Journal of Earth Sciences and Geotechnical Engineering, vol. 3, no. 3, 2013, 147-162
ISSN: 1792-9040 (print), 1792-9660 (online)
Scienpress Ltd, 2013

Principles of Designing Hazardous Landfills: The Lithuanian LLW Case

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Abstract

A candidate concept for a Lithuanian repository for low-level radioactive waste (LLW) implies location above the groundwater level and placement of the waste in concrete vaults surrounded by smectitic clay of appreciable thickness as all-around embedment for retarding wetting and minimizing exposure of the waste packages to oxygen. For rational and cost-saving reasons the clay material is not processed, only stockpiled for desiccation to a suitable water content before placement and compaction. The clay is laterally confined by sandy fill and covered by erosion-resisting soil. The vaults rest on stable, elevated ground for minimizing subsidence and maintaining dry conditions. The time for water saturation of the clay is more than the required 300-500 years of isolation of the waste, which keeps the waste packages dry in this period. The paper describes the principles of design of the repository and its predicted performance of which creep strain of the clay is of particular importance.

Keywords: clay, creep, erosion, landfill, Lithuania, radioactive waste, repository, slope stability, smectite

1 Scope

Disposal of radioactive waste is made with the objective to effectively isolate it from the biosphere. For high-level waste deep geological storage is required while for intermedium and low-level waste without long-lived radionuclides disposal in a near-surface (NSR) repository is a possible alternative. The required time for isolation of such waste from the biosphere can be 300-500 years. The paper describes a design principle worked out in co-operation of the companies Drawrite AB, Lund, Westinghouse Atom Co, Stockholm, and the International Swedish Nuclear Fuel and Waste Handling Company AB, Stockholm, for Lithuanian authorities. It fulfils this criterion and was accepted by IAEA for construction in north-eastern Lithuania. The fundamental features of the repository are

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that the waste containers are placed in concrete vaults resting on stable ground and surrounded by low-permeable soil for minimizing infiltration of water (Figure 1). The paper describes the principles of site location and design of the repository, and its predicted performance.

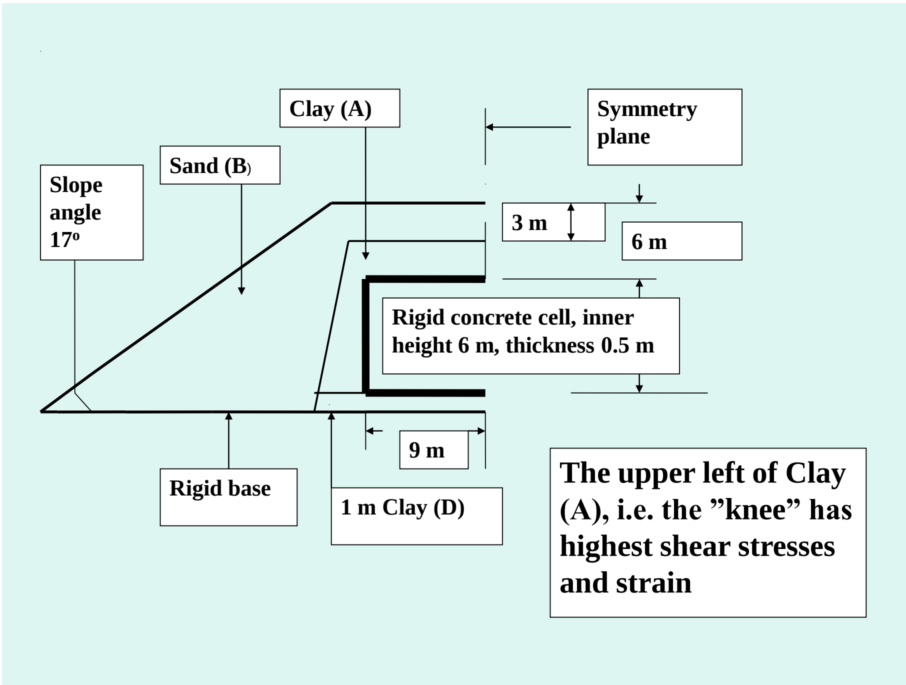


Figure 1: Schematic section of the proposed NSR reposity according to the proposed SKB- conceptual design. The clay denoted A plays a major role.

2 Waste Characterization and Handling

The waste is confined in disposal containers, mostly steel drums, that will be placed in concrete vaults under a movable steel shelter with no extra radiation shielding. With maximum surface dose rates on the disposal containers of 50 mSv/h the distance must be at least 130 - 160 m to "non-nuclear" construction workers. As the disposal area may need to be more than 500 m long it will be possible to perform the placement without extra shielding and construction of the last third can be done at the same time as disposal is going on in the first third.

On wetting, the steel containers and their waste content will produce hydrogen and methane gas, which must be released in a controlled way for avoiding damage to the top seal of the repository. Maintaining dry conditions of the interior of the cells can eliminate the need for gas vents while wet conditions in the vaults would require such facilities.

3 Design Criteria

The NSR repository shall be located so that the following criteria are fulfilled:

- It must not become flooded, which requires location on elevated terrain ("hill-type"), so that discharge of rain and thawing snow precipitated on and around the repository can take place without causing erosion or accumulation of surface water,
- It must not be located on unstable ground like sloping clay with a potential to undergo failure, or on limestone where karst phenomena can appear,
- It must not be significantly affected by earthquakes, hence requiring that location shall not be where there are major fracture zones in the bedrock,
- The construction cost should be as low as possible still implying that the repository serves acceptably. This requires access to suitable soil materials and transport facilities.

Three alternative sites were proposed by the Lithuanian investigators and one (Galilauké) was taken as primary candidate because of the suitable topography and stratigraphy. Here, the underground consists of dense, mainly sandy soil of Quaternary origin.

4 Conceptual Design

4.1 Major Principles

The aim was to design and construct the repository for effective isolation of the waste from water, requiring that the disposal vaults are so tight that water will not enter or percolate them in 300-500 years. High-quality concrete and effectively sealing clay layers where thence major components. Properly composed and cast concrete has a hydraulic conductivity of no more than E-10 m/s, which should also be the conductivity of clays A and D in Figure 1, requiring a certain minimum content of smectite minerals [1]. Sand B in the figure serves to give lateral support to clay A and for providing drainage of precipitated water. Construction of clay A can be made stepwise in layers parallel to placement and compaction of sand B. The vaults rest on a bottom liner D of effectively compacted clay.

Clay membranes, polyurethane films, and bitumen were initially considered as tight top liners but were deemed unsuitable because of the risk of physical damage and chemical degradation. Polyurethane films may not be chemically stable for more than a few decades and should only be used for temporary sealing. Bitumen may become brittle and fracture and should not be relied on as seals but can be accepted as temporarily water-tight barriers.

4.2 Geometry

Two rows with 25 cell groups in each constitute the storage rooms, the free inner dimensions being 5-6 m. The need for eliminating freezing and erosion of any of the clay components led to the decision to make the filling on top of the concrete vaults 3 m thick. The major seal, clay A, consists of 1 m smectitic clay and filters of sandy, silty clay at its upper and lower boundaries for eliminating particle migration from the clay. On top of the upper filter 1-2 m of gravel and boulders with upward increasing size is placed as erosion protection. The sand B shall have a slope angle that provides sufficient safety for avoiding slope failure and for creating a nice-looking landscape. An angle of 17o implies a safety factor of at least 2.

4.3 Longevity of Repository Components

Concrete vaults exposed to water will degrade by dissolution of the cement. Portland cement will be dissolved to an extent that will affect the bulk strength of the concrete significantly but low-pH cement may turn out to make it more longlived. pH of the concrete porewater will be higher than 10 and hence cause degradation of the clay, which is not chemically stable for pH lower than 6 and higher than 10. Use of available models for the chemical interaction of cement and clay for assessing the risk of clay degradation by the pH plume emanating from the cement in contacting concrete shows that chemical degradation of contacting clay can be minimized [2]. The overall conclusion from such calculations was that if the clay embedment will be hydrated so slowly that the wetting front has not reached the concrete vaults, these will stay mechanically and chemically intact and not harm the contacting clay.

The risk of liquefaction or triggering of slope failure by earthquakes is eliminated if the density of all soil materials is sufficiently high. For the densities of clays A and D that are aimed at, i.e. at 1850 kg/m3 at saturation, earthquakes of Richter magnitudes up to 6 will not cause damage [3]. For sand B, softening and slope failure would be generated if there were no heavy top layer of erosion-resisting coarse soil. An additional measure taken to avoid this risk is to construct a berm of coarse soil at the foot. The berm and erosion protection on top of sand B are not shown in Figure 1.

5 Clay Materials

5.1 Criteria

The major criteria for selection of clays A and D are the following:

Clay A shall have a very low hydraulic conductivity and be placeable under normal conditions with respect to temperature and precipitation. It must also stay coherent and stable and changes in homogeneity and porosity by temperature and moisture variations must not endanger its tightness. This is offered by smectitic soil that can take up and give off moisture and self-seal at wetting after periods of draught. It should exert a pressure on the concrete vaults and confining fills for establishing good contact with them. This pressure, which is effective and termed swelling pressure [1], must balance the effective overburden pressure, and the earth pressure exerted by Sand B, on Clay A. If these criteria are not fulfilled, Clay A will expand when saturated with water and contract on desiccation, which may result in displacements and cracking. Clay D is primarily for preventing oxygen and moist air to enter the vaults and can be less smectite-rich. However, the desire to use just one clay type and to effectively reduce migration of oxygen from below into the cells led to the decision to use clay of A type also for the foundation of the cells.

5.2 Selection of Clay

5.2.1 Criteria

Three clay materials were tested for use as isolation of the forthcoming repository and one of them, a red clay of Triassic age and emanating from northwest Lithuania, termed “C; Pit Saltiskes II”, was proposed as candidate material. It fulfills the following criteria:

- the raw material source must be sufficiently homogeneous. It is suggested that the content of clay-sized particles must not vary by more than 10 % from the average figure. Also, the content of the major (smectite) clay mineral must not vary by more than 10 % from the average figure,
- the raw material source must be sufficiently large. The amount identified with certainty by proper prospection should be at least twice the amount required by the decided design,
- the hydraulic conductivity of the clay-based top cover and lateral fills must not exceed E-10 m/s,
- the swelling pressure should not exceed 200 kPa for avoiding too strong expansion and upheaval of overlying soil,
- the clay material must be effectively compactable so that the dry density (ratio of solids and total volume including voids) yielding the required hydraulic conductivity and swelling pressure is reached.

The selected clay had a content of minus 2 μm particles of 30 %. The content of smectite was 20 % with other clay minerals (illite and chlorite) making up 10 % and rock-forming minerals the rest. It has an average liquid limit of 55 % and a plastic limit of 22 %. Testing of samples extracted from a stockpile comprised determination of the mineral content by XRD technique and Atterberg consistency limits, as well as of the hydraulic conductivity and swelling pressure as functions of density. Since the clay will be exposed to shear stresses and can undergo time-dependent strain, creep tests were made as well.

5.2.2 Physical properties

Performance

The artificially prepared clay components A and D will not perform as sedimentary clay and there is no groundwater level formed in them. They have no preconsolidation pressure but will react on loading and unloading much like strongly overconsolidated clay. The required coherence requires that no or very little desiccation must take place in dry seasons. Of particular importance is that they must not undergo large creep strain, which implies that shear stresses in the clays must be significantly lower than those causing practically important creep strain. The rheological properties are therefore very important and related questions are given much space in this paper.

Mineralogy

A typical XRD diagram of air-dry and traditionally treated material is shown in Figure 2. The evaluated clay mineral content was dominated by 1) Fe-rich chlorite, 2) IS-mixed layer minerals (illite/smectite), 3) well- to poorly ordered illite, 4) kaolinite (2 types), feldspar, quartz and traces of calcite. The percentage of illitic layers in the IS varied between 40 and 85 % indicating that the content of expanding components was relatively small, i.e. 15-25 %.

Physical properties

The soils were prepared by drying the material at 105oC with subsequent crushing to crumb size ranging between 0.01 to 2 mm. The crushed soil was placed in oedometers and compressed to different densities for each material. The intended density at water saturation was 1750 to 2110 kg/m3, representing low and heavy compaction, respectively. After complete hydration of the confined samples a hydraulic gradient of 20-60 was applied and the percolation rate measured using distilled water as percolate. Evaluation of the hydraulic conductivity from these tests, which also gave the swelling pressure, was made by applying Darcy's law after reaching steady state conditions. The results are given in Table 1.

For a density of 2110 kg/m3 at water saturation (dry density 1590 kg/m3) the clay exerts a swelling pressure on the overlying, capping material of 300 kPa. For Clay A this pressure would be balanced by 15 m thick overburden, which is not realistic. For a density at saturation of 1750 kg/m3 the swelling pressure is around 20 kPa requiring a thickness of the overburden of slightly more than 1 m, which is much less than required for preventing freezing of the clay layer. For a density at saturation of 2045 kg/m3 (dry density 1660 kg/m3) the overburden would have to be about 8 m thick for balancing the swelling pressure. Taking the density at saturation as 1850 kg/m3 (dry density 1350 kg/m3) one finds that pressure balance is obtained for an overburden of about 3 m. For this density the hydraulic conductivity is about E-10 m/s, which means that the two major criteria are fulfilled.

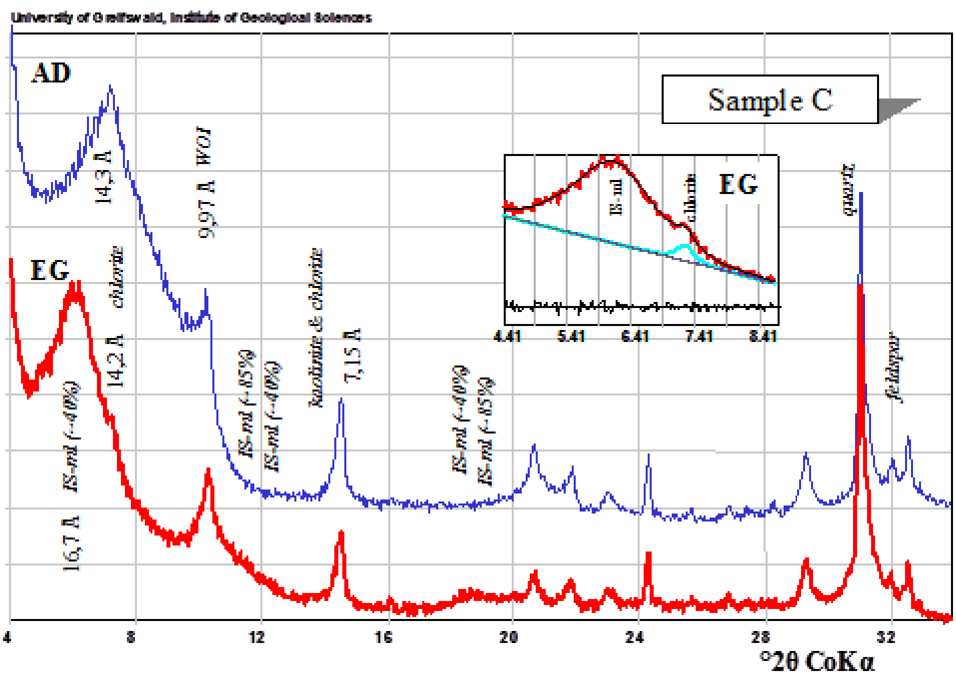


Figure 2: Legend: AD – air dried, EG – ethylene-glycol saturated; IS-ml (~ 85%) – illite-smectite mixed layer with 85 % illitic layers. (J. Kasbohm, Greifswald University).

Table 1: Evaluated hydraulic conductivity and swelling pressure after reaching steady state.

Dry density, kg/m^3	Density at water saturation, kg/m^3	Water content, weight percent	Hydraulic conductivity, m/s	Swelling pressure, kPa
1190	1750	39.2	1.2E-10	20
1660	2045	29.6	6.0E-11	170
1760	2110	21.4	1.4E-11	300

Stability and creep

The artificially prepared clay is not water-saturated when placed and will probably not become saturated in the period for which it has to provide isolation. It does not have a preconsolidation pressure and behaves as a visco/elasto/plastic medium. Like all smectitic clays it is thixotropic and can therefore gain strength by resting under constant volume conditions. Figure 3 illustrates the typical creep strain pattern for moderate deviator stresses that allow for microstructural recovery. If there is successive retardation of the creep rate according to the logarithmic time law it is expected that failure will not be caused even after a very long periods of time.

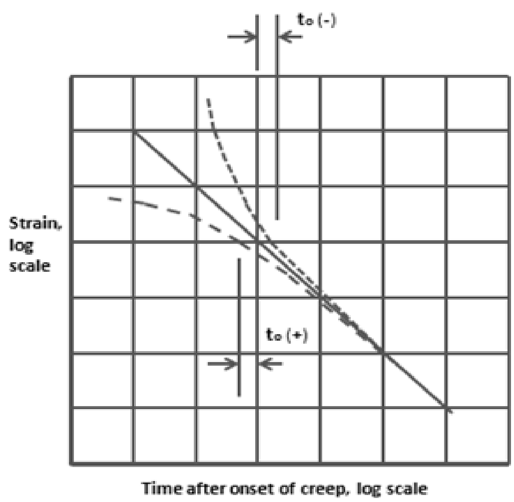


Figure 3: Generalization of creep curves of log time type.
Lower shear stresses give strong retardation of the creep that soon after onset dies off (first phase of “primary creep”; [1]). Higher shear stresses lead to “secondary creep” with a strain rate that is proportional to time, which is too high to allow for microstructural self-repair, hence leading to failure.

Determination of the shear strength and creep behaviour is commonly made by using the triaxial cell but the shear box is suitable for conducting long term creep tests although the principal stresses are not well known [1]. In the present study a shear box of the type shown in Figure 4 for symmetrically loaded samples was used. Excellent lubrication of the contact between the yoke and ring halves is required.

The very stiff “double” shear box was used for determining the time-dependent shear strain of samples of clay saturated with distilled water. The shearing took place under drained conditions, i.e. with the filters at the basal surfaces connected to burettes for measuring uptake or discharge of porewater. In all the tests the strain turned out to take place without measurable volume change indicating that dilatancy or compression did not take place.

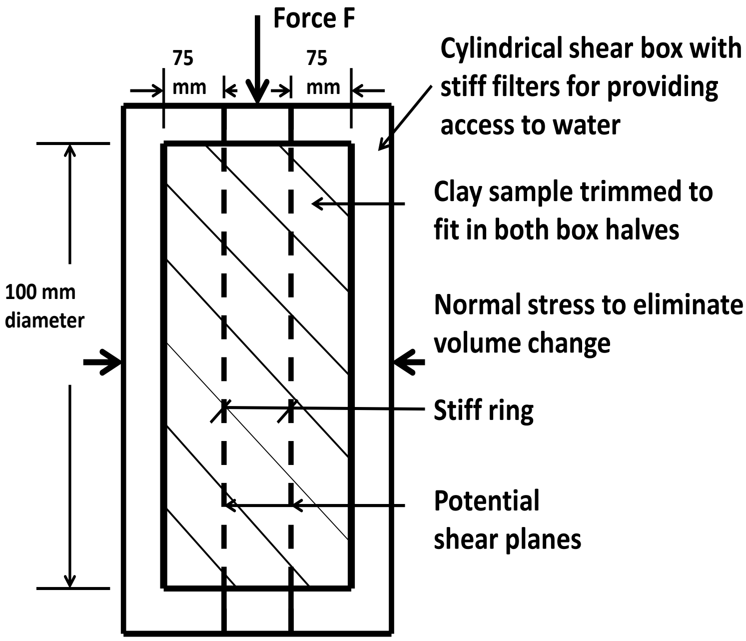


Figure 4: Ring ”double” shear box.

While the general shear stress conditions in the sheared sample at failure can be estimated as the load F divided by the total shear plane area, i.e 2pD2/4 where D is the diameter of the shear box, calculation of the shear stress distribution at lower stress levels requires numerical techniques. In the present study the ABAQUS code was used and the bulk strain taken as main variable. Based on preliminary estimates the shear modulus was taken as 1.5 MPa, Young's modulus as 4.44 MPa, Poisson’s ratio as 0.48, the yield shear stress 15 kPa, and the angle of friction and friction between clay and ring as 0o. The calculation was made by Computational Mechanics Center, Southampton, UK.

The concept of yield was incorporated in the model by using a Drucker-Prager condition in the material plastic property definition process. For 1 mm ring displacement (1 %) the theoretical maximum contact force between the clay and ring was 50 N, while for 3 and 5 % strain it was 52-53 N. A representative average shear stress can be derived from the expression:

$$F/2=[A_p\, \tau_{crit} + (A_{tot} - A_p)\tau_{av})] \tag{1}$$

where F is the ring load, Ap the area of the plasticized part of the whole cross section Atot, tcrit the shear stress in the plasticized clay and tav the average shear stress in the non-plasticized clay. Figure 5 shows the calculated shear stress distribution in a cross section trough the center of the sample. The maximum shear stress in the plasticized zones is about 1.5E-2 MPa i.e. 15 kPa, while the shear stress in the non-plasticized clay ranges between about 2-7 kPa.

Actual creep testing of Clay A samples saturated with distilled water and with a density at water saturation of 1885 kg/m3 (dry density of 1510 kg/m3) have been performed. The load on the ring was applied in steps and the creep recorded with an accuracy of 0.001 mm. Load increase was made when the creep strain rate had dropped to less than E-8 sec-1. The load steps corresponded to the average shear stresses 7 kPa, 11 kPa, and 14 kPa. The lowest stress gave almost no time-dependent strain while the shear stress 11 kPa caused creep strain according to Figure 6, obeying, in principle, the relationship in Equation 2:

$$\gamma=\alpha t - \beta t^2, \ (t<a/2\beta), \ \alpha \text{ and } \beta \text{ are constants} \tag{2}$$

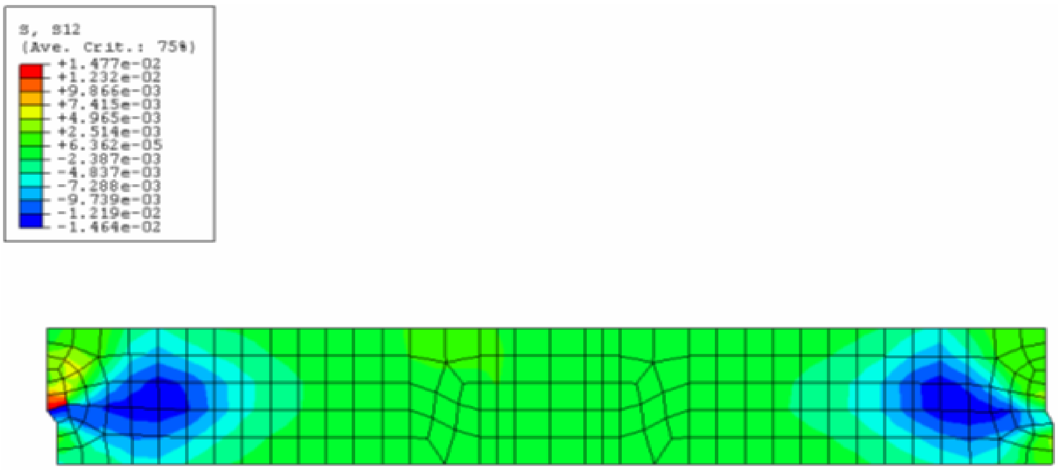


Figure 5: Example of stress distribution in the sheared sample for 1 % shear strain. The scale is in kN/m^2 (kPa).

The equation means that creep starts off linearly with time and then dies out, which is in agreement with the creep theory proposed by Feltham [4] and found to apply to a number of expandable and nonexapandable clays [1]. It is based on the concept that macroscopic strain is the accumulated amount of slip in clay elements subjected to a deviator stress and that slip on the microstructural scale occurs when the energy barriers represented by interparticle bonds are overcome. The energy barriers have different heights and the entire process is stochastic, meaning that slipping units will meet higher and lower barriers in the strain process and that the firstmentioned dominates for shear stresses lower than 11 kPa for the investigated clay.

For higher stresses the strain on the microstructural level yields some irreversible changes associated with local breakdown and reorganization of particle network. Still, there is repair by inflow of new low-energy barriers parallel to the strain retardation caused by the successively increased number of slip units being haulted by meeting higher energy barriers. This type of creep can go on forever without approaching failure [1]. It is valid for thermodynamically appropriately defined limits of the energy spectrum, the strain rate appertaining to logarithmic creep (Equation 3):

$$d\gamma/dt=BT\tau/(t+t_o) \tag{3}$$

where B=is a function of the shear stress t.

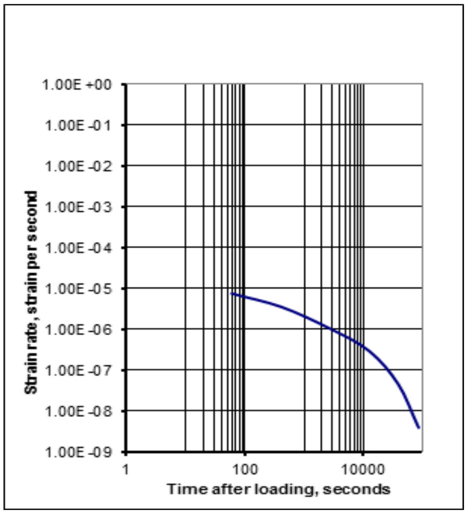


Figure 6: Creep strain of the clay for a load generating a shear stress of 11 kPa.

The physical implication of this expression is that the lower end of the energy spectrum mainly relates to breakage of weak bonds and establishment of new bonds where stress relaxation has taken place due to stress transfer from overloaded parts of the particle network to stronger parts, while the higher barriers are located in more rigid components of the structure. Figure 7 shows the behaviour for the shear stress 14 kPa, which implied initiation of microstructural breakdown and a performance that is in agreement with the strain rate described by Equation 3 up to some E6 seconds. This equation contains an integration coefficient to that is defined in Figure 4 and that implies a creep relation closely representing the commonly observed logarithmic type, i.e. the creep strain being proportional to log(t+to). The significance of to is understood by considering that in the course of applying a deviatoric stress, at the onset of the creep test, the deviator rises from zero to its nominal, final value.

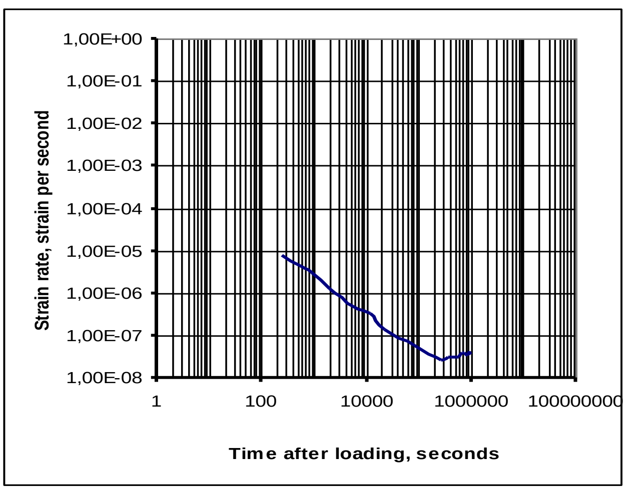


Figure 7: Creep behaviour at the average shear stress 14 kPa. The slight upward trend of the curve after 3E5 s suggests that the secondary creep phase has been reached and creep failure initiated.

The major conclusion from the determination of the physical properties was that physical stability of Clay A, i.e. the embedment of the concrete vaults with a density of 1850 kg/m3, will be achieved at complete water saturation if the shear stresses do not exceed about 11 kPa. Volume constancy requires that the overburden pressure from erosion-protective layers and adjacent frictional fill is at least 50 kPa and that the lateral pressure, representing Rankine earth pressure, is on the same order. Under the unsaturated conditions that prevail during most of the 300-500 years of required waste isolation, the shear strength is considerably higher since the angle of internal friction of air-dry material has been found to be 28o and the cohesion 80 kPa according to conventional cf analyses [5].

6 Construction

6.1 Principle

The construction site is prepared by removing vegetation and shallow organic soil down to firm, undisturbed soil that is compacted be 10-20 runs of heavy padfoot vibratory rollers or other dynamic tools. Drainage pipes of ceramic material are then installed around the construction area and connected to ditches filled with self-draining frictional soil. The base of the vaults and lateral supports consists of effectively compacted sand and gravel for eliminating capillary water rise. Its uppermost part consists of fine-grained silty sand on which the bottom liner D is built. On top of it the bottom slab of the concrete cells is cast. Clay A and sand B are placed layerwise, forming a saw-tooth contact. B is composed of sandy gravel with Fuller-type granulometry [1].

6.2 Clay Preparation

Experience from a number of practical cases of constructing landfills with clay-based top- and bottom liners shows that processing of the clay by mixing the raw material with sodium carbonate for bringing it in Na form or with other soil material for obtaining a suitable grain size distribution, with subsequent drying, grinding and sieving, is very expensive and does not always lead to a homogeneous condition. A major issue of the project was therefore to minimize processing and to find out if a repository design could be found that makes use of clay, directly taken from stockpiled clay that had been excavated in an open pit, spread out and compacted to the required minimum dry density 1350 kg/m3. The clay material brought to a test site for full-scale compaction had a lump size varying from centimetres to less than a millimetre and its compactability and achievable density were determined by use of a 420 kg vibrating plate and a 7 t vibrating, smooth-cylinder roller, respectively.

6.3 Constructability

The layers will have to be rather thin in practice and the tests were therefore made with 11 cm thick layers for the vibrating plate and 17 cm for the roller. 10 runs of each were used. The results are summarized in Table 2 showing that the compaction by both the vibratory plate and the roller is sufficient for reaching the required dry density of 1350 kg/m3 of Clays A and D. The finally selected techniques and equipments will have to be decided on the construction site with special respect to the available space for compacting Clay A.

7 Evolution and Performance

7.1 General

The rate of hydration of the initially air-dry upper clay liner and the vault-surrounding clay is assumed to take part by diffusion and the high degree of saturation that is a prerequisite for percolation would take at least 300 years assuming 1 m clay thickness and continuous contact with water in overlying coarser soil. This low rate is because the diffusion coefficient is of the order of E-10 m2/s [1]. Considering the very low hydraulic conductivity of concrete practically no water would hence enter the vaults in this period of time and no radionuclides would hence escape from them.

Table 2: Compacted clay layers for plate-loading tests. ρ_n represents the density of the compacted unsaturated clay in kg/m^3 [5]. ρ_d is the dry density in kg/m^3.

Layer	Compaction technique	Thickness, cm	Density, kg/m^3, ρ_n/ρ_d	Water content, %
1	Vibrating plate 420 kg	11	2001/1677	19
2	Vibrating roller, 7 t	17	1938/1611**	20
3	Vibrating roller, 7 t	17	1857/1583*	17
Base	Concrete floor	>10	2400	-
* First, lower layer. ** Second, upper layer				

Still, a more conservative case was assumed in the assessment of the performance, namely that the clay is initially fully water saturated, which would imply percolation of the vaults from start. Using the commercially available code Help 3.80 D for calculating the rate of inflow into the vaults it was found that the annual inflow of water would be 3 liters per square meter horizontal area of the vault roofs, assuming that the concrete has fractured and attained a hydraulic conductivity of E-4 m/s. In practice, intact concrete has a conductivity of less than E-13 m/s meaning that water passing through the top cover will not flow into and through the vault roof but be discharged laterally through the filter below the clay layer. The piezometric pressure in this layer may occasionally correspond to a groundwater level that coincides with the surface of the top cover, giving an inflow into the vaults by less than 3 milliliters per year and square meter. In 300 years it would amount to about 1 liter per square meter (36 liters per 125 m3 cell), which is much less than required for significant filling of the open voids in the space that contains nuclear waste.

7.2 Gas Production

A common problem for landfills is production of gas, like methane and hydrogen, caused by chemical reactions in the waste. It has to dissipate without disturbing the clay liner for avoiding accumulation of bubbles containing gas of successively increasing pressure that can ultimately break up the top seals and form channels that increases water inflow into the space with waste. The dry conditions for the waste in the vaults according to the proposed repository concept means that the risk of gas production is negligible.

7.3 Stability

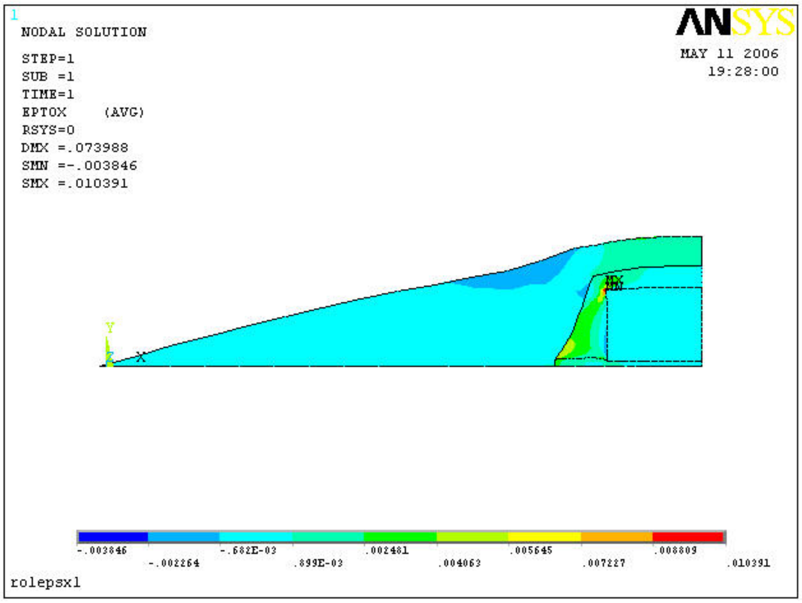
The concrete vaults can be designed so that they remain tight even if earth shocks of magnitude 6 occur repeatedly. The joints are critical features but they can maintain tightness by using seals of titanium or copper metal foils. The average shear stresses in Clay A with a dry density of 1350 kg/m3 are determined by the vertical load and the lateral earth pressure caused by the contacting Sand B. The latter pressure will range between active and passive Rankine pressure and presumably average as the earth pressure at rest. The effective confinement of Clay A eliminates any risk of bulk failure but internal displacements caused by shear stresses that will evolve in the saturation and maturation phases can be significant for shear stresses exceeding about 11 kPa.
The large clay mass is believed to stay coherent and to contain sufficiently much expanding minerals to self-seal after wetting/desiccation cycles with associated strain. The displacements in the clay by shear strain under the own weight of the soil components are not large enough to cause openings or weakening that threaten its tightness as concluded from stress/strain calculation using finite element technique. A typical example, indicating the most critical element of Clay A, is shown in Figure 8.

7.4 Steps Taken to Reach High Quality

Quality assurance has to be achieved by testing construction materials and checking of the accuracy of the various construction and waste-emplacement operations. A particularly important issue is to shield the site where placement and compaction of clay takes place from precipitation and cold. For this purpose the mobile shelter of steel intended for controlled placement of waste can be used (Figure 9).

8 Discussion and Conclusions

The design has undergone international and national reviews including detailed studies by IAEA and major design features have been identified for reconsideration and possible alteration. The reviewers specified a number of questions and suggested application of design principles of already decided or constructed low-level repositories. The most important issues brought up by the reviewers and responded by the designers were:



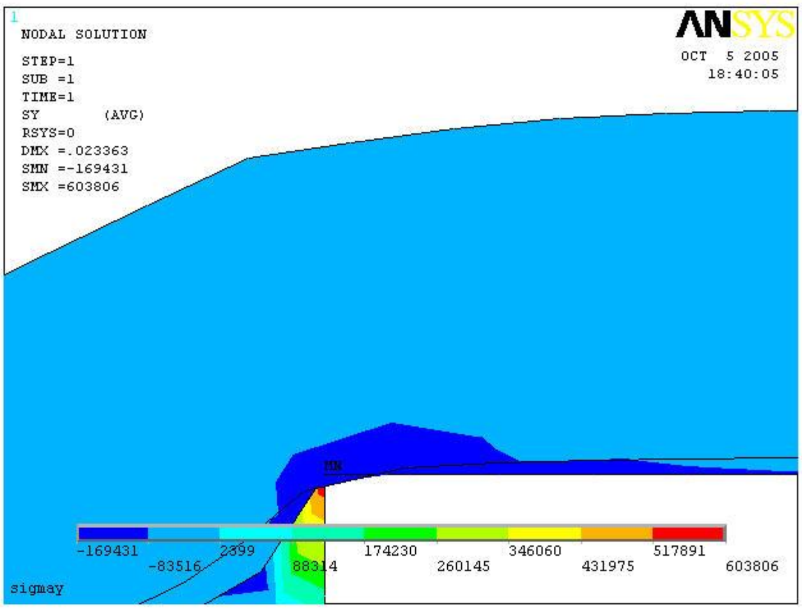


Figure 8: Upper: Horizontal strain ranges from compression up to about 0.3 % (dark blue) to tension by up to 0.8 % (red). The soil supporting Clay A embedding the vaults undergoes very little strain. Lower: Close-up showing the vertical stress (- is pressure, + tension). Dark and light blue (minus sign) represent pressure and greenish, yellowish and reddish (plus sign) represent tension. At the upper edge of the vault the clay is locally undergoing vertical tension but the rest is compressed and remains coherent.

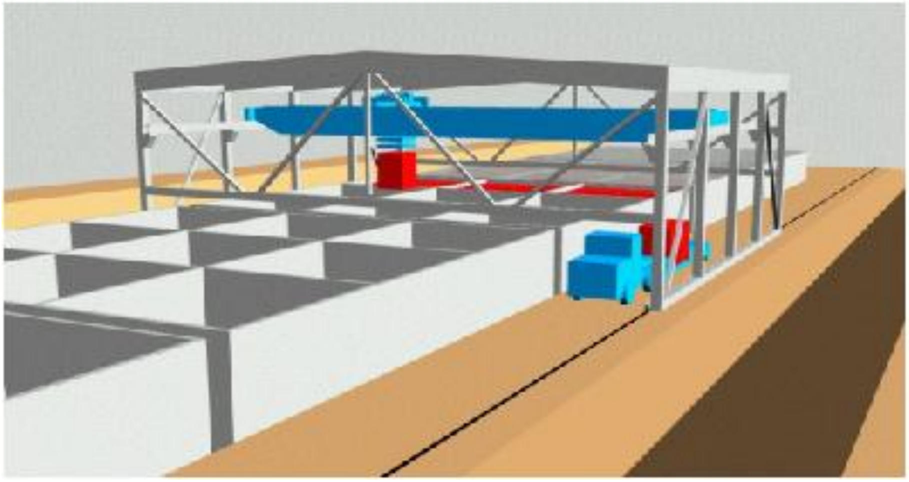


Figure 9: Vault system and movable shelter (After Westinghouse Electric).

- Base of vaults
- Closure of repository, inspection galleries, monitoring
- Gas release
- Water collection and drainage system
- Long-lasting water-proof cover

Conclusive remarks to "Base of vaults"

The design implies that insignificant amounts of water enters the vault systems in 300-500 years and that bathtub conditions can not arise. A high-permeable base would mean that oxygen can enter the vaults and cause corrosion of iron components like drums and concrete reinforcement and thereby have a negative effect on the isolation capacity of the system.

Conclusive remarks to "Closure of repository"

Effective top isolation of the vaults cannot be achieved by bentonite membranes since they will be too thin to withstand movements in the top cover and because they are not sufficiently erosion-resistant. Bitumen and polyurethane film become brittle and fracture and should not be relied on as long-lasting seals. In contrast, smectite clay of appreciable thickness applied as all-around embedment, provides effective protection of the concrete and joints. However, plastic films can be used as temporary seals in the construction period.

Conclusive remarks to "Gas release"

Controlled gas release by using vents is difficult in a longer time perspective. The proposed design implies that critically high gas pressures will not be caused because of the dry conditions in the vaults.

Conclusive remarks to "Water collection system, drainage and galleries"

The proposed design implies that practically no inflow of water into the vaults will take place. Inspection galleries for checking the tightness can jeopardize the sealing function by serving as inflow paths of water and oxygen to the interior of the repository.

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