R-03-30

Design, construction and performance of the clay-based isolation of the SFR silo

Roland Pusch, Geodevelopment AB

September 2003

Svensk Kärnbränslehantering AB
Swedish Nuclear Fuel
and Waste Management Co
Box 5864
SE-102 40 Stockholm Sweden
Tel 08-459 84 00
+46 8 459 84 00
Fax 08-661 57 19
+46 8 661 57 19

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This report concerns a study which was conducted for SKB. The conclusions and viewpoints presented in the report are those of the author and do not necessarily coincide with those of the client.

Abstract

The report describes the host rock performance, recording systems, silo movement, and performance of the wall filling and recording systems. Recommendations for future performance checking are given as well.

Rock movements are concluded to be small but may be intermittent and occur as a result of creep-induced stress accumulation, or earthquakes. They may affect all recordings at SFR.

The silo has undergone slight settlement primarily caused by compression of the bottom bed. It is within the initially assumed range. The analysis can be taken as a basis of the design of the future top fill with respect to the desired continuity of the silo/top-fill/rock system.

The wall fill, which has not undergone significant compression or expansion, will not give swelling pressures that can damage the silo or displace it. No piezometric pressures in the rock walls have been observed, which shows that the drainage system works satisfactorily. Complete water saturation of the entire clay mass will take several hundred years after closing the repository.

In order to certify acceptable performance of the engineered barriers in the future recording of the settlement of the silo, build-up of soil pressure on the silo and rock, and drainage of the rock are recommended. The equipments used presently for the recording turn out to serve satisfactorily.

Summary

The report describes the host rock performance, recording systems, silo movement, and performance of the wall filling and recording systems. Recommendations for future performance checking are given as well.

The measurements of rock movements show that creep is very limited but that instantaneous block movements may have taken place soon after the excavation. Such movements may occur in future as a result of creep-induced stress accumulation, or generated by earthquakes. One cannot draw safe conclusions concerning the magnitude and direction of the vertical component of the strain of the cavern wall, which largely determines the accuracy of silo settlement recordings made by use of reference bolts anchored in the walls. However, it is reasonable to believe that the net movement is directed downwards and amounts at a couple of hundred micrometers to date.

The initial settlement of the silo was somewhat faster than predicted while there is rather good agreement between actual and predicted settlement of the silo top for the period 1991 to 1997 when the waste loading rate was slightly below the assumed lower limit. From 1997 the annual waste loading rate dropped by about 50 % of this limit and gave a theoretical overestimation of the settlement by about 15 %. Considering the uncertainties and conservatism in selecting parameter values for the calculations it can be stated that the agreement between the predicted and actual subsidence of the silo top is acceptable. Major conclusions from the analysis are that the dominant part of the silo top settlement is caused by compression of the bottom bed and that this compression and the silo top subsidence are within the initially assumed ranges. The analysis can be taken as a basis of the design of the future top fill with respect to the desired continuity of the silo/top fill/rock system.

The pressure of the wall fill on the silo and rock has reached a maximum value of 100 kPa, which equals the lower limit for highly accurate pressure evaluation for the utilized Gloetzl cell system and the predicted pressure level. Most of the recorded wall pressures are lower then predicted. The slight, successive pressure increase at the base and top of the silo is probably due to slow hydration but may also be partly or wholly caused by creep effects. Slip of rock wedges can hardly take place because of the lateral support provided by the wall fill but slight movement of a major wedge can occur before resistance is fully mobilized. It can cause minor densification of the fill and a slight increase in pressure on the silo wall. A major conclusion is that such events and build-up of swelling pressures can not damage the silo or displace it. No piezometric pressures have been recorded, which shows that the drainage system works satisfactorily.

Movements yielding some compression or expansion of the wall fill are still small but may be important when the hydration becomes significant, which is not yet the case. Complete water saturation of the entire clay mass may take several hundred years. The wetting process should be of diffusion type as shown by pilot tests in the field and laboratory but local piping by inflowing water followed by self-sealing seems to have taken place as well.

The system of interconnected drains works satisfactorily as demonstrated by the absence of porewater pressures in the pizometers and by the very small movements of the wall fill found by levelling of the cement pavement that covers the wall fill.

In order to certify acceptable performance of the engineered barriers in the future recording of the settlement of the silo, build-up of soil pressure on the silo and rock, and drainage of the rock are recommended. The equipments used presently for the recording turn out to serve satisfactorily in principle.

Sammanfattning

Rapporten beskriver funktionen hos bergmassan och ingenjörsbarriärerna och mätsystemen med fokus på silorörelser och tryck från väggfyllningen. Rekommendationer för att kontrollera framtida skeenden ges också.

Mätningarna av bergrörelser visar att krypningen är mycket liten men att momentana blockrörelser kan ha utbildats någon tid efter utsprängningen. Sådana rörelser kan inträffa i framtiden som följd av krypningsbetingad spänningsuppbyggnad eller jordbävningar. Man kan inte dra säkra slutsatser beträffande riktningen och storleken hos den vertikala komposanten hos bergrumsväggen som i hög grad bestämmer noggrannheten hos mätningen av silosättningen för vilka man använder bultar som förankrats i väggen men det är sannolikt att nettorörelsen är nedåtriktad och uppgår till några hundra mikrometer i dagsläget.

Initialsättningen hos silon blev något snabbare än som predikterats medan god överensstämmelse konstaterades mellan verklig och förutsagd rörelse hos silotoppen för perioden 1991 till 1997 då avfallslagringen var något långsammare än antagits. Från 1997 avtog hastigheten hos denna lagring med ca 50 % och det ledde till en överskattning av den predikterade sättningen med ca 15 %. Med hänsyn till osäkerheter och konservativt val av parametervärden i beräkningarna kan man hävda att överensstämmelsen mellan förutsagd och verklig sjunkning hos silotoppen är acceptabel. Huvudsakliga slutsatser från analysen är att silotoppens sättning i huvudsak orsakas av kompression av bottenbädden och att denna kompression och toppens sjunkning är inom de ursprungligen antagna intervallen. Analysen kan läggas till grund för utformningen av den framtida toppfyllningen med sikte på den önskade kontinuiteten hos systemet silo/toppfyllning/berg.

Trycket som utövas på silo och berg av väggfyllningen har nått ett högsta värde av 100 kPa, vilket är lika med undre gränsen för noggrann tryckmätning för det använda Glötzlsystemet och svarande mot förutsagd trycknivå. Det flesta tryckvärdena är lägre än förutsagt. Den obetydliga, successiva tryckökningen vid silons undre och övre ändar är sannolikt orsakad av bevätning men kan också delvis ha åstadkommits av krypeffekter. Glidning av mindre bergkilar kan inte bli betydelsefull på grund av väggfyllningens sidostöd men en obetydlig rörelse hos en stor kil kan inträffa innan stödfunktionen är fullt utbildad. Den kan orsaka en liten lokal densitetsökning hos fyllningen och en obetydlig ökning av trycket mot silon. En viktig slutsats är att fyllningen inte kommer att ge tryck som kan skada silon eller förskjuta den. Inga vattentryck har uppmätts, vilket visar att dräneringssystemet fungerar tillfredsställande.

Rörelser hos väggfyllningen i form av expansion eller kompression är fortfarande små men kan bli av betydelse när vattenmättnadsgraden blivit hög, vilket ännu inte skett. Fullständig vattenmättnad kan ta flera hundra år efter förvarets förslutning. Bevätningsprocessen bör vara av diffusionstyp enligt fält- och laboratorieförsök men lokal kanalbildning genom inströmning åtföljd av självläkning tycks också ha ägt rum.

Systemet av förbundna dräner fungerar tillfredställande, vilket bevisas av frånvaron av porvattentryck i piezometrarna och av de obetydliga rörelser hos fyllningen som uppmätts genom avvägning hos cementbeläggningen på fyllningens överyta.

För att säkerställa att ingenjörsbarriärerna fungerar tillfredställande i framtiden rekommenderas mätning av silons sättning och av uppbyggnaden av jordtryck mot silon och berget, samt av avbördningen från dräneringen. Utrustningarna som används idag visar sig i huvudsak fungera.

Contents

1 Scope and organisation of the report

The objective of the present report is to describe the construction and performance of the SFR silo and its immediate surroundings (Figure 1-1). Focus is on the engineered barriers, i.e. the concrete silo, the bottom bed on which it rests, and the clay material that surrounds it. The silo was completed in 1987 and the most important factors involved in the evolution of the system, i.e. the silo settlement and the pressure conditions, have been recorded from start. A very important issue is the instrumentation for recording settlement and pressure: the accuracy of the firstmentioned depends on the degree to which the reference bolts remain in their original positions, while the lastmentioned depends on the sensitivity of the gauges. The positions of the reference bolts undergo some change with time because of creep in the rock and this matter, which is closely related to the rock structure and performance, is therefore given considerable space in the report.

The report starts with a short summary of the site selection of the repository and of the regional and local structural geology, followed by descriptions of the design and construction of the silo. Predictions of the major processes, i.e. the settlement of the silo and build-up of soil and water pressures around the silo, and the recording of these processes, occupy the major part of the report.

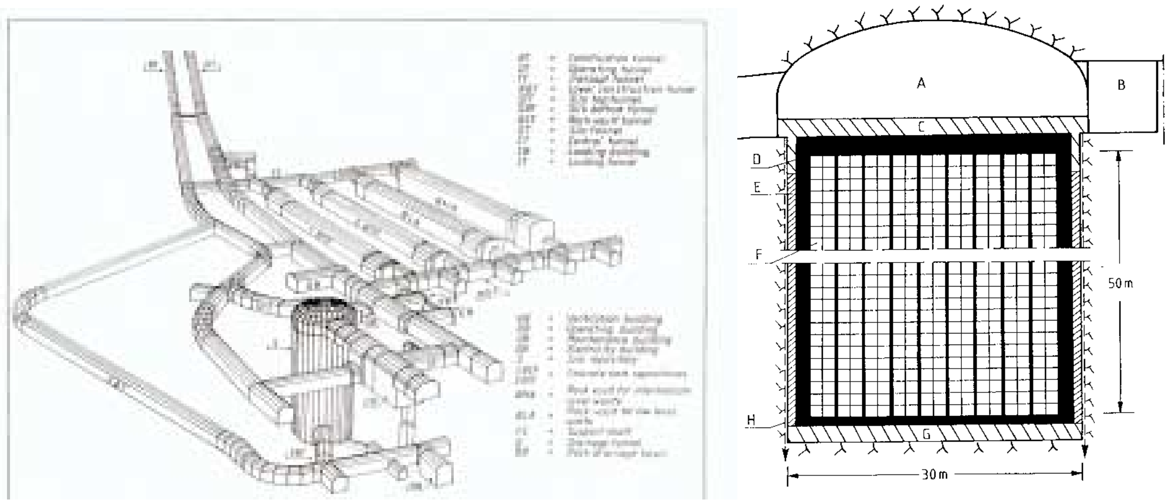


Figure 1-1. *The SFR repository. Left: Layout of the SFR repository with the big silo for intermediate level waste and four big vaults for low-level waste. Right: Schematic cross section of the silo cavern. A) Cement-stabilized sand (not yet defined and applied), B) Concrete plugs, C) Bentonite/sand top bed with gas outlets, D) Concrete silo, E) Wall fill of bentonite granules, F) Waste, G) Bottom bed of bentonite/sand, H) Drains connected to tunnel system.*

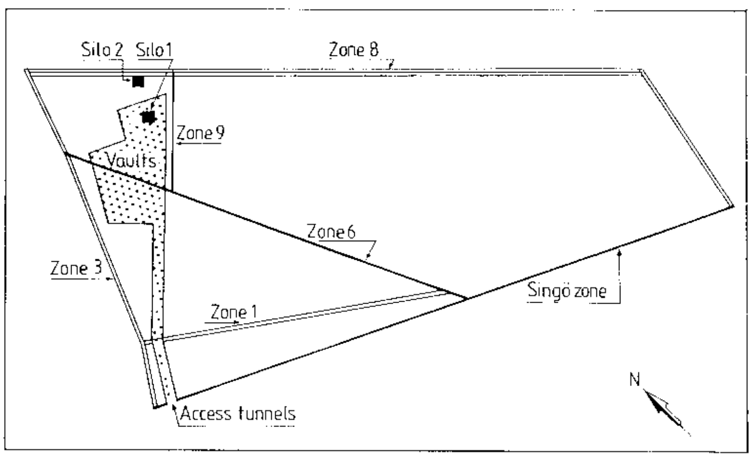
2 Host rock conditions

2.1 Rock structure

The most important criterion for locating the repository was that it should not interfere with any major fracture zone of which several were known to be present in the area. They are oriented NW/SE, NNW/SSE, NE/SW and NNE/SSW and are recognized in a 100 km^2 regional map over the Forsmark area as well in the 3 km^2 mapped area of the finally selected repository area (Figure 2-1). The same pattern is seen on a much smaller scale as illustrated by fracture mapping of the silo cavern.

The hydraulic and mechanical performance of the rock and silo cannot be fully realized without considering the rock structure and stresses. Thus, with the major principal stress being oriented NW/SE, fractures oriented in this direction are the most water-bearing ones, which means that the NW and SE walls should give off more water to the cavern than the others and that hydration and swelling of the bentonite wall fill would hence be faster here when the rock is no longer drained. Water pressure measurements verify the existence of a skin effect causing reduction of inflow particularly from NE and SW /1/. The NW/SE-oriented fractures have chlorite as major mineral coatings, which suggests that creep is most pronounced in the NW/SE direction.

At the time of design and prediction of the hydraulic performance of the silo cavern no attention was paid to the excavation disturbance and the formation of an EDZ with enhanced hydraulic conductivity. With the understanding one has today it is believed that the skin zone may extend from about 3 to 8 m from the cavern while the nearest 3 m zone may have an isotropic conductivity that can be about ten times higher than the average conductivity of the virgin rock /2/. The EDZ is believed to be supplied with water primarily from NW and SE and to distribute water over the periphery of the cavern.



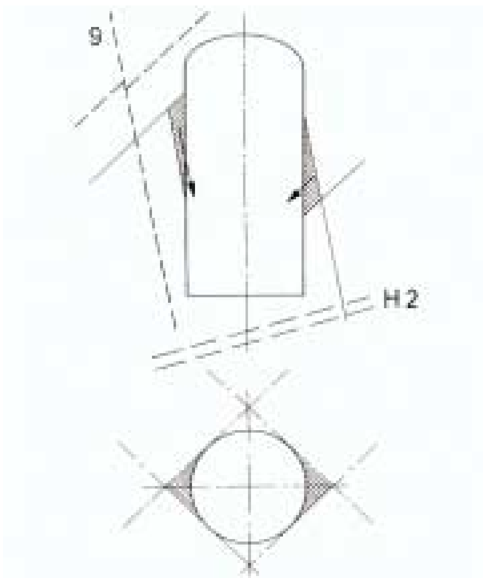


Figure 2-1. Upper: Identification of major fracture zones and location of the silo (Silo 1). Lower: Identified potentially unstable large rock wedges /3/.

2.2 Rock stability

2.2.1 Stress conditions

The major and intermediate principal stresses in the rock mass at mid-height silo are taken to be horizontal with a magnitude of 10 and 5 MPa in NW/SE and NE/SW directions, respectively, while the vertical, lowest, stress is about 3 MPa /1/.

2.2.2 Failure risks

The stability needs to be considered with respect to the risk of rock fall from the walls of the cavern, and to the risk of overstressing the rock, as well as to possible creep strain. The firstmentioned issue is illustrated in Figure 2-1, which shows the location and orientation of one of the major fracture zones (9) along which slip can take place. The inclination of weaknesses of this type and their estimated shear strength expressed in terms of Coulomb failure parameters /4/, as well as the water pressure acting on them, combine to cause a risk of slope failure in the form of slipping of wedges. This can lead to local downward displacement of the cavern walls and of reference bolts anchored in them for recording the settlement of the silo, hence giving a tendency of underestimating the settlement. The same effect may be caused by the fracture-rich and weak EDZ that causes distribution over the entire periphery of local block movements and a tendency of downward movement of the shallow part of the cavern wall.

2.2.3 Creep strain in the rock

Instrumentation

Extensometers are anchored in the rock for measurement of time-dependent rock strain, particularly in the roof and at the upper edge of the vertical wall. The latter ones, of which those termed E4 and E7 will be referred to in this document, are anchored at about 15 m distance from the cavern. The positions of the extensometers are shown by Figures 2-2 and 2-3.

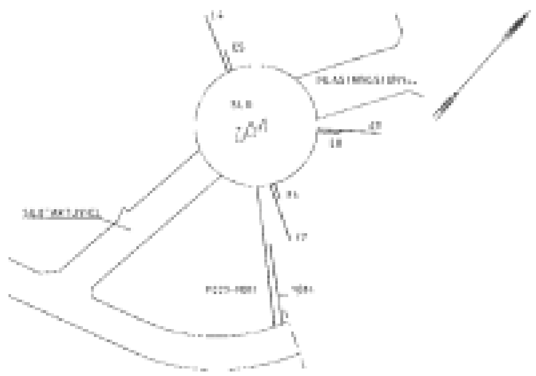


Figure 2-2. *Plan view of the location of the extensometers.*

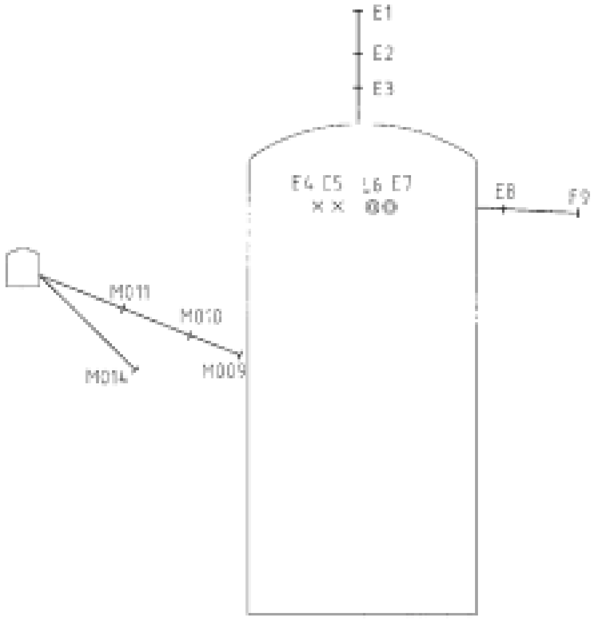


***Figure 2-3**. Section showing the location of the extensometers.*

Theoretical strain

Time-dependent strain will tend to yield convergence of the cavern and thereby lateral compression of the granular bentonite fill and upheaval of the base of the cavern and silo. Attempts have been made by Harald Hökmark, Clay Technology AB, /5/ to calculate the time-dependent convergence of the cavern using the Kelvin model (spring and dash-pot in parallel, /4/), but the uncertaincy in selecting rheological rock data – Young's modulus was taken as E4 MPa and the shear modulus as 4E3 MPa, while the viscosity was set at E12 MPas – make the results debatable. Still, taking them as a rough estimate of the order of magnitude of possible rock strain one can conclude that the maximum total strain caused by the excavation will be about 1 cm appearing in the cupola with about 95 % of the deformation being developed after 150 years. The dominant part of the strain in the rock mass was found to occur within less than 15 m from the periphery of the cavern. The movements imply that reference bolts located at the lower end of the cupola i.e. at the same level as the silo top, will not remain in constant positions and that the subsidence of the silo top, evaluated by measuring the vertical distance between these bolts and reference bolts in the silo, may be somewhat overestimated. No unanimous conclusions concerning the creep strain of the rock and movement of reference bolts anchored in it could be drawn on purely theoretical grounds but the present author estimates that the accuracy of determining the rock bolt positions from measurements to be within the interval +/–200 µm, i.e. +/–0.2 mm in the first 10–15 years.

Actual creep strain

The actual creep movement in radial direction measured and reported by Vattenfall Hydropower /6/ for the period 1985 to 1993 is shown in Figure 2-4 for the two extensometers in the silo cavern that are assumed to have yielded relevant data, namely E4 and E7[1)]. The recordings showed that the creep rate was initially high, i.e. 3–4 mm strain in the first few months, and then strongly retarded as expected for Kelvin-type materials. The behaviour is similar to the predictions and can hence lead to an ultimate radial strain of about 1 cm. However, due to the fact that the measurements did not start until several months after the excavation, the initial strain is not known and the total ultimate strain may therefore be larger.

The shape of the E7 curve indicates that discontinuous strain took place at some time in the first 3 years. Assuming that the strain was not related to the bolt installation (cement/steel/rock reactions) or insufficient accuracy of the measurements it can have been caused by an instantaneous small slip of a large block along some major fracture or by a number of small movements along critically loaded weaknesses.

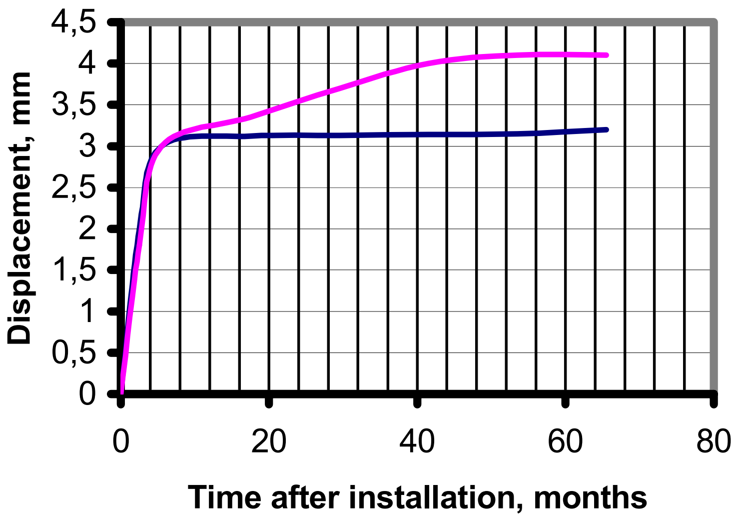


Figure 2-4. *Recorded radial creep of the cavern walls measured by the extensometers E4 (lower) and E7 (upper). The curves are derived by evaluating data in /6/.*

[1)] Most of the extensometers have given a total strain less than 1 mm, a few 2–4 mm and one – questionable (E2) – about 7 mm.

3 Preparation of the construction site

3.1 Rock

A silo with the requested storage capacity and stability occupies a space of about 50,000 m^3 and since rock mechanical stability criteria imply that the diameter of the cavern should not exceed 25–30 m its height must be around 50 m. The finally selected diameter of the cavern was 29 m and the height 50 m at the walls and 65 m from the base to the top of the cupola, which was given elliptical shape. The cavern was excavated by normal blasting, implying that a pervious excavation-disturbed zone with a depth of several meters surrounds the periphery of the cavern.

The rock cavern has been virtually stable since it was excavated and relatively few bolts had to be installed in the construction phase, most of them in the roof. The rock wedge that can be formed in the NW part of the cavern due to the rock structure including Zone 9, may slip after a certain period of time because of accumulated creep strain but the bentonite fill around the silo exerts a stabilizing pressure on it and is expected to prevent large sudden movement of the wedge. However, the counter-pressure may not be sufficient to prevent slight creep along wedge boundaries and this may possibly be manifested by the jerky strain indicated by the E7 extensometer (cf Figure 2-4) and the need for replacing the reference bolt C in September year 1997 for recording of the silo settlement

3.2 Drainage

The planning of the design and performance assessment of the silo had shown that wetting of the clay-based engineered barriers should be allowed as late as possible in order to avoid buoyancy effects that can cause uplift of the silo and risk of non-uniform development of swelling pressures. It was therefore decided to apply a dense network of interconnected fibre drains between the rock and the shotcrete applied on it from the top level of the silo to its base, where they were connected to discharge tubes cast in the concrete base of the silo. The discharge from the tubes, which have open ends in a tunnel below the silo, has been recorded regularly from start for making sure that the drains do not get clogged and cause rise in water pressure. Clear indication of clogging would require pumping of fresh water from below for rinsing the drains.

4 Design and construction of cavern and engineered barriers

4.1 Silo

4.1.1 Design and short-term performance

The design of the monolithic cell structure of the silo, which was constructed by use of a huge and complex slipform, was based on finite element analysis assuming different load constellations. For the design of the 1 m thick silo walls the most critical load constellation implied hydrostatic water pressure on the entire periphery and superimposed on it a swelling pressure of 500 kPa acting on all wall elements or every second element. The thickness of the outer and inner walls and of the base of the silo (1.5 m), and the lack of joints in the monolith, makes it extremely stiff. The stiffness and the low stresses implied by the high safety factor (3-fold) give insignificant strain, which was a major requirement in the design phase for eliminating risk of fracturing by uneven settlement. However, shrinkage and creep as well as impact by heating, combine to cause changes in height of the silo that need to be considered in evaluating the settlement of the silo.

Surrounding the silo with smectitic clay – wall fill – for diverting groundwater flow is a very efficient isolating principle provided that the clay has a much lower hydraulic conductivity than the surrounding rock and that it is not chemically altered to a significant extent. FEM calculations of the SFR case of rock with an average gross hydraulic conductivity of E-8 m/s and smectite-rich clay with a conductivity of E-10 m/s around it showed that no through-flow of the silo will take place under a conservatively taken regional hydraulic gradient i in the area (i<1E-1 m/m). The conductivity of the clay was derived from comprehensive laboratory tests on the selected type of clay, i.e. bentonite from Greece (Milos) converted from its original Ca-form to Na-state by soda treatment at the processing plant. For a bulk dry density of at least 1000 kg/m^3 (1650 kg/m^3 after complete water saturation), which was almost the same for the entire backfill of the granulated bentonite in full-scale tests as well as on site at SFR, the conductivity will be lower than E-10 m/s for saturation and percolation of the strongly-brackish water at Forsmark. The filling was made by use of a 20 cm tube connected to a hopper moved around the periphery of the silo top /7,8/.

The swelling pressure, which will be exerted on the silo walls in the course of water uptake from the surrounding rock after closing the repository, will be 100–150 kPa, i.e. significantly less than the design pressure. The very uniform density distribution achieved will lead to a largely uniform pressure on the silo and surrounding rock.

4.1.2 Longevity

The required minimum time of unaltered isolating capacity of the repository was defined as 500 years. In this period of time one foresees land upheaval by one or a few meters with concomitant displacement of the shore by many hundreds of meters, chemical interaction of clay and concrete, and degradation of the silo walls by corrosion of the reinforcement and dissolution of the cement component of the concrete. The firstmentioned effect will lower the regional hydraulic gradient and hence reduce the risk of percolation of the silo, and the lastmentioned will be rather unimportant for the expected rate of corrosion 3 µm per year. Degradation of the concrete was believed to be most important and led to the development of a very conservative disintegration model with Ca being released from the cement and diffusing out through the clay, replacing the initially adsorbed Na /9/. The release of Ca in the various cement components was assumed to cause loss of coherence of the concrete and hence ultimate collapse. Application of this conceptual model yielded a destruction rate of 1/3 of the concrete walls in 500 years and of destruction of 2/3 in 1000 years. Complete breakdown of the concrete would take 3000 years.

4.2 Bottom bed

4.2.1 Performance criteria and design principles

The bottom bed has a number of important functions:

1. Provide a stable foundation of the silo.

2. Undergo minimum compression in the waste application phase and thereafter, and undergo minimum expansion when the groundwater conditions are restored (recommended interval for settlement/expansion 2–5 cm).

3. Have a hydraulic conductivity that does not exceed the average conductivity of the rock mass (recommended maximum value 1/10 of the average conductivity of the host rock, i.e. E-9 m/s).

The first and second functions require that the soil should be a densely layered friction soil material, while the third implies that it should contain a small fraction of smectitic clay for sealing the voids between the ballast grains without causing significant compressibility and expandability on changes in pressure. Optimum behaviour was concluded to be offered by a 1.5 m thick bed made of a mixture of 10 % Na bentonite clay and 90 % ballast compacted to a dry density of at least 2100 kg/m^3.

4.2.2 Composition, construction, and properties

Composition

The ballast material was basically the same as used for preparation of the concrete but three size distributions were tested for finding the composition that yields the highest dry density. A mixture of 10 % finely ground GEKO/QI bentonite and 90 % ballast without adding water gave the best results, i.e. a dry density of about 2170 kg/m^3

(2370 kg/m^3 at water saturation) /6/. The maximum effective pressure exerted by the silo under drained conditions was estimated at 1.2 MPa while it will drop to about 0.7 MPa after restoration of the groundwater conditions.

Construction

The clay and ballast components were mixed in 500 l free-fall concrete mixers without adding water. The material was applied in 0.2 m layers and compacted in 12 runs by a 3.5 t vibratory roller and a 400 kg plate vibrator for the fill located close to the confining concrete ring. Frequent measurements using water-balloon technique as major method gave an actual average dry density of 2190 kg/m^3, varying from 2140 to 2220 kg/m^3. The density at water saturation will range between 2340 and 2390 kg/m^3 /7/.

Compressibility

Oedometer tests (Rowe) of the 10/90 material with 14 % (natural) water content of the clay material and about 4 % water content of the ballast grains compacted in air-dry form to 2050 kg/m^3 gave a compression of 2.5 % when the pressure was increased from 0.1 to 1 MPa. By unloading to 0.1 MPa the remaining compression was 1.9 %. Applying these data to the 1.5 m thick bottom bed the expected maximum effective pressure 1.2 MPa would give a compression of about 30 mm, while unloading to an effective pressure of 600 kPa would yield expansion and a net compression of about 20 mm. Plate loading tests gave an approximate modulus of elasticity of 150 MPa /3,7/ representing a volume of about 1/10 of a cubic meter. A modulus representing the entire 1000 m^3 volume was estimated at 50–100 MPa considering scale effects.

Hydraulic conductivity

Several series of oedometer tests were made using ordinary oedometers and "mega-permeameters" (oedometers with 0.7 m diameter and 0.3 m height) at different hydraulic gradients using distilled water and Forsmark water as permeates. The average hydraulic conductivity varied from 1.1E-10 m/s for a density at saturation of 2350 kg/m^3 to 3.4E-10 m/s for 2200 kg/m^3 density at saturation /6/. These values were obtained at room temperature for a hydraulic gradient of 1000 (m per m). The influence of the hydraulic gradient, which will only be about 0.02 in a long-term perspective, and of the temperature that prevails in the repository (10–15°C), was determined and it was concluded that the ultimate hydraulic conductivity at percolation with Forsmark water will not exceed E-10 m/s /7/

Gas conductivity

Gas percolation tests using mega-permeameters showed that gas will penetrate the 10/90 % mixture with a density of 2100 kg/m^3 at a gas pressure of somewhat less than 50 kPa when there is no piezometric pressure /7/. The latter pressure will be about 1 MPa in the repository and the critical gas pressure hence about 1050 kPa.

Swelling pressure

The swelling pressure of the 10/90 % mixture with a density of 2100 kg/m^3 has been found to be about 100 kPa for saturation with electrolyte-poor water and less than 50 kPa for ocean water. The same data are believed to be valid for the actual, slightly higher density. It is obvious that the swelling pressure developed by saturation with Forsmark water will have a very small impact on the movement of the silo /7/.

4.3 Wall fill

4.3.1 Performance criteria

The wall fill should have the following functions:

1. Provide lateral support of the rock mass and the silo.

2. Exert a swelling pressure that must be sufficient to resist compression in vertical direction under own weight and not exceed the design pressure 500 kPa.

3. Have a hydraulic conductivity that does not exceed the average conductivity of the rock mass (recommended maximum value 1/10 of the average conductivity of the host rock, i.e. E-9 m/s).

The first and second criteria imply that the soil should be a loosely layered granular bentonite. Large-scale filling tests in the Stripa mine and at Forsmark indicated that the density of granular bentonite fill in the lowest part of the about 2 m wide gap between the silo and the shotcreted rock wall would be about 1700 kg/m^3 after water saturation and 1600 kg/m^3 in most of the gap /8/.

4.3.2 Composition, construction, and properties

Composition

About 6000 m^3 GEKO/QI bentonite granulate – Ca bentonite converted to Na form – with a grain size ranging between 0.1 and 20 mm was used for filling the gap /7/.

Construction

Application of the bentonite fill was made by using a hopper connected to detachable, up to 50 m long 6–8" tubes. The hopper was slowly moved around the periphery of the silo for uniform filling, which was not compacted. The dry density was currently evaluated and found to be about 1000 kg/m^3 for the lower 15 height, 990 kg/m^3 for the 15–30 m height, and 980 kg/m^3 for the 30–50 m height. At water saturation these values correspond to 1650, 1625 and 1600 kg/m^3, respectively.

Hydraulic conductivity

Several series of oedometer tests were made with ordinary oedometers and "mega-permeameters" using distilled water and Forsmark water as permeates. The average hydraulic conductivity varied from 9E-12 m/s for a density at saturation of 1800 kg/m^3 to 6.9E-9 m/s for 1300 kg/m^3 density at saturation with Forsmark water. It was concluded that the hydraulic conductivity will be less than about E-10 m/s for all parts of the fill /7/.

Gas conductivity

Gas percolation tests using mega-permeameters showed that gas penetrated water-saturated bentonite fill with a density of 1700 kg/m^3 at a gas pressure of 50–100 kPa when there is no piezometric pressure /6/. When the groundwater situation is finally restored the piezometric pressure will range between 500 and 1000 kPa, and the critical gas pressure will hence be 550 to 1100 kPa of the wall fill.

Swelling pressure

The swelling pressure of bentonite fill saturated with Forsmark water has been found to be 50 kPa for the density 1600 kg/m^3 and 150 kPa for 1700 kg/m^3 /7/. This means that the swelling pressure is well below the silo design pressure and that it is much more uniformly distributed over the silo periphery than assumed for the design. There is hence no risk of local overloading of the silo walls by swelling pressure exerted by fully water saturated wall fill.

5 Instrumentation for recording of vertical silo movements, wall pressure, and discharge from drains

5.1 Arrangement for measurement of silo movements

Reference bolts were anchored in the shotcreted rock early after the construction of the silo and corresponding bolts were cast in the silo so that its vertical movements can be measured by precision levelling. The accuracy of the recordings, which have been made by Vattenfall Hydropower from the start, has been reported to be +/–10 μm. Data are given in meters with an accuracy of a micrometer. The bolts in the rock are termed A, B, C and D and placed as indicated in Table 5-1 with respect to rock appearing wet or dry in the construction phase (cf Figure A in the Appendix).

Table 5-1. Reference bolts anchored in the shotcreted rock.

Reference bolt	Initial level in 1987	Wet (W) or dry (D) rock	Remark
A	+422.0708	D	
B	+422.3043	W	
C	+422.4624	D	Replaced in 1997
D	+422.1789	D	

On a few occasions the traverse used for waste handling has been in such positions at the recordings that certain measurement could not be performed (1991, 1992, 1996.) and these data are hence missing. The reference bolt C had to be replaced in 1997 because of assumed malfunctioning interpreted from an obvious discrepancy between recorded settlement this year and the previous year. C is located just opposite to the waste loading drift and at the entrance of the silo roof tunnel, which may imply somewhat unstable rock that can possibly have caused movement of rock and bolt C. Otherwise, the system for measuring silo settlement is adequate and assumed to work for a very long time.

5.2 Pressure gauges

Glötzl pressure cells designed for giving accurate total pressures in the interval 100 to 1000 kPa were fixed in niches cut in the shotcreted rock so that the flat cell surface would be in contact with the wall fill. This interval was selected on the ground that the swelling pressure was initially assumed to be in the interval 100–1000 kPa. For the actual soil pressures the accuracy of the gauges is poor, however. Glötzl piezometers were placed at shallow depth in the rock.

The gauges were located as specified in Table 5-2 (see also Figure B in the Appendix). According to the standard procedure the pressure is measured by applying a hydraulic counter-pressure that opens the valves in the cells using oil as pressure medium. Since the stations are not on the same level as the gauges the evaluation of the true pressures requires correction for the oil pressure in the copper tubings, which lead from the gauges to the stations. They will have to be sealed by injection of a long-lasting polymer or other material before the repository is closed since they may otherwise short-circuit the entire wall fill. Strong corrosion of valves and tubings was observed after about 10 years and small electrical heaters had to be installed in the wooden boxes of the stations for eliminating this problem.

Table 5-2. Pressure cells in the shotcreted rock.

Cell	Level	Wet (W) or dry (D) rock	Remark
G1	+369.8	W	Total pressure at the base of the wall fill
G2	+369.8	D	Total pressure at the base of the wall fill
G3	+369.8	D	Total pressure at the base of the wall fill
G4	+370.3	W	Total pressure on the lowest part of the wall
G5	+370.3	D	Total pressure on the lowest part of the wall
G6	+395.0	W	Total pressure on mid-height of the wall
G7	+395.0	D	Total pressure on mid-height of the wall
G8	+417.7	W	Total pressure on the highest part of the wall
G9	+417.7	D	Total pressure on the highest part of the wall
P10	+370.3	W	Piezometer in the lowest part of the wall
P11	+370.3	D	Piezometer in the lowest part of the wall
P12	+395.0	W	Piezometer at mid-height of the wall
P13	+395.0	D	Piezometer at mid-height of the wall
P14	+417.7	W	Piezometer in the highest part of the wall
P15	+417.7	D	Piezometer in the highest part of the wall

5.3 Measurement of water discharged from drains

The water discharged from the V and G drains was collected at intervals for determining the flux, Since the drains are connected one cannot identify from where the water emanates. Drains from the walls of the cavern are termed *V* while those in the floor are termed *G*.

6 Predicted and actual performance of clay-based engineered barriers

6.1 Stability of silo

6.1.1 Stress/strain phenomena

The pressure conditions in the fill are of importance for the overall stability of the silo and rock in the early evolution of the engineered barriers and we will consider three important issues, which all depend on the swelling pressure: 1) the risk of displacement of the silo by non-uniform hydration of the fill, 2) the risk of big rock wedges slipping towards the silo, and 3) the development of pressure against shotcreted rock and within the fill. The two firstmentioned issues concern the stability of the silo while the third has to do with vertical compression or expansion and hence how the pavement on the top of the fill will behave.

1) Hydration of the fill on just one side of the silo, which could result from the nearness of the fracture zone E9 and lead to pressure build-up from NW can theoretically produce a unilateral force that may displace the silo if the swelling pressure is critically high. This force is equal to the product of the silo wall area exposed to the pressure (maximum 1250 m^2) and the swelling pressure. Taking the latter as 100 kPa the force on the silo would be 12,500 tons, which has to be resisted by the shear strength of the bottom bed for avoiding slip. Assuming conservatively that the bed material behaves as a friction soil with a friction angle of 30°, the mobilized counterforce will be the product of the effective normal stress, about 120 t/m^2 (1.2 MPa), the friction coefficient tan30, and the bottom area 490 m^2, i.e. 33,500 t, which yields a threefold safety factor. Such failure can hence not take place.

2) The biggest rock wedge that can become unstable and slip by accumulated creep along a major chlorite-coated discontinuity in the repository rock is estimated to generate a lateral force of 10,000 t on the fill and further to the silo /7/. Like for the non-uniform hydration case this force will not cause any risk of displacement of the silo. The resulting compressive pressure on the fill will be about 200 kPa and hence cause some lateral compression of the fill, which should be measurable using the existing instrumentation. The pressure on the silo walls will be somewhat less than 200 kPa due to stress redistribution by the fill and can therefore, with the swelling pressure added, yield a maximum pressure of around 300 kPa, which is lower than the design pressure. Although slipping of wedges can not threaten the function of the silo it may produce weak instantaneous chocks that may be manifested by very slight instantaneous settlement of the silo and soil pressure changes.

6.1.2 Soil pressure

Neglecting the influence of wall friction one finds that the vertical effective pressure in the lower 15 m height of the fill before hydration takes place would be about 800 kPa. This would lead to compression of the fill and subsidence of the upper surface of the fill, which is cement-paved, by several decimetres /7/. However, the very low width/height ratio (1/20) of the gap implies very significant wall friction and development of "silo" pressure conditions. Using ordinary silo theory and a wall friction angle of the unsaturated bentonite granulate of 26° one finds the vertical effective pressure to be about 37 kPa in the lowest part of the fill and 35 kPa at mid-height, while the lateral pressure would be 100 kPa from the base up to mid-height silo, linearly dropping to 0 at the silo top /10/. This means that the vertical pressure will not cause measurable compression after some initial slight strain. A proof of this would be that the recorded settlement of the cement pavement is negligible if no wetting of the fill has taken place.

In a long-term perspective the fill will be completely water saturated and the effective pressure will change. Using again silo theory and applying a wall friction angle of 5° that is believed to be representative of the hydrated clay /10/, the vertical effective pressure will be slightly less than 200 kPa at the base of the fill, 150 kPa at mid-height and 60 kPa at 5 m from the top of the fill /10/. These pressures are slightly higher than the swelling pressure at the respective level and may therefore lead to some slight compression of the fill, which has to be compensated by application of a limited amount of highly compacted bentonite blocks as part of the top backfill that must be on site before closing the repository for obtaining continuity of the rock/fill system.

6.1.3 Maturation of the clay barriers

Conceptual model

Maturation involves hydration and microstructural homogenization, which is reached long after the wetting has ended due to creep-induced delay in particle reorganization and which may go on in cycles depending on variations in porewater chemistry. Still, most of the practically important physical properties are developed in conjunction with the water saturation that is partly controlled by the rising groundwater pressure after closure of the repository. The time for complete hydration of the bottom bed is estimated to be a few decades.

The wall fill of smectite-rich granules hydrates in two ways: diffusive migration of water, and flow caused by the rising groundwater pressure after closure of the repository. The fact that the shotcreted rock is kept drained for many years does not imply that hydration will not take place until closure takes place. Thus, at the very high relative humidity that is assumed to prevail after complete hydration of the cement in the shotcrete, a rather high degree of water saturation can evolve by uptake of water from water vapour. However, for the highly porous wall fill it is believed that complete water saturation can not be reached before closing the repository – this would require that the space between the hydrophilic clay aggregates would be filled by inflowing water – and the estimated maximum degree of water saturation is hence estimated at 60–80 %. The matter is of great importance for defining the pressure conditions at different times and preliminary modelling of the hydration process has therefore been made /10/ as described below. Comparison of theoretically derived data with actually recorded ones was made 5 years after completing the construction.

Theoretical modelling of the hydration of the wall fill

Evaluation of pilot tests in a big shaft in the Stripa mine, with direct contact between fill of Forsmark type and water-saturated rock /10/, had shown that the general hypothesis of diffusion-type hydration is valid in principle. Thus the increase in water content of the bentonite granulate with an initial water content of 12–17 % at different distances from the rock contact fitted calculations of diffusive water migration based on the diffusion coefficient E-10 m^2/s. This gave a predicted water uptake corresponding to 50 % degree of saturation within 3 decimeters distance from the rock in about 2 years and to 80 % in about 10 years. It would take more than 50 years to reach complete saturation within 1 m distance from the rock. A second estimate was made later assuming the diffusion coefficient to be the same as derived from laboratory tests, i.e. 3E-10 m^2/s and it gave the water content distributions shown as curves in Figure 6-1. It naturally gave a somewhat quicker wetting rate but still a required time of 5 years for reaching more than 50 % degree of water saturation closer to the rock than 0.3 m. Complete water saturation of the entire clay mass would take several hundred years.

The figure also shows the predicted wetting process in the fill by uptake of water assumed to be available at the base of the cement pavement. These data, represented by the fine dots in the diagram, refer to different depths D from the pavement. This case is only relevant if the plastic cover put on top of the fill before casting the cement pavement would leak and there is still no clear indication that such leakage has taken place other than perhaps very locally.

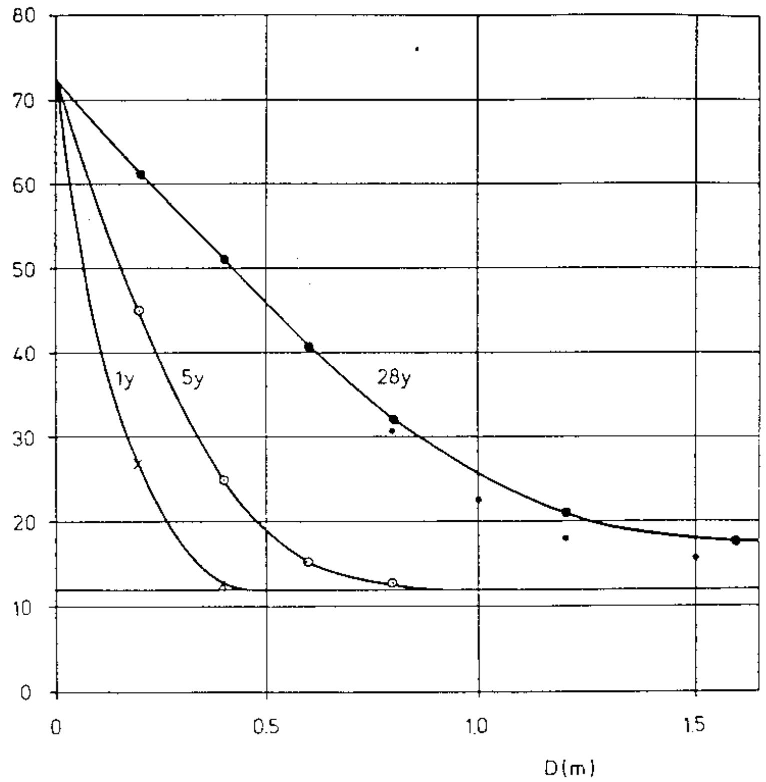


Figure 6-1. *FEM-based prediction of the increase in water content at distance D from the schotcreted rock and the cement pavement. The fine dots represent the calculated water content in the fill at the silo wall assuming diffusive uptake of water only from the cement/fill contact.*

Checking of the hydration process

Five years after completing the construction sampling was made for determining the actual degree of water saturation of the wall fill down to 1 m below the pavement. The determination of the water content gave the data in Table 6-1, which also shows the predicted water content by diffusive water uptake, assuming the shotcrete to be fully water saturated and to provide the wall fill with unlimited amounts of water. Complete water saturation corresponds to a water content of about 60 %. The table shows that there were large variations in degree of wetting and that the clay had taken up some water all the way from the rock to the silo, which shows that water had entered the fill not only by diffusion in liquid form but also through some other mechanism. It may have had the form of inflow from leaks in the shotcrete through channels but the fact that the wetting was not quicker at "wet" than at "dry" rock surfaces indicates that the drain system effectively discharged water from wetter parts and eliminated build-up of water pressures. A more probable process is that, under the prevailing conditions with high RH, water vapour entered open void systems where they fomred continuous paths. Local wetting by either mechanism created water-rich zones that will give off water to neighbouring, drier clay matrix through diffusion and the degree of water saturation will therefore successively become more homogeneous.

The slow rate of diffusive wetting means that the initially developed silo pressure will prevail until water begins to fill the drain system. An early estimate of the saturation rate led to the conservative estimate that the pressure on all levels would increase by about 30–60 % in 5–10 years (1992–1997) and by 50–100 % after complete hydration. The recorded, actual wetting rate and the prediction vizualised in Figure 6-1 suggests that there will be only an insignificant pressure increase in the first 10–20 years.

Table 6-1. Meaured and predicted water contents 5 years after completing the wall fill construction. Data refer to a depth below the pavement of 0.05 to 1 m.

Wet (W) or Dry (D) rock	Distance from shotcreted rock, m	Measured water content, %	Predicted water content, %
D	0.1–0.2	51–59	60
D	0.6–0.8	24–32	25
D	2.0–2.5	18–29	10
W	0.1–0.2	24–39	60
W	0.6–0.8	19–20	25
W	2.0–2.5	19–41	10

6.2 Settlement of silo

6.2.1 Practical importance

One must distinguish between two settlement-related phenomena, i.e. settlement of the foundation of the silo, and subsidence of the top of the silo. The firstmentioned, which is solely caused by deformation of the bottom bed, should not exceed a few centimeters since larger movement could lead to tilting of the silo and problems with the traverse if the movement is not uniformly distributed over the periphery. Subsidence of the silo top, which is expected to deviate somewhat from that of the foundation because of deformation of the silo *per se*, may cause a practically important and unwanted gap between the forthcoming backfill on the silo top cover and the roof of the cavern if it exceeds a few centimetres. If the top backfill will contain expandable minerals and is given a sufficiently high density, a subsidence of the silo top by more than a few centimeters may still be acceptable.

No criteria for the subsidence of the foundation and top of the silo have been defined but it has been proposed that the settlement of the silo top should not exceed 3–5 cm. This means, in turn, that the required accuracy of the settlement measurements does not need to be better than +/– a few hundred micrometers, which is fulfilled by the levelling system used.

6.2.2 Waste load conditions

The weight of the empty silo is about 16,000 t while the ultimate total weight including waste, which is enclosed in steel drums and concrete containers, and porous concrete that is cast in the silo cells for supporting and embedding the waste containers, is about 60,000 t. Soon after completion of the silo repository the annual increase in weight was assumed to be in the interval 1600 to 3200 t per year but the actual rate has been lower in recent years. This matter is dealt with in detail in the analysis of silo movements in chapter 6.2.6.

6.2.3 Predicted compression of bottom bed

Compression of the bottom bed and settlement of the silo foundation

Using data from oedometer tests and assuming the bed to behave as an elastic medium, it was estimated that the 1.5 m thick bottom bed compacted to 2100 kg/m^3 dry density would undergo compression by maximum 3 cm on loading to the 1.2 MPa effective pressure that the silo will finally exert on the bed, and that subsequent unloading to 600 kPa by bouyance at restoration of the original groundwater pressure, would yield subsequent expansion by 0.75 cm to a net compression of 1.5 cm. Since the actual average dry density was 2190 kg/m^3 the ultimate compression is expected to be slightly smaller.

A separate way of predicting the settlement was to perform plate loading tests, which gave an approximate modulus of elasticity of 150 MPa /8/ representing a soil volume of about 1/10 of a cubic meter. A modulus representing the entire 1000 m^3 volume would, by experience, be 50–100 MPa considering scale effects and this would yield an approximate compression of 2 to 4 cm for loading to 1.2 MPa, i.e. about the same as recorded in laboratory testing, assuming the mass to behave as an elastic body. Like all soils, water saturated or not, elastic strain is not developed instantaneously but with some viscous delay. Compression of the bottom bed by about 3 cm is hence expected to be developed both parallel to the filling of the silo with waste and long after.

Rate of settlement of the silo foundation

In principle, the elastic deformation of the bottom bed can be described by using the Kelvin rheological model, i.e. the same as was tried for rough prediction of the creep strain of the rock mass. It implies that the elastic strain, yielding the ultimate total value, is delayed by a viscous component represented by stochastically distributed interparticle displacements. However, as for the rock the selection of representative soil parameters is uncertain and another approach was therefore made. It was based on the fact that most examples of time-dependent settlement of foundations on clayey soil exposed to moderate mechanical stresses are proportional to log time and a preliminary estimate based on measurement of the settlement of the silo top after 7 years (1994) gave the following empirical expression where t is in years:

$$d_{tot} = \Sigma[\Delta_{el}(1+\log 2t_i)\} \qquad (6\text{-}1)$$

where:

d_{tot} = total strain for any time t
Δ_{el} = elastic strain caused by each load step; in practice the annual load increase
t_i = time for creep settlement caused by each load step; in practice 1 year

Application of this expression, which hence implies accumulation of the settlement for successive load steps, gave the expected total strain shown in Table 6-2.

Table 6-2. Expected total settlement of the silo foundation in mm. Estimation on the basis of preliminary model (1994).

Time in years after construction of silo	Empty silo	1600 t annual load increase	3200 t annual load increase
1	4	4 (no load)	4 (no load)
5 to 6	10	10 (start of loading)	10 (start of loading)
10 (1997)	12	18	22
20 (2007)	15	32	40

6.2.4 Factors influencing the evaluated subsidence of the silo top

The subsidence of the silo top is recorded by measuring the distance in vertical direction between reference bolts anchored in the rock (cf Table 5-1) and bolts anchored in the silo wall. Some of the recorded movement is caused by axial shortening of the silo but this effect is counteracted by the heat-induced axial expansion of the silo caused by hydration of the cement mortar cast in the space between waste containers and silo walls. Further impact on the movement of the silo top may be caused by upheaval of the base of the rock cavern, which lifts up the silo relative to the surrounding rock, as well as by compression of the EDZ that would make the silo settle. Load transfer from the wall fill, which exerts downward directed friction forces on the silo may contribute to its settlement but it has been neglected in the analysis. To this comes the possible movement of the reference bolts as discussed earlier in the report. The various effects are summarized and assessed here.

Creep and shrinkage of the concrete

The compressive strain of the silo due to shrinkage and creep of the concrete was predicted in 1987 by the chief designer S Halvarsson, Swedish State Powerboard. The calculations were based on the BBK 79 regulations and gave 1.5 mm axial shortening of the silo in the first year and thereafter an annual average compression by shrinkage and creep of about 0.5 mm until the end of the first 20 year period, yielding a total axial shortening of the silo and hence downward movement of the silo top by 10 mm in this period of time /11,12/.

Heat effects

A numerical calculation of the temperature evolution caused by the hydration of the cement mortar filled in the space between the waste containers and silo walls had been made in 1982 /13/. The conclusion was that complete temperature equilibrium between cement mortar and waste is achieved in about 2 weeks, and that the temperature increase of the system of waste, cement mortar and silo walls at the level where cement casting is made, will be about 15°C. Heat dissipation into the surroundings was not included in the analysis. In 1996 a calculation was made of the axial expansion of the silo generated by heating and it showed that for a cement filling with 5 m height, corresponding to about 240 t cement weight (1420 t waste), the silo would expand axially by 0.75 mm. Assuming proportionality and no heat loss to the surroundings one would expect that the expansions are accumulated and vary from roughly 0.5 mm per year for 200 t cement (1600 t waste) to 1 mm per year for 400 t cement (3200 t waste).

Upheaval of base of cavern and compression of the EDZ

The numerical calculations yielding rock wall movements as described and discussed in chapter 2.2.2 also included estimation of possible upheaval of the base of the rock cavern. Using the same rheological parameters as earlier and taking the load of the silo into consideration, the maximum upward movement of the center of the floor of the cavern was concluded to be 0.2 mm per year in the first 15 years, yielding an upheaval of around 2 mm in this period /5/. Compression of the EDZ, assuming it to be 3 m thick and having a modulus of elasticity of E3 MPa, is of the same order of magnitude and the net uplift of the silo may therefore be neglected. Still, it is conservatively taken as 0.2 mm per year in the present analysis.

Movement of reference bolts

The possible movement of the shallow rock walls discussed in chapter 2.2.2 may have caused vertical movement of the reference bolts such that the accuracy of individual evaluations of the settlement of the silo top can be taken as +/–0.2 mm. Here, the settlement of the silo top is conservatively taken to be 0.2 mm larger than actually evaluated.

6.2.5 Estimated subsidence of the silo top

The various factors affecting the movements of the silo top are specified in Table 6-3.

Table 6-3. Impact of processes on the subsidence of the silo top. Figures in mm. Plus-signs mean downward movement of the silo top and minus-signs the opposite.

Factor	1 year (1988)	5 years (1992)	10 years (1997)	20 years (2007)
Compression of bottom bed	+4	+10	+18* to +22**	+33* to 40**
Creep and shrinkage of silo	+0.5	+3.5	+6	+10
Heat effects	0	–0.5	–3* to –6**	–11* to –16**
Upheaval	–0.2	–1	–2	–4.0
Net downward movement of silo top	+4.3	+12.0	+22* to +19**	+28* to +30**

* 1600 t waste (about 200 t cement) per year, ** 3200 t waste (about 400 t cement) per year

Assuming that the reference bolts have not moved by more than +/–0.2 mm in vertical direction (cf chapter 2.2.2) and that the accuracy of the measurements is better than +/–0.1 mm, the recorded settlement should be nearly the same as specified in the bottom line of Table 6-3.

6.2.6 Recorded and true settlement of the silo foundation

Table 6-4 summarizes the recorded subsidence of the silo top from 1987 to 2002. The silo was completed in February 1987 and the measurements began in April the same year and were repeated in June and October. The last measurement this year hence took place about 0.7 years after the completion of the silo and since the settlement of the foundation after 1 year is predicted to be 4.3 mm the first 3 mm subsidence was not recorded. This figure is therefore added to the recorded values in a separate column (corrected settlement) in the table in order to give the probable total movement of the silo top that has taken place.

Table 6-4. Recorded and predicted subsidence of the silo top, mm.

Year (measured in September-October)	Recorded average subsidence, mm	Corrected subsidence (actual value increased by 3 mm)	Predicted subsidence*, mm	Maximum difference between indiv. measurements, mm	Approx. waste loading rate, ton per year
1987 (0.7 years)	1.8	4.8	4	0.7	<1600
1990 (First half year)	6.9	9.9	6	0.7	
1990 (Second half year)	7.7	10.7	7	1.9	
1991	7.6	10.6	11	1.0	
1992	8.8	11.8	12	0.8	
1993	9.9	12,9	14–15	1.1	
1994	10.8	13.8	15–17	1.1	1000–2000
1995	11.3	14.3	17–19	0.8	
1996	12.0	15.0	18–20	0.7	
1997	13.7	16.7	19–22	0.2	
1998	13.6	16.6	20–23	1.4	800
1999	14.3	17.3	21–24	1.2	
2000	–	–	22–25	–	
2001	–	–	23–26	–	
2002	15.1	18.1	24–27	1.0	

* Interpolation of data in Table 6-3

One concludes from the table that the initial subsidence rate was somewhat higher than predicted while there is rather good agreement between (corrected) actual and predicted subsidence of the silo top in the period 1991 to 1997 when the waste loading rate was slightly below the assumed lower limit of 1600 t per year. From 1997 the annual waste loading rate dropped by about 50 % of the assumed lower limit (annual average of 840 t) and the expected silo top settlement, obtained by interpolation, would therefore be about 19 mm in 1998 and 21 mm in 2002, which corresponds to an overestimation of the predictions by up to 15 %. Considering the uncertainties and conservatism in selecting parameter values for the calculations it can be stated that the agreement between predicted and actual subsidence of the silo top is acceptable. Experience from applied soil mechanics and foundation engineering tells that it is indeed remarkably good.

Major conclusions from the analysis are that the dominant part of the silo top settlement is caused by compression of the bottom bed and that this compression and the silo top subsidence are within the initially assumed range. The analysis can be taken as a basis of the design of the future top fill with respect to the desired continuity of the silo/top fill/rock system. A complete list of the individual recordings appear in Table 6-5.

Table 6-5. Recorded subsidence of the silo top at bolts A, B, C and D, mm.

Year (measured in September-October)	Recorded subsidence, mm.				Approx. waste loading rate, ton per year
	A	B	C	D	
1987 (0.7 years)	1.6	2.3	–	1.6	<1600
1990 (First half year)	6.6	7.3	–	6.7	
1990 (Second half year)	7.0	7.5	7.5	8.9	
1991	7.9	8.6	–	8.5	
1992	8.8	9.3	–	8.5	
1993	10.0	10.4	9.8	9.3	
1994	11.1	11.4	10.5	10.3	1000–2000
1995	11.4	11.7	11.0	10.9	
1996	12.3	12.0	–	11.6	
1997	13.6	13.8	–	13.6	
1998	14.3	13.7	–	12.9	800
1999	14.6	14.8	–	13.6	
2000	–	–	–	–	
2001	–	–	–	–	
2002	15.4	15.4	–	14.4	

Table 6-5 shows that the subsidence of the silo top is very uniformly distributed over the periphery in the last 10 years. The small anomalies illustrate the limited accuracy in recording with special respect to uncertainties concerning possible movements of the reference bolts.

6.3 Soil pressure

6.3.1 Practical importance

The predicted pressure of the wall fill on the silo and bottom bed implies no risk of damage or otherwise unacceptable performance of the silo/rock system. The purpose of recording pressures is to verify that they in fact do not exceed the predicted ones.

6.3.2 Recordings

Table 6-6 summarizes the recorded total and piezometric pressures from 1987 to 2002. Only data for every third year are given because of the insignificant variations. A view of the positions of the gauges is shown in Figure B in the Appendix.

A number of important conclusions can be drawn from the measurements the major ones being:

1. Nowhere the pressure reached the interval for highly accurate pressure evaluation for the utilized system of Gloetzl cells (>100 kPa). The accuracy of most recordings is therefore rather low.

2. The highest recorded value was 100 kPa meaning that there was no influence of a sliding rock wedge, since this would have yielded a pressure of up to 200 kPa.

3. The successive pressure increase at the base and top of the silo is probably due to slow hydration but may also be partly or wholly caused by creep effects.

4. The gauges at mid-height silo have given very low pressures in 1990–1995 and no pressure at all after this period of time. This may be explained by internal movements in the fill leading to unloading through arching. However, it can not be excluded that the these gauges have been out of order from 1995 although it is not obvious from their performance at the measuring operations.

5. At the top and base of the silo the recorded pressures have approached or reached the predicted pressures.

6. No piezometric pressures have been recorded, which shows that the drainage system works satisfactorily.

7. The pressures developed so far are well below earlier predictions (>100 kPa deeper than about 35 m from the silo top) and way below the critical pressure on the silo walls.

Table 6-6. Recorded maximum total and piezometric pressures.

Year (measured in September-October)	Level	Recorded total pressure, kPa	Predicted total pressure, kPa	Recorded piezometric pressure, kPa	Predicted piezometric pressure, kPa
1987	Bottom bed[1)]	40	100	0	0
	Silo base[2)]	20	100	0	0
	Mid-height silo[2)]	0	100	0	0
	Silo top[2)]	5	0	0	0
1990	Bottom bed[1)]	70	100	0	0
	Silo base[2)]	50	100	0	0
	Mid-height silo[2)]	15	100	0	0
	Silo top[2)]	35	0	0	0
1993	Bottom bed[1)]	87	100	0	0
	Silo base[2)]	60	100	0	0
	Mid-height silo[2)]	5	100	0	0
	Silo top[3)]	40	50	0	0
1996	Bottom bed[1)]	80	100	0	0
	Silo base[2)]	70	100	0	0
	Mid-height silo[2)]	0	100	0	0
	Silo top[2)]	30	80	0	0
1999	Bottom bed[1)]	75	100	0	0
	Silo base[2)]	60	100	0	0
	Mid-height silo[2)]	0	100	0	0
	Silo top[2)]	45	55	0	0
2002	Bottom bed[1)]	100	100	0	0
	Silo base[2)]	70	100	0	0
	Mid-height silo[2)]	0	100	0	0
	Silo top[2)]	30	60	0	0

1) Vertical pressure, 2) Lateral pressure

6.4 Movement of wall fill

6.4.1 Practical importance

Wetting of the wall fill can make it expand or compress depending on density and porewater chemistry and recording of changes in the position of the pavement helps to assess to what extent hydration has taken place.

6.4.2 Recordings

Recording has been made since 1990 when reference bolts were installed in the cement pavement near the rock reference bolts A, B, C, and D. The movements of the four measured positions from 1990 and onwards are specified in Table 6-7.

Table 6-7. Movement of the pavement covering the wall backfill in mm. Movement is marked + for subsidence and – for rise. Figure A in A/B is close to silo and B is close to rock.

Time	Point A	Point B	Point C	Point D
1991	+0.8/–0.4	–0.8/0.5	–	–6.9
1992	+1.1–0.7/	+1.1/–11.1	–	–9.5/–2.0
1993	+3.4/–0.5	–0.8/–1.4	–17.3/3.9	–17.8/2.8
1994	+3.2/–0.3	–0.6/–1.9	–24.4/–5.4	–23.7/–3.8
1995	+3.9/0	–0.6/–1.6	–28.1/–5.5	–26.7/–3.7
1996	+6.0//+32.0	+0.7/–0.5	–	–33.6/–3.9
1997	+8.0/+1.0	+1.0/–2.0	–	–42.0/+1.0
1999	+7.5/+1.0	0/–1.0	–	–37.0/–4.0
2000	+7.0/+8.0	+2.0/–1.0	–	–40.0/–4.0
2001	+8.0/+1.0	+2.0/–2.0	–	–41.0/–5.0
2002	+8.0/+1.0	+1.0/–2.0	–	–42.0/+1.0

The table shows that the movements are very moderate and have not exceeded about 42 mm. At Point A the settlement is largely downwards, while there is a general tendency of upward movement at Point D. At Point B, which is the only part that is located in "wet" rock, there has been very little movement indicating that the drainage has worked well and prevented significant wetting of the wall fill.

One concludes that the movements do not give definite information of whether heave or subsidence will take place in conjunction with water saturation of the wall fill. While upward movement must be related to wetting of the uppermost part of the fill, downward movement is probably due to creep-controlled compaction and redistribution of stresses in the fill.

Since 1996 the presence of fractures in the pavement has been examined and it has been shown that the number fractures has increased successively from about 10 to about 25. The majority of them have been open at most inspections the rest being sealed by precipitation of calcite emanating from the cement. The fractures are usually oriented oblique to the silo and rock walls (45°) and are caused by uneven movement of the underlying wall fill. Since the cement pavement, which is often covered with water especially in the fall, has been cast on plastic sheets and the fractures do therefore not supply the fill with water except where the sheets may have been broken.

6.5 Rock wall drainage

6.5.1 Practical importance

The current recording of the discharge from the tubes connected to the drainage system indicates whether clogging of the drains is taking place and also gives a measure on changes in piezometric pressures and groundwater flow to the silo cavern.

6.5.2 Recordings

Table 6-8 exemplifies typical recorded discharge from the tubes in the tunnel below the silo base (period 1992 to 1999). The drains in the floor are discharged by the G-tubes and transfer only very little water, while the wall drains, which are discharged through the V-tubes, give off considerable quantities. Certain significant changes in flow are found for some of the V-tubes but they all appear to work. The fact that the drains are all coupled means that clogging of a single drain causes transfer of water to neighbouring drains and it is therefore of greater interest to consider changes in total flow for drawing conclusions concerning the drainage function than to examine individual tube discharge figures.

The successive drop in total flow with time is demonstrated by Figure 6-2. One finds that it has proceeded since 1992 with a peak in late 2001 and a minimum value in late 2002. The annual variations are believed to depend on short- and long-term differences in precipitation, while the general trend mirrors the effect of successively dropping piezometric pressures caused by the draining function of the cavern and drifts. There is no indication of malfunction of the drain system by clogging or other wise.

Table 6-8. Recorded outflow in ml/min from the discharge tubes in the tunnel below the silo.

Tube	92-02-03	93-02-08	94-02-08	95-02-15	96-02-02	97-03-10	98-03-03	99-03-08
V11	10	10	12	15	20	18	15	13
V12	7	7	455	340	240	260	400	240
V13	1000	760	220	225	240	220	210	170
V14	10	10	14	4	4	1	3	3
V15	730	800	420	370	360	360	300	330
V16	9	8	40	1	5	2	4	3
V17	9	12	2	2	0	1	4	4
V18	40	30	670	470	660	580	430	400
Sum	1815	1636	1833	1427	1529	1443	1366	1163
G19	0.3	–	–	0	0	0	0	0
G20	0.6	0.3	–	0	0	0	0	0
G21 & G22	0.3	0.3	0	0.3	0	0	0	0
G23	0	0	–	0	1.2	0	0	0
G24	1.8	2.1	1.8	1.5	0	0.3	0	0
G25	1.5	1.5	–	0	0	0	0	0
G26	0.2	0.3	–	0	0	0	0.3	0
G27	0	0	–	0	0	0	0	0
G28 & G29	1.8	3.6	0.3	0	0	0	0	0
G30	0.6	0.3	–	0	0	0	0	0
Sum	7.1	8.4	2.4	1.8	1.2	0.3	0.3	0
Total V+G	**1822.1**	**1645**	**1835**	**1429**	**1530**	**1443**	**1366**	**1163**

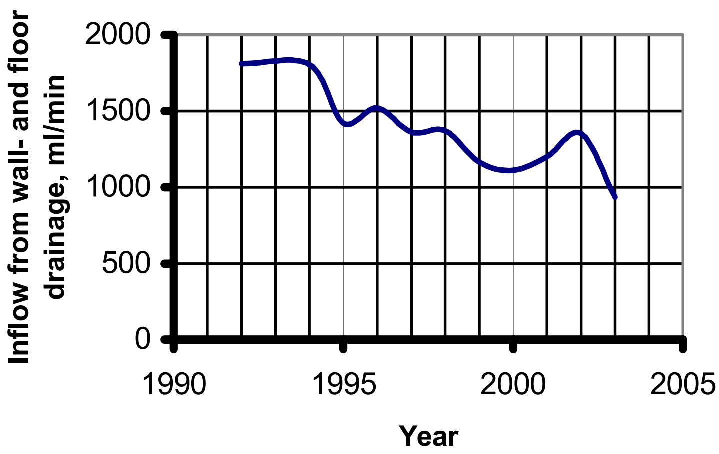


Figure 6-2. *Variation in discharge from the silo drain system in the period 1992 to 2002.*

7 Aspects on future program for checking the performance of the clay-based engineered barriers

7.1 Stability of rock and silo

The possible impact of seismic events on the stability of the rock mass and its influence on the pressure on the silo walls should be considered in the evaluation of future measurements of stress and strain phenomena. Hence, it is recommended that information on such events is collected annually and whenever major quakes take place. For the same reason, Swedpower's ongoing recording of rock movements should continue.

7.2 Settlement of silo

Although there are very good reasons to believe that the settlement of the silo will proceed according to the predictions it should still be checked by measurements using the present technique. The frequency of measurements is preferably once per year except if seismic events in the area by magnitudes exceeding 3 take place. Additional measurements should then be made.

7.3 Wall fill pressure

Measurement of the wall pressure is essential for making sure that it acts sufficiently uniformly on the silo wall and for providing information on the rate of hydration, which is in turn an indication of how well the rock drainage works. Also, it provides information on possible rock movements, like slip of major rock wedges induced by creep or seismic events. The frequency of measurements is preferably once per year except if seismic events in the area by magnitudes exceeding 3 take place. Additional measurements should then be made.

For getting fresh information on the hydration rate as a basis for updated modelling of wetting and expansion/compression processes, sampling of fill material of the same type as earlier should be made in 2004. It should be extended to comprise sampling both in "wet" and "dry" rock areas and where maximum upheaval and subsidence of the pavement have been recorded.

7.4 Wall fill movements

Measurement of wall fill movements as manifested by the change in level of the cement pavement is essential since it may be significant when hydration proceeds. The frequency of measurements is preferably once per year.

7.5 Drainage

Measurement of the discharge from the tubes connected to the rock drain system is very important since it gives early information on possibly insufficient capacity of the system to drain off water from the rock. The frequency of measurements is preferably once per month.

8 Major conclusions

Host rock performance

The measurements of rock movements show that creep is very limited but that instantaneous block movements may have taken place at some time after the excavation. Such movements may occur in future as a result of creep-induced stress accumulation, or generated by earthquakes. One cannot draw safe conclusions concerning the magnitude and direction of the vertical component of the measured strain of the cavern wall, which largely determines the accuracy of silo settlement recordings made by use of reference bolts anchored in the walls. However, it is reasonable to believe that the net movement is directed downwards and amounts at a couple of hundred micrometers to date.

Silo movement

- Recordings show that the initial settlement of the silo was somewhat faster than predicted while there is rather good agreement in recent years. The subsidence of the silo top, which is important for the future design of the top fill, is primarily dependent on the compression of the bottom bed.

- The composition and density of the wall fill will not give swelling pressures that can displace the silo.

- Non-uniform (uni-lateral) hydration can not lead to displacement of the silo.

- Slip of rock wedges is not expected because of the lateral support provided by the wall fill but slight movement of a major wedge can occur before resistance is fully mobilized. It can cause minor densification of the fill and a slight increase in pressure on the silo wall.

- The effective pressure in the wall fill can be slightly higher than the swelling pressure, except in the uppermost part, and may therefore lead to some slight compression of the fill, which has to be compensated before closing the repository.

- Wetting of the wall fill will give a high degree of water saturation early within a few decimeters distance from the shotcreted rock wall. Complete water saturation of the entire clay mass may take several hundred years.

- The wetting process in the wall fill should be due to diffusive migration of liquid water as shown by pilot tests in the field and laboratory but wetting by water vapour and local piping are possible additional hydration mechanisms.

Soil pressure

The following major conclusions can be drawn:

- The silo will not fail by pressure induced by slip of large rock wedges.

- Hydration of the wall fill will be very slow and complete water saturation may take several hundred years.

- The lateral pressure exerted by the wall fill on the silo and rock agrees fairly well with the predictions and are well below the critical pressure on the silo.

- No piezometric pressures have been recorded, which shows that the drainage system works satisfactorily.

- The movement of the wall fill as evaluated from levelling the pavement on top of it is small. It did not exceed about 42 mm and was downwards in some areas and upwards in others.

- The recorded discharge from the tubes in the tunnel below the silo base verifies that the drain system works satisfactorily.

Recording systems

The most important processes that require recording systems are the settlement of the silo, the build-up of soil pressure on the silo and rock, and the drainage of the rock. The equipments used for the recording were selected as to provide reliability under a long period of time and they turn out to serve satisfactorily.

Future performance checking

The following issues are important and require measurements and evaluation:

- *Stability of rock and silo*. It is recommended that information on seismic events is collected annually and whenever major quakes take place. For the same reason, Swedpower's ongoing recording of rock movements should continue.

- *Settlement of silo.* The frequency of measurements is preferably once per year except if seismic events in the area by magnitudes exceeding 3 take place. Additional measurements should then be made.

- *Wall fill pressure*. The frequency of measurements is preferably once per year except if seismic events in the area by magnitudes exceeding 3 take place. Additional measurements should then be made.

- *Wall fill maturation.* Sampling for measurement of the water content of the wall fill should be made in 2004 for better prediction of the hydration rate.

- *Wall fill movements.* Measurement of wall fill movements as manifested by the change in level of the cement pavement should be made once per year.
- *Drainage.* Measurement once per month of the discharge from the tubes connected to the rock drain system gives early information on the performance of the system.

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Appendix

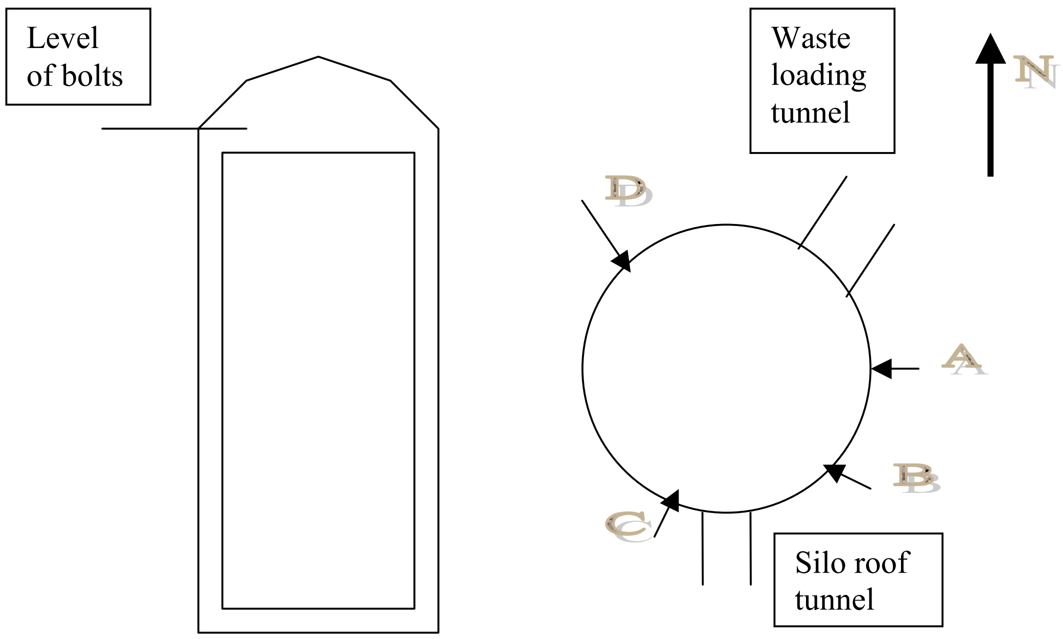

Figure A. *Location of bolts for recording of silo top subsidence (cf page 26).*

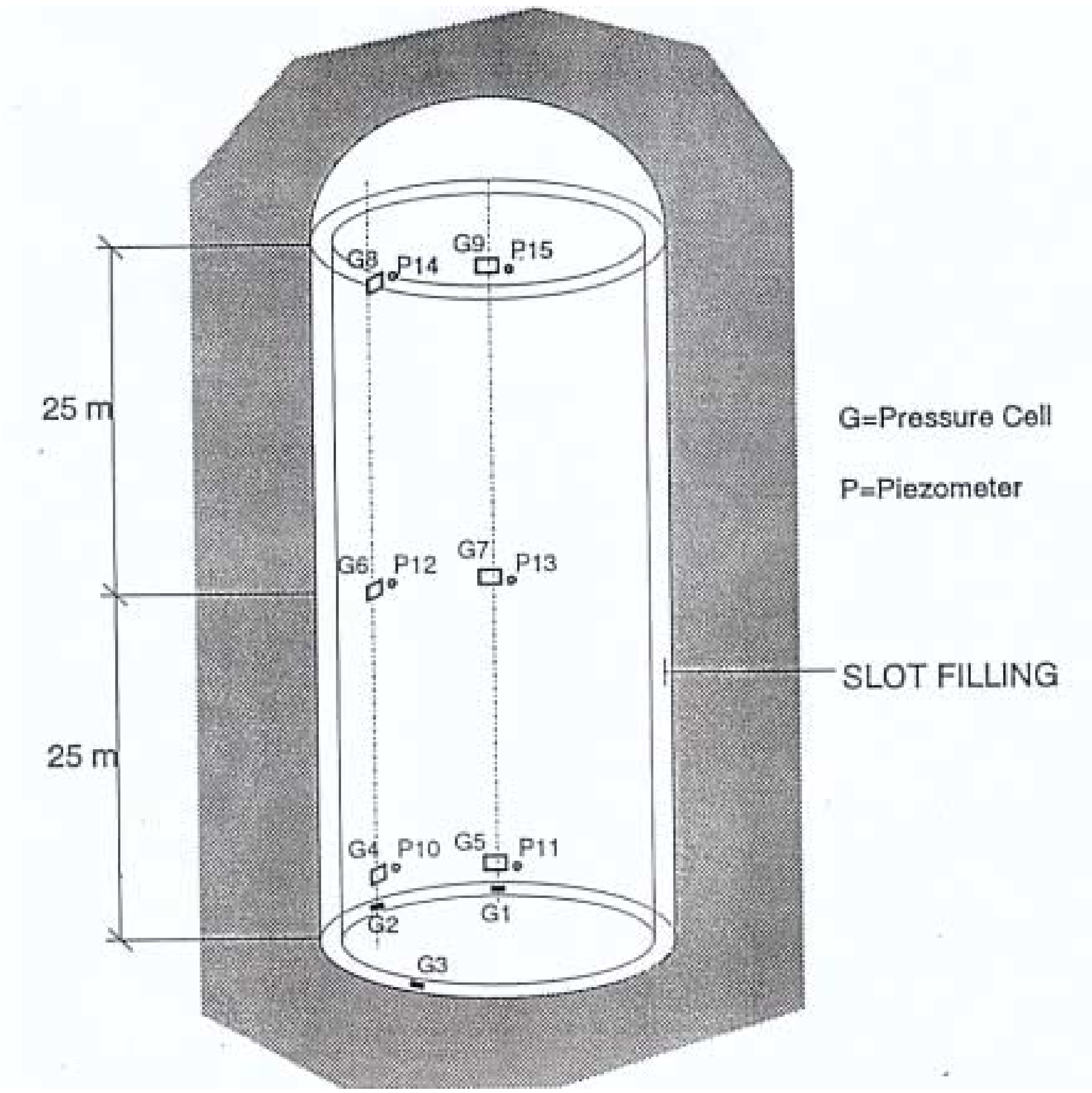

Figure B. *Overview of the location of pressure gauges (cf page 38).*