non-cohesive soil / Till	4 – >9	0 – >3

Made ground / Artificial fill
Made ground has been found in 3 of the 4 investigations points where soil sampling has been carried out (point No 1, 3 and 4). The made ground consists in the investigation points of 10 cm of sandy organic topsoil or organic sand, used to prepare the lawn at site. This soil is mainly underlain by artificial fill of sand or clay with a depth of 30 – 90 cm in the investigation points. At the eastern part of the area the ditch has been covered and the depth of the made ground is probably here at least 3 m. According to information achieved at site, pellets of incinerated expanded clay (in Sweden known as Leca) have here been used to decrease the pressure on the underlying clay.

In the area there are also paths and access roads with pavement, but no sampling has been carried out in these, but the artificial fill consists here probably of sand, gravel or crushed aggregates.

Organic topsoil
Organic topsoil has been found upon the made ground as mentioned above. In point No 5 10 cm organic topsoil was found directly on the naturally layered soil. In the other investigation points where sampling has been carried out no signs of naturally layered organic topsoil has been found, therefore it seems as this soil has been taken away before the artificial fill was put on the ground.

Clay
Clay is the predominant soil in the area. As shown above clay can also be found in the made ground, probably it has been excavated from the site of the existing buildings and refilled.

The naturally layered clay can be divided into hard dry crust and soft clay. The limit between the dry crust and the soft clay may indicate the normal level of the ground water table, but other aspects can also influence in the origin of dry crust, e.g. ground frost. At the site an approximately 0.5 m thick intermediate zone between dry crust and soft clay can be found.

The thickness of the dry crust varies between 0.5 and 1 m in the sampling points. This means that the dry crust reaches depths of 1 – 2 m below the ground surface. The moisture content of the dry crust increases from 25 % at the upper part of the layer to 40 % at the bottom. The shear strength of the dry crust may be estimated through vane tests in 4 points and CPT in 6 points (undisturbed samples of dry crust often turn out disturbed and cannot be used for this assessment). The vane tests show shear strengths between 60 and 140 kPa. The evaluation from CPT presents shear strength above 60 kPa down to a depth of 1.3 – 1.7 m below the ground surface, except in investigation point No 5 where the shear strength hardly exceeds 50 kPa.

The soft clay reaches down to a depth of approximately 4 m below the ground surface in the western part of the area and 5.5 m in the southeastern part of the planned building. At the area of the pipe rack the depth increases even more to 6.5 m at the eastern part (7 m according to earlier geotechnical investigations). The clay is generally varved due to different sedimentation conditions during the times of the year. The clay may also contain sulphide due to natural geochemical reactions. The moisture content in the soft clay reaches approximately 65 % and the liquid limit 60 %. The density of the soft clay varies between 1.6 and 1.7 t/m^3 and the sensitivity varies between 11 and 57, which indicates that the clay at certain levels may be highly sensitive.

The shear strength of soft clay may be estimated through vane tests in 4 points, fall-cone tests on undisturbed samples in 2 points and CPT in 6 points. The Swedish Commission on Slope Stability warns of the use of CPT as single method to assess the shear strength in soft clay, and the results show that the evaluated shear strength from CPT in some of the investigation points are considerably higher than the shear strengths settled with the other methods mentioned above. The vane tests and fall-cone tests indicate shear strengths down to 10 – 12 kPa at depths below 3 m from the ground surface.

In investigation point No 6 an interlayer of sand has been found in the clay at 2 – 2.4 m below the ground surface.

Non-cohesive soil / Till
The clay lies on hard non-cohesive soil. It hasn't been possible to penetrate into this soil layer with CPT and only 1 of 4 weight soundings has achieved this, reaching a penetration of 80 cm. The ram sounding is the method that has been used to penetrate the non-

cohesive soil, reaching penetrations of maximum 2 m (3 m according to earlier geotechnical investigations) where the soundings stopped towards boulders or rock. No samples of the non-cohesive soil have been taken, but due to the sounding resistance it can be presumed as hard till.

Rock
There are no proved levels of the rock at site, but the ram soundings may have stopped on the rock surface. Those stops have been obtained at 4 m below the ground surface in the southwestern part of the area and 9.5 m in the eastern part, representing levels of +1 - -5.5 relative site datum.

3.4 HYDROGEOLOGICAL CONDITIONS

The soil conditions at site lead to different hydraulic environments. The made ground and the till may be relatively permeable, depending on the grain size, but the clay can be considered compact. This means that rainwater can infiltrate into the made ground, but will stop on the clay and start to move horizontally along the clay. In the area the ditch will probably receive and drain some of this infiltrated water. The water in the made ground can be characterised as a surface water reservoir and the water table in this reservoir depends strongly on the weather conditions. During the geotechnical investigations no surface water was found.

The ground water table will be found inside the clay, but due to the compactness of the clay this water will not pour into an excavated pit though its bottom is below the ground water table. To find out the ground water table two types of standpipes have been used in four points, two with filter tips reaching into the till and two with piezometers in the clay. There is slowness in the monitoring procedure due to the low permeability of the soil, especially with piezometers. The two piezometers have been monitored twice and the pore pressures represent a ground water table at the level +2.5 in point No 1 (in the western part of the area) and +1.5 in point No 4 (in the eastern part). The filter tip standpipes have been monitored three times. After the installation the standpipes were filled with water, which should sink if the standpipe works correctly depending on the compactness of the soil at the tip. In point No 5 (at the southeastern part of the planned building) the water level has ascended from +1.7 to +2.0 between November 24 and December 8, which is a good indication that the standpipe works correctly. In point No 3 (at the central part of the pipe rack) the water level has descended from +2.9 to +1.6 between November 24 and December 8, which shows that the table probably not had stabilized in November. Between the two monitoring on December 8 and 22 there were small variations in groundwater and pore pressure levels

Earlier geotechnical investigations south of building B200, which means close to the extension of the pipe rack, show some groundwater monitoring. The ground water tables have during the years 1993 and 1999 varied between +0.9 and +1.3, though one monitoring even shows –0.1.

Some 100 meters east from the site the Lake Mälaren is found. The ground water table has a correlation with the water level in the lake due to hydraulic contact. The mean level of Lake Mälaren is +0.6, but the level is regulated at the outflow in Stockholm and the level varies between +0.2 and +1.5.

It seems probable that the ground water table descends towards east and the Lake Mälaren, which the monitoring also shows. The ground water table surely vary during the year and even between the years, but the monitoring has been carried out during too short time to determine the variations.

3.5 CONTAMINATION

The geotechnical investigations show no signs of contaminated soils. The compact clay makes it also almost impossible for any water bound contaminants to infiltrate into the ground and stay at site.

It is known that the Pfizer Plant regularly carries out sampling of groundwater to revise that there is no contamination of the groundwater.

3.6 GROUND CHEMISTRY

The ground chemistry has been investigated in two samples at each end of the area. The samples were taken in the soft clay with moisture contents of 52 – 63 %. The pH of the clay is 7.6 – 7.8, which makes it light alkaline. The resistivity of the samples is 1670 – 2140 Ωcm and the conductivity 0.23 – 0.26 mS/cm. There is presence of sulphide in the clay. The chemical environment may be considered regularly aggressive to concrete (class XA2 according to

SS-EN 206-1). The steel piles shall be design with 2 mm excess for corrosion according to the Swedish "Bro 2004", chapter 31.1.

4.1 FOUNDATION OF EXISTING BUILDINGS

The documentation at the Pfizer archives tells that building B200 has been founded on point-bearing piles. The IT-block, which will be demolished, is constructed on steel piles.

4.2 FOUNDATIONS OF NEW CONSTRUCTIONS, PROPOSAL

The design of the foundation shall be carried out according to the Swedish BKR. The design shall consider Geotechnical Class 2 (Gk2).

Due to the geotechnical conditions at site, we recommend that the planned building and the pipe rack shall be founded with driven point-bearing piles of steel or concrete. The design shall take into consideration both the geotechnical bearing capacity and the load capacity.

Closer than 10 m from the existing building the use of slender steel piles, e.g. X-piles or RR-piles, is recommended to avoid damage on the existing pile foundation due to vibrations and ground compaction. In this case the piling shall start at the existing building and the direction of installation shall be away from the building. Alternatively concrete piles can be installed through preboring to reduce the vibrations.

It has to be taken in consideration that the piles of the IT-block may cause problems during the installation of new piles. Also the possible existence of boulders may complicate the installation.

The lengths of the piles can be estimated through the stop levels at the ram soundings, shown on drawings G10-01-002 and G10-01-003 in document No 1G03001 and earlier geotechnical investigations. The contractor ought to carry out test piling to optimize the pile lengths.

The geotechnical bearing capacity deals with the capacity of the soil or rock at the point of the piles to transmit the load on the pile. This capacity is due to the stiffness of the rock achieved if the piles are driven down to rock and the normal Swedish criterion for driving the pile to refusal is considered. The geotechnical bearing capacity can also be calculated according to the Swedish "Pål-grundläggning", chapter 5.33.3 (the modified pile formula of Kreüger).

$$R = 0.8 * \eta * H * Q_r * (1 - 0.1 * Q_p / Q_r) / (e + 0.5 * c)$$

where

R = the bearing capacity of the pile (kN)
η = factor depending on the type of the pile hammer
H = the stroke length of the pile hammer (m)
Q_r = the weight of the pile hammer (kN)
Q_p = the weight of the pile including drive head and pile-block (kN)
e = sinking that remains after the last stroke (m)
c = springing of the soil, pile and pile-block (m)

The value of η varies normally between 0.8 and 1.0.

The elastic springing c can at a concrete pile be estimated as;

$$c = (R * l_e) / (A_p * E_p) + (R * l_f) / (A_f * E_f)$$

where

l_e = Q_r / q_p (m)
q_p = the weight of the pile per meter (kN/m)
l_f = the length of pile-block (m)
A_p = the skin area of the pile (m^2)
A_f = the skin area of the pile-block (m^2)
E_p = the elasticity modulus of the pile material (kPa)
E_f = the elasticity modulus of the pile-block material (kPa)

The contractor shall through dynamic testing verify the bearing capacity on at least 10 % of the installed piles, well distributed over the area. The choice of the test piles shall be based on the results from the pile driving, which means that where the driving piles to refusal gives suspects of inferior capacity shall be chosen.

The load capacity deals with the capacity of the pile and shall be calculated according the report No 96:1 from the Swedish Commission on Pile Research (unfortunately not available in English). For design the following parameters shall be considered;

c_{uk} = 12 kPa (undrained shear strength of the supporting soil)
δ_2 = 0.1 for concrete piles and 0.2 for X steel piles (factor of reduction to material properties considering soil and rock conditions)

The bearing loads of piles may also be obtained from the producers, which can deliver certified standard piles. *A working load of 900 kN may be expected at a concrete pile with the dimension 270 x 270 mm.*

Settlements of the pile foundation shall be calculated according to "Pålgrundläggning", formula 6.33-4 with M_{sd} = 5 GPa (typical value for till according to table 6.33.:3 in "Pålgrundläggning", the value is 4 – 8 times higher if the pile is driven down to rock). *The expected settlements beneath the pile are less than 5 mm.*

If the lower floor will be placed directly on the ground, the risk of ground settlement must be considered (se chapter 4.3). The ground settlements may be even larger due to the development of dry crust below the building. It is strongly recommended that the load on the ground shall not increase, which may be achieved through the use of light fill, e.g. the Leca mentioned in chapter 3.3 or blocks of expanded or extruded polystyrene. This shall be designed to compensate for 1.2 times the load from the floor and permanent loads on the floor. If e.g. the load is 11 kPa, the compensation shall be 1.2 * 11 = 13.2 kPa. The density of the existing upper soil can be set to 19 kN/m^3, the Leca to 6 kN/m^3 and the polystyrene to 1 kN/m^3. To compensate the load above there is a need of 13.2 / (19 – 6) = 102 cm of Leca or 13.2 / (19 – 1) = 73 cm of polystyrene. The solution with ground compensation shall be compared with suspended ground slabs for the best economical solution.

At the existing ditch certain precautions shall be taken due to the refill and increased loads on the ground. It is here recommended that the floor shall be designed as a suspended ground slab.

As stated above, the recommendation is that the foundation of the building and the pipe rack shall be on piles. However, below is given the conditions for foundation with shallow ground slabs. This may be used for lighter constructions than the planned building and the pipe rack in the investigated area.

The design of ground slabs shall be carried out according to the Swedish "Plattgrundläggning" formula 2.60. As stated in chapter 4.2 the foundation of the new building and the pipe rack shall be with piles, but there may be smaller adjacent buildings that can be design with ground slabs if the settlements calculated according to chapter 4.3 are acceptable. Formula 2.60 gives the bearing capacity q_b (kPa);

$$q_b = c_{u1} * k_1 * N_c^\circ * \xi_c + q * \xi_q$$

where

$$k_1 = \frac{2*h*(b_{ef} + l_{ef})}{b_{ef} * l_{ef} * N_c^\circ} + \frac{c_{u2}}{c_{u1}} * s_c \qquad k_1 < 1.0$$

b_{ef} = effective width of the ground slab (m)
l_{ef} = effective length of the ground slab (m)
h = distance between the ground slab and the bottom of the firm clay (m)
c_{u1} = undrained shear strength of the upper firm clay (kPa)
c_{u2} = undrained shear strength of the lower soft clay (kPa)
N_c° = 5.14
q = overburden pressure at the foundation level (kPa)

ξ_c, ξ_q and s_c are factors considering position of load on the ground slab, depth of foundation, shape of ground slab, inclination of load and inclination of the ground surface according to "Plattgrundläggning".

In this case the following design values shall be applied;

c_{u1} = 60 / ($\gamma_m * \gamma_n$) kPa, where γ_m is a partial coefficient considering material parameters and γ_n is a partial coefficient considering the security class of the construction and shall be decided by the structural engineer
c_{u2} = 12 / ($\gamma_m * \gamma_n$) kPa
γ_m = 1.6
h = shall be chosen considering the limit between the firm and soft clay 2 m beneath ground level

4.3 GROUND SETTLEMENTS

Evaluation through results from oedometer tests in investigation point No 4 shows that the soft clay is consolidated for a groundwater table at the level +1.2, which seems to be an expected groundwater table according to chapter 3.4, therefore the soft clay mainly has to be considered normally consolidated. The preconsolidation pressure can also be estimated with the formula of Hansbo;

$\sigma_c' = 2.2 * \tau_{fu} / w_L$

σ_c' = preconsolidation pressure (kPa)
τ_{fu} = undrained shear strength, in this case determined by fall-cone tests (kPa)
w_L = liquid limit (decimal form)

The results from the formula of Hansbo shows in this case a good correlation with the preconsolidation pressure measured through the oedometer tests with a difference of only 3 kPa.

The evaluation with the formula of Hansbo shows that the dry crust and the clay in the intermediate zone between the dry crust and soft clay are overconsolidated, even strongly overconsolidated. The evaluation at the depth of 2.5 m in investigation point No 4 shows an overconsolidation of 80 kPa or 200 %.

The compressibility modulus has been investigated through oedometer tests on two samples, resulting in the values 216 and 230 kPa. This leads to that a characteristic modulus of 200 kPa can be used for the normally consolidated soft clay (design value 140 kPa). The soft clay has to be considered strongly compressible. The expected ground settlements can be estimated as shown below.

If the load on the ground is distributed over a large area, e.g. through ground profiling, the expected ground settlements, s, at the investigated site can be calculated according to the law of Hook as;

$s = \Delta\sigma * (L - 200) / M$ (cm)

$\Delta\sigma$ = the load on the ground in kPa, e.g. 2 kPa for each 0.1 m of landfill, the formula is valid if $\Delta\sigma \leq 80$ kPa
L = the total thickness of the clay layer in cm (in this formula reduced with 200 cm due to overconsolidation in the upper layer)
M = the modulus of the soft clay (characteristic value 200 kPa, design value 140 kPa)

E.g. a landfill of 50 cm on 400 cm clay leads to an expected settlement of 10 cm and a design settlement of 14 cm.

If the load on the ground is distributed over a limited area, e.g. a ground slab, the load $\Delta\sigma$ is reduced due to the distance between the load and soil where the settlements are calculated. The compressibility moduli differ between the different soil types, and the following ground model may be used (design values);

Soil type	Thickness, h	Compressibility modulus, M
Made ground	0 – 1 m	6500 kPa
Dry crust clay	1 m	7000 kPa
Intermediate clay*	1 m	3000 kPa
Soft clay	1 – 4 m	140 kPa

* *overconsolidated soft clay is included in this soil type*

The values of moduli above are based on experience values, but in the soft clay values measured through oedometer test (see above). The values can be used for estimation of settlements in the investigated site as long as $\Delta\sigma$ not exceeds 100 kPa in the dry crust or 80 kPa in the intermediate clay. Even if the settlements are acceptable, the stability of the construction has to be considered according to chapter 4.2 and 4.3.

The load on the soil shall be reduced depending on the distance between the applied load and the soil. The modification is carried out according to the Swedish "Plattgrundläggning" chapter 2.13 (2:1-metoden) as;

$\Delta\sigma = P / ((z+l) * (z+b))$ (kPa)

P = the total load on the ground (kN)
z = the distance between the load and the centre of the soil type (m)
l = the length of the area affected by the load, i.e. the ground slab (m)
b = the width of the area affected by the load, i.e. the ground slab (m)

The ground settlements can than be calculated as the sum of the settlements in each soil type.

E.g., the load P = 200 kN is applied on a ground slab with the dimension 2 x 2 m 0.5 m below the ground surface. The soil at the site consists of 1 m made ground on 400 cm clay. This leads to;

soil	*z*	*Δσ*	*h*	*M*	*s*
made ground	*0.5 m*	*40 kPa*	*50 cm*	*6500 kPa*	*0.3 cm*
dry crust	*1 m*	*22 kPa*	*100 cm*	*7000 kPa*	*0.3 cm*
interm. clay	*1 m*	*12 kPa*	*100 cm*	*3000 kPa*	*0.4 cm*
soft clay	*2 m*	*7 kPa*	*200 cm*	*140 kPa*	*10 cm*

This example gives the final design settlement of 11 cm.

The ground settlements will develop relatively slowly in the clay, depending on the compactness and layer thickness. Through the oedometer tests the settlement velocity can be estimated as $c_v = 6 * 10^{-9}$ m²/s and the time of development, t, can be calculated as;

$$t = T_v * 0.25 * h^2 / 6 * 10^{-9} \text{ (s)}$$

h = the thickness of the clay layer (m)

T_v is a time factor depending on the requested consolidation, U. The values of T_v are 0.07 for U = 30 %, 0.2 for U = 50 %, 0.4 for U =70 %, 0.85 for U = 90 % and 1.25 for U = 95 %.

In the example shown above the thickness of the clay layer is 4 m, which e.g. means that the development of half of the ground settlements (U = 50 %) can be expected 4 years after the application of the load.

4.4 GROUND STABILITY

The ground stability can be calculated through geotechnical stability programmes. In these calculations the depths of the clay layers and the variations of shear strength can be varied in a better way than in the design in chapter 4.2 according to "Plattgrundläggning".

According to normal Swedish procedures a safety factor in the order of 1.5 normally is accepted, where the safety factor is the quota between retaining and pushing moments.

Calculations show that the horizontal ground may resist a superficial string ground load of 125 kN/m with acceptable security (see appendix A). However this load will decrease due to variations in the ground levels, e.g. the existing ditches or excavated pits. If the load is applied closer than 7 times the depth of the pit or ditch the load has to be reduced. This means e.g. at a ditch with the depth of 3 m, no load of 125 kN/m shall be applied closer than 21 m from the bottom of the ditch (see appendex B).

If ground loads from e.g. cranes are placed closer to pits or ditches this has to be considered though certain calculations of stability. The quota between retaining and pushing moments has to be at least 1.5.

Vehicles for excavation and transport may have to work close to slopes, e.g. at the border of the ditch. Considering a probable ground load of 10 kPa from the vehicles, the slope at a 3 m deep pit or ditch must have an inclination not steeper than 35° to avoid slope failure (see appendix C).

During piling the stability in the area may decrease due to the sensitivity of the clay, why the contractor has to pay certain attention to slopes during these occasions.

5.1 EXCAVATION

Excavation shall be carried out with equipment suitable for the area. The bearing capacity of the ground related to the risk that vehicles get stuck in muddy ground depends strongly on the weather conditions. At dry conditions the bearing capacity will be relatively fair, but at rainfall in may convert in slippery ground due to the high content of fine grain material in the soil as clay and silt. The contractor may to facilitate his work reinforce the ground through e.g. filling with gravel or crushed aggregates to increase the bearing capacity and make the work possible even in rainy weather. The existing paved paths and access roads will represent better ground for transports and may be used. If these paved areas will stay in use after finishing the construction work, the contractor shall be prepared to re-make the pavement because of damages during the work.

The contractor shall carry out excavations considering the stability of slopes. This may vary due to the weather conditions, e.g. a rainfall can cause a failure at a slope that in dry conditions looks stable. Slopes that shall be exposed for long time may be covered. For deeper excavations chapter 4.4 shall be considered.

Water in the excavated pits shall be taken away through pumping. The pumping shall be carried out smoothly to avoid loosening up the soil in the bottom of the pits. The amount of water in the excavated pits depends on the weather conditions, as it is mainly surface and rainwater that will pour into the pit. The presence of sandlayers in the clay, which was found in investigation point No 6, may cause higher inflow of water.

The contractor shall take certain preparations if excavation is carried out below the groundwater table. This table can be considered found at the level +2.5 in the western border of the area and +1.5 in the eastern, but the contractor shall keep himself informed about the actual levels, e.g. through groundwater monitoring. There is a risk of hydraulic up-lift of the pit bottom if;

$$10 * h_w > 15 * h_l$$

h_w = the distance between the groundwater table and the bottom of the clay layer
h_l = the distance between the bottom of the pit and the bottom of the clay layer

If there is a risk of hydraulic up-lift the groundwater table has to be temporarily lowered through e.g. pumping with well points installed in the till. The effects of this measure may be limited due to the compactness of the till and well points may be installed each 10 m to succeed in the lowering of the groundwater table.

The contractor shall pay attention to any changes of colour or smell of the soil as this can indicate the presence of contaminants. If this happens, the contractor shall immediately stop the excavation at the site and a named environmental supervisor shall carry out control and sampling. Any soil that may be contaminated shall not be taken away until the environmental supervisor has delivered a decision. The contractor shall cover the soil to avoid spreading of any contaminants in the area.

The existing IT block will be demolished and this building has probably been founded on piles. The existing piles shall be cut at a depth not closer than 1 m under the floor of the new building. The amount of water may be higher at this building because the groundwater may pour into the pit along the piles.

5.2 FILLING

Filling at paved area shall be carried out with material of class 2 according to the Swedish "AnläggningsAMA 98" table CE/1, which means sand, gravel, sandy or gravely till or crushed aggregate. The content of fine grain material (grain size less than 0.074 mm) shall not exceed 15 % and the content of organic materials not 2 %. The filling shall not be put on frozen ground or organic topsoil. Before filling the ground shall be covered with a cover of geo syntetics according to "AnläggningsAMA 98", chapter DBB "bruksklass 3" or higher.

The refill at paved areas shall be compacted according to table CE/3 in "AnläggningsAMA 98".

The design of the fill at paved areas shall be carried out considering traffic loads and underground according to material type 4B and frost class 3 in "AnläggningsAMA 98", table CE/1.

The ground presents shear strengths of at least 50 kPa down to approximately 1.5 m below the ground surface.

6.1 ENVIRONMENTAL CONTROL

The contractor shall present an environmental supervisor that shall follow the work at site. If necessary the environmental supervisor shall carry out sampling. The contractor shall in his tender present a programme of environmental control and references of the supervisor.

6.2 GEOTECHNICAL CONTROL

The contractor shall present a geotechnical supervisor that shall follow the work at site. The supervisor shall e.g. consider stabilities of slopes at site and the condition of excavated pits. The contractor shall in his tender present a programme of environmental control and references of the supervisor.

The contractor shall during all underground works stay informed about the groundwater levels at site and consider different levels in the western and eastern part of the area (see chapter 5.1).

6.3 AFFECTS ON THE SURROUNDINGS

During the work the surroundings will be affected due to e.g. vibrations, noise and dust.

The contractor shall consider the restrictions of noise established by the Municipality of Strängnäs and special restrictions that may be used at the Pfizer Plant.

Vibrations from piling, excavation and compaction will probably only affect the Pfizer Plant. The Pfizer Plant may before the start of works that may cause vibrations, carry out a revision of the condition of existing buildings. The contractor needs the permission from Pfizer before the start of any work that may cause vibrations.

The vibration levels that can be accepted have to be decided through a special risk analysis at site. The risk analysis shall also consider aspects like e.g. dust, fire, and gases and have to be carried out in collaboration with the Pfizer Plant of Strängnäs.

To avoid damages to existing ground constructions, the piles closest to an existing building shall be driven first and the piling shall move in direction away from the building.

The pumping of water in the excavation pits is not assumed to affect the surroundings. If larger amounts of water flows from the sandlayer in the western part of the area and show no tendency of diminish in one day, the geotechnical controller shall be contacted for advice and decision.

Örebro, December 16, 2005

WSP Civil Engineering
Geo department

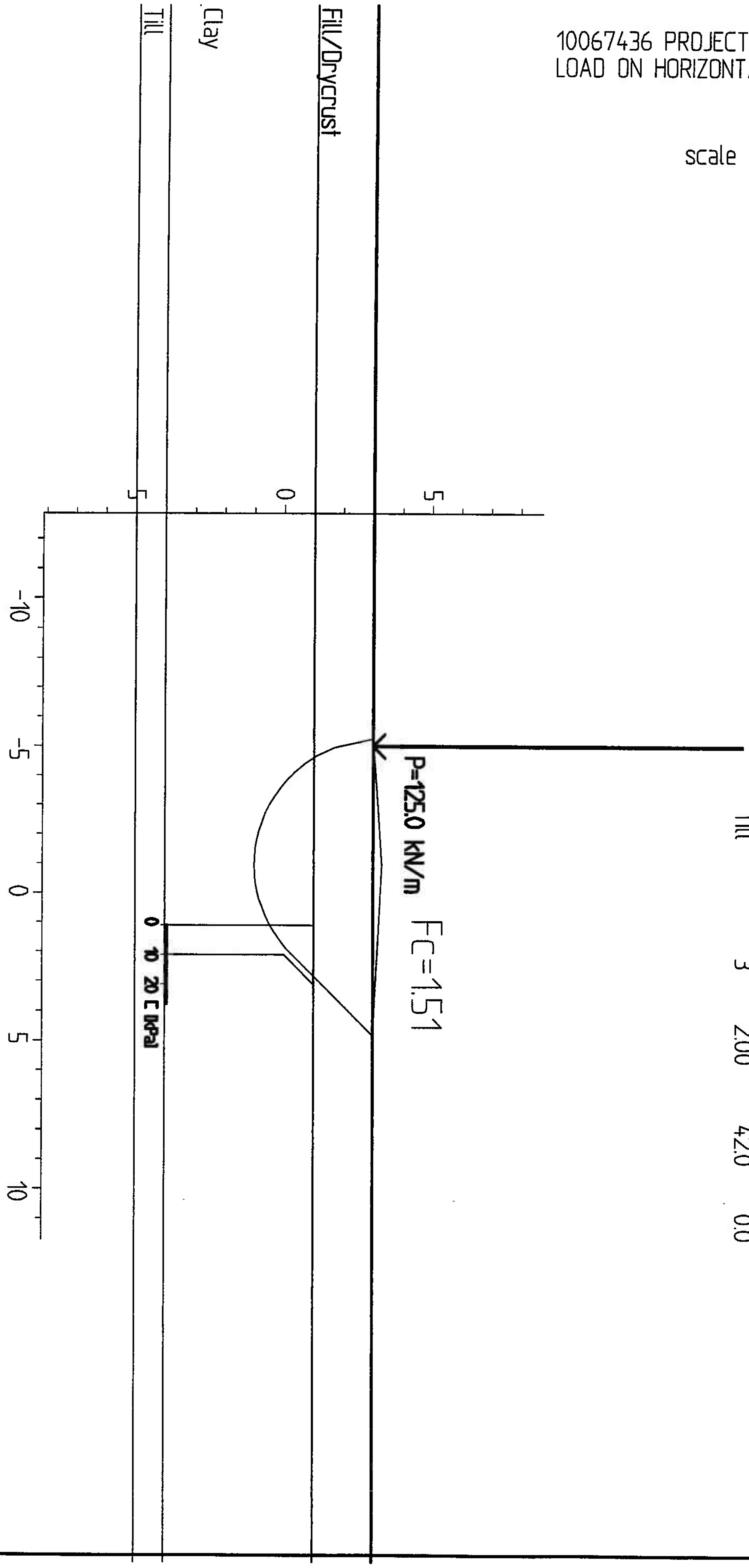
Lars Johansson

WSP, GEOTEKNIK ÖREBRO

APPENDIX A

10067436 PROJECT PEGASUS
LOAD ON HORIZONTAL GROUND

scale 1:200



Material	nr	Density	Fi	C'	C	Aa	Ad	Ap
Fill/Drycrust	1	1.90	---	---	40.0+C	1.00	1.00	1.00
Clay	2	1.70	---	---	C-profil	1.00	1.00	1.00
Till	3	2.00	42.0	0.0				

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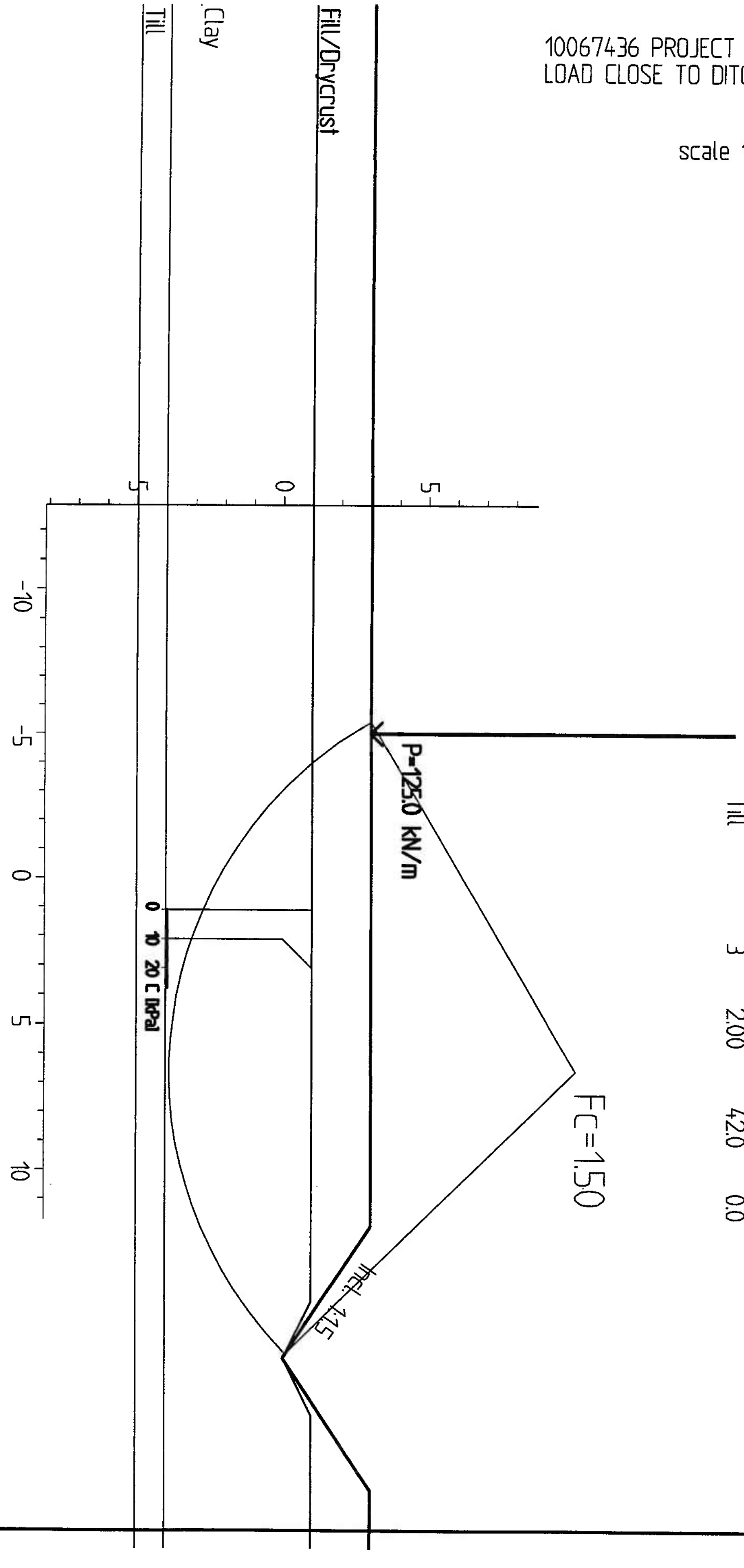
APPENDIX B

10067436 PROJECT PEGASUS
LOAD CLOSE TO DITCH

scale 1:200



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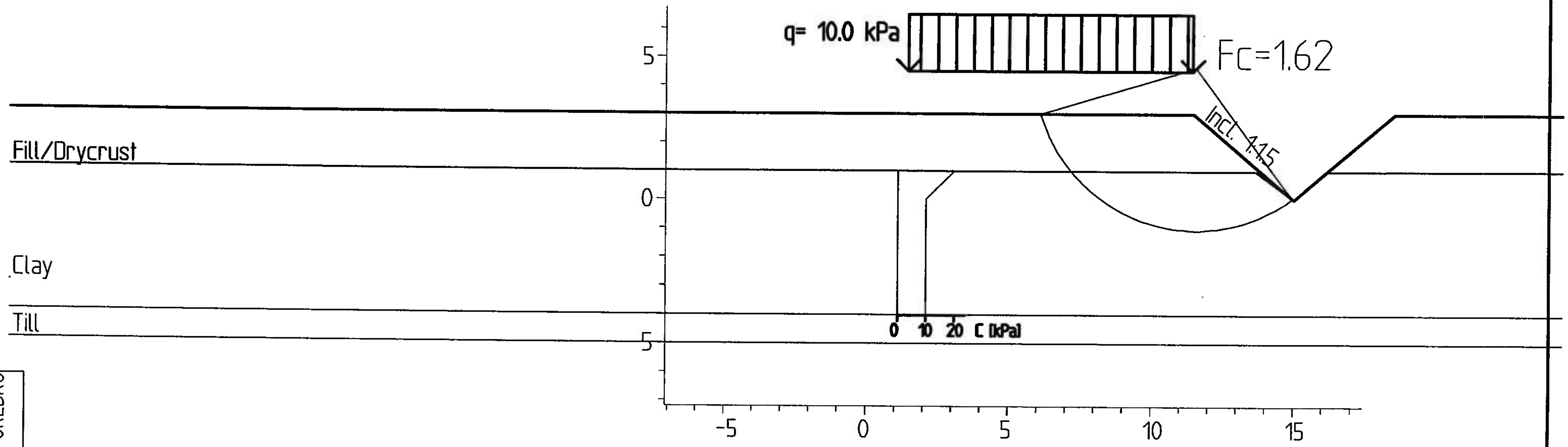
Clay	2	1.70	---	---	C-profil	1.00	1.00	1.00
Till	3	2.00	42.0	0.0				

APPENDIX C

10067436 PROJECT PEGASUS
LOAD AT DITCHES

scale 1:200

Material	nr	Density	Fi	C`	C	Aa	Ad	Ap
Fill/Drycrust	1	1.90	---	---	40.0+C	1.00	1.00	1.00
Clay	2	1.70	---	---	C-profil	1.00	1.00	1.00
Till	3	2.00	42.0	0.0				



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