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Design of Geomembrane Liner for the Proton Decay Experiment

Design considerations and theoretical analyses of a water reservoir for the Proton Decay Experiment are presented. The design embodies the site location/conditions, operational considerations, and requirements imposed by the physics experiment itself. Compatibility of the geomembrane with the ultra-pure water used for the experiment led to the selection of HDPE. The reservoir location (600 m underground in a salt mine) imposed stringent conditions on the maximum leakage allowed due to the risk of salt solution. Hence, a double HDPE lining system with a monitored double drainage system was designed, installed, and put into operation. The double liner concept and the associated drainage system are discussed in detail. Theoretical analyses were performed to anticipate the mechanical behavior of the liner. Laboratory tests, which simulated the expected stresses on the liner, complemented the results of the theoretical analyses. Consequently, the final reservoir design proposed measures to alleviate stresses.

INTRODUCTION

Matter may not be forever. The lifespan of protons may be 10^{32} years according to a theory for which the Nobel Prize in physics was granted in 1979. To verify this theory, a group of physicists at the Universities of California and Michigan have set up the Proton Decay Experiment, sponsored by the U.S. Department of Energy. The experiment consists of monitoring a $18\text{m} \times 24\text{m} \times 20\text{m}$ deep reservoir containing 8500 m^3 (2.2 million gallons) of water, i.e., approximately 10^{33} protons. If the theory is correct, only a few of the protons in the reservoir would spontaneously disintegrate, or "decay", each year, giving off ultraviolet light detected by 2048 photomultiplier tubes immersed in the water. Transparency is essential and the water is permanently recycled and purified by a sophisticated facility adjacent to the reservoir. Detection of proton decay should not be disturbed by natural radioactivity and cosmic radiation. Hence, the reservoir has been excavated in salt, where natural radioactivity is low, and is located 600 m (2000 ft) underground where cosmic radiation is minimal.

Concrete and steel reservoirs were considered first, but were eliminated because of the cost associated with difficult access to the reservoir. Consequently, a geomembrane liner was considered. The design of the geomembrane lining system for the Proton Decay Experiment reservoir was an exceptional challenge due to the stringent requirements discussed in Section 1 of the paper. Sections 2 and 3 discuss the two most delicate aspects of the design: the leakage collection/detection system and the mechanical behavior of the liner.

1. DESIGN APPROACH**1.1 Operational Requirements**

The following requirements were dictated by the physics experiment:

- **Water Purity.** The degree of water purity required by the Proton Decay Experiment is several times more stringent than current distilled water standards.
- **Water Level.** During the operation of the experiment, the water level should remain nearly constant with a tolerance of 0.2 m (8 in.), during several years. Emptying the reservoir for repair or maintenance would not be critical to the experiment but would require a considerable amount of time (it takes five weeks to empty and five weeks to fill the reservoir through a pipe going to the ground surface).
- **Reservoir Depth.** The volume of the reservoir (8500 m^3 or 11000 cu.yd.) was dictated by the number of protons to be stored. The shape of the reservoir needed to be massive (i.e., almost equal in the three directions) to optimize measurements. Consequently, the depth of the reservoir had to be of order 20 m (70 ft). As a result, a high water pressure was exerted on the liner.

The above operational requirements resulted in the following requirements for the liner:

- **Geomembrane/Water Compatibility.** Release of chemicals (eg. plasticizers) by the geomembrane in contact with the extremely pure water should be minimal. Tests conducted at the University of California-Irvine led to the selection of a high density polyethylene geomembrane.
- **Maximum Leakage.** The rate of leakage through the liner should not affect the water level more than indicated above. It was determined that leakage rate should be under 40 liters (10 gallons) per minute to be compatible with the pumps evacuating leakage to the ground surface and the capacity of the purification facility to recycle leakage.

1.2 Site Conditions

In addition to the stringent requirements dictated by the physics experiment, equally stringent requirements were dictated by the following site conditions:

- **Risk of Salt Solution.** A large portion of the cavity walls consist of massive crystalline salt. Pure water in contact with salt could result in serious damage to the walls or even collapse of the cavity. Consequently, any water leaking through the liner should be collected and prevented from direct contact with the salt.

- **No Emergency Storage Capacity.** If it appears necessary to empty the reservoir, for repair or any other emergency, this can be done only through a pipe going to the ground surface, 600 m (2000 ft) above. It would have been extremely expensive to excavate another large cavity only for emergency purposes and it was not possible to spill water into the adjacent salt mine. Consequently, the conceptual design had to be such that even if a major leak occurred, the leaking water would remain contained in the reservoir area for the duration of the emptying operation.

- **Irregular Shape of the Cavity.** The cavity was being excavated when the liner design started. It was too late to change the shape of the cavity and the method of excavation. Consequently, the following geometrical conditions were imposed for the design of the liner: (i) cubical shape with rather angular corners (rounded corners generate more uniform stresses); (ii) wall irregularities caused by the excavation technique and large irregularities at the connection between the walls and the bulkhead which closes the access ramp used by excavation equipment; and (iii) chainlink mesh rockbolted to the walls to prevent spalling of salt or shale.

1.3 Design Considerations and Requirements

The above operational requirements and site conditions lead to three major design considerations:

- **Geomembrane Selection.** As mentioned above, geomembrane selection resulted primarily from pure water/geomembrane compatibility.

- **Leakage Collection.** Zero leakage is impossible to guarantee with any type of liner. Therefore, to meet the above mentioned stringent requirements regarding leakage, the following was done: (1) through liner quality control, ensure that leakage from the reservoir is under 40 liters (10 gallons) per minute; and (2) design a leakage collection system which: (i) prevents water/salt contact, (ii) prevents external "dirty" water from coming into the reservoir, and (iii) is able to contain leaking water for thirty days if leakage rate exceeds pumping capacity. Leakage collection design is discussed in Section 2.

- **Liner Mechanical Behavior.** The liner is subjected to high stresses as a result of wall irregularities and high water pressure. A design was conducted to evaluate stresses, propose measures to alleviate excessive stresses, and verify that the selected geomembrane would withstand remaining stresses. Mechanical behavior of the lining system is discussed in Section 3.

2. LEAKAGE COLLECTION

2.1 Double Liner Concept

A lining system should always be designed assuming that there will be a leak (1). A single liner was not deemed adequate because it would have been unable to prevent leaking water from being in contact with salt.

Description of Double Liner. A double liner with two drainage systems was selected (Fig. 1). The internal drainage system collects water leaking from the reservoir through the inner liner. A pump, located in the "clean sump", sends this water to the purification facility. If this pump is operated on a regular basis, water does not accumulate in the clean sump or the internal drainage system. As a result, the outer liner is subjected to negligible water pressure. Leakage through the outer liner should therefore be negligible, unless there are large holes at the bottom of the outer liner where water leaking through the inner liner flows.

The external drainage system collects: water that might have leaked through the outer liner; ground water; and water that could leak from the supply pipe or the adjacent purification facility. The water collected by the external drainage system is too dirty to be recycled. A pump located in the "dirty sump" sends it to the ground surface. Being promptly evacuated, the external dirty water is thus prevented from entering the internal drainage system through holes in the outer liner.

Functions of the Double Liner. The double lining/double drainage system was selected because it nearly meets all design requirements mentioned in Section 1.3: viz. (i) it prevents almost completely clean water from being in contact with salt; and (ii) it prevents dirty water from reaching the internal drainage system and the reservoir.

In addition, the vertical shafts surmounting the two sumps would hold water in case of contingency. If leakage through the inner liner exceeds the recycling capacity, the water level would rise in the "clean shaft" to the same level as in the reservoir, thereby balancing pressure and stopping leakage. However, leakage could then occur through the outer liner because it would become subjected to water pressure. This leakage would be collected by the external drainage system and conveyed to the dirty sump. If this leakage exceeds the capacity of the "dirty pump", the level of water would rise in the "dirty shaft", stopping all leakage but putting a large amount of water in contact with the salt. In that case, the reservoir should be emptied and water in contact with the salt should be evacuated. In this situation, the dirty water can be pumped back into the reservoir during the emptying operation, thereby preventing or minimizing damage to the salt cavity.

Therefore, it was recognized that a double liner did not provide a guarantee that the reservoir will never be emptied due to leakage. However, as mentioned in Section 1.1, interrupting the experiment is not critical. The double lining system also provided for leakage detection. This has proved to be essential (2, 3).

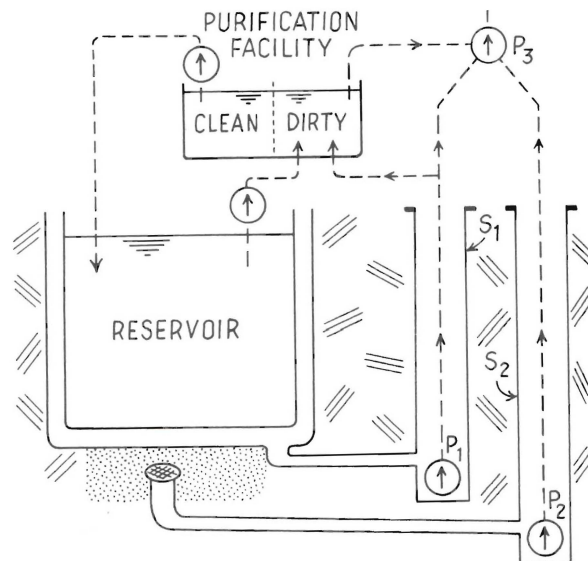


Fig. 1. A double lining system. Legend: P1, pump in "clean sump"; P2, pump in "dirty sump"; P3, pump rejecting dirty water to ground surface; S1, "clean shaft"; S2, "dirty shaft".

2.2 Leakage Evaluation

The following values of leakage through the inner liner were determined using equations given in (4).

- 0.06 liters/minute due to geomembrane permeability (assuming a coefficient of permeability of $k=10^{-13}$ m/s which is probably conservative for HDPE). The thickness of the geomembrane is 2.5 mm (100 mils).
- 15 liters/minute if there is one pinhole (diameter 0.1 mm) per m^2 , i.e., a total of 2100 pinholes.
- 50 liters/minute for one 10 mm diameter hole and 5 liters/minute for one 3 mm diameter hole, located in the lower half of the reservoir. These two values were calculated assuming a free draining medium outside the inner liner. They should be reduced, considering that such flow rates would locally saturate the internal drainage system.

Therefore, it was concluded that quality control of the installation, particularly the seams, would be important to ensure that the maximum allowable leakage rate of 40 liters/minute is not exceeded.

2.3 Selection of Drainage Material

To design the internal drainage system, it was conservatively assumed that leakage through each of the four wall liners would be 25% of total allowable leakage, i.e., 10 liters/minute ($1.7 \times 10^{-4} m^3/s$) through each wall liner (in reality a fraction of the leakage could have been expected through the floor liner). Therefore, the required hydraulic transmissivity of the drainage system on a 18 m wide wall is :

$$1.7 \times 10^{-4} \div 18 = 1 \times 10^{-5} m^2/s.$$

On the floor, where the slope was approximately 4%, the required hydraulic transmissivity at the base of the wall opposite to the sump is:

$$1 \times 10^{-5} \div 0.04 = 2.5 \times 10^{-4} m^2/s;$$

and the required hydraulic transmissivity near the sump, where the flow concentrates, was found to be $4 \times 10^{-3} m^2/s$.

Gravel was considered as a draining material for the internal drainage system on the floor, but gravel was deemed too radioactive and too cumbersome to transport to the underground reservoir. Synthetic drainage layers such as geotextiles, plastic mats and plastic nets were considered for the floor and the walls. Geotextiles did not have enough hydraulic transmissivity and mats were too compressible, losing a large fraction of their initially high hydraulic transmissivity under the design compressive stress (200 kPa, i.e., 20 m of water). The high density polyethylene net tested exhibited little compressibility and had a hydraulic transmissivity of $4 \times 10^{-4} m^2/s$. Tests to evaluate hydraulic transmissivity under compressive stress had been conducted at the University of Michigan, using an apparatus similar to the one described in (5).

Comparing required and measured hydraulic transmissivities, it was decided to use the following amount of plastic net: strips 0.15 m (6 in) wide every 1.5 m (5 ft) on the walls; a number of layers on the floor progressively increasing from 2 at the toe of the wall opposite to the sump to at least 10 in the sump area.

Nets, protected from salt and shale dust by a geotextile filter, were also used under the outer liner for the external drainage system. Also, reclaimed

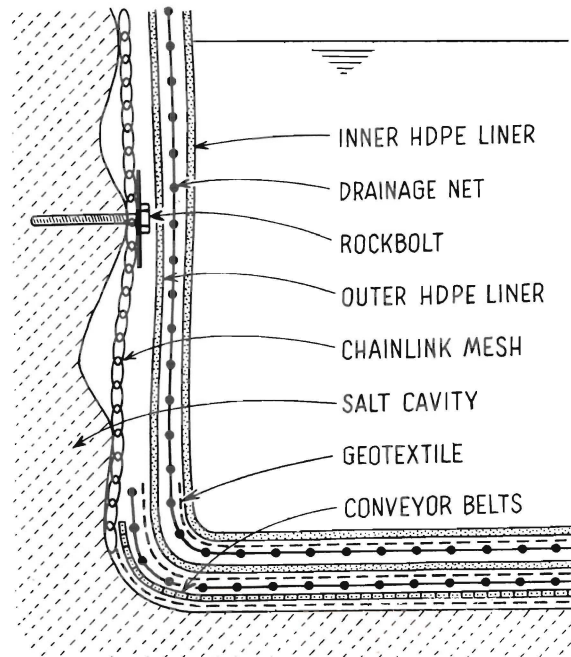


Fig. 2. Cross section showing layers forming the double lining system and typical chainlink mesh rockbolted to the wall (Not to scale).

conveyor belts were placed on the floor of the cavity under the outer liner to ensure a smooth support for the liner which is subjected to high water pressure. A summary of the materials used to form the lining system is shown schematically in Fig. 2.

Net strips used on the walls between the two liners as part of the internal drainage system were spot welded to the outer liner. Also, nets placed on the floor (external and internal drainage system) were spot welded together to prevent movement during liner installation. The drainage system was kept open at top for free escape of air entrapped between or under the liners.

2.4 Outlet Design

The "clean" and "dirty" outlets are part of the same concrete structure presented in Fig. 3. This structure is separated from the concrete structure which includes the two sumps by a 50 mm polystyrene foam-PVC waterstop joint. This joint allows for flexibility between the two structures. The outlet concrete structure is under the reservoir while the sump structure is under the bulkhead closing the access ramp used by excavation equipment. For the sump structure, the rock and the lowest bulkhead timber are solidly connected by the concrete. The outlet concrete structure is separated from the rock by 100 mm foam joints on 3 sides to allow the structure to withstand a significant reduction in the size of the salt cavity due to salt creep. The foam loses approximately 70% of its thickness under a compressive stress of 1 MPa. Stresses of that magnitude would be expected in the case of salt creep. Hence, the foam joints could absorb 14 cm (i.e., $2 \times 10 \times 70\%$) in the direction of the length of the outlet structure and 10 cm ($15 \times 70\%$) in the direction of the width, without damage to the structure. According to salt mine specialists, these displacements are smaller than salt displacements likely to occur over several years.

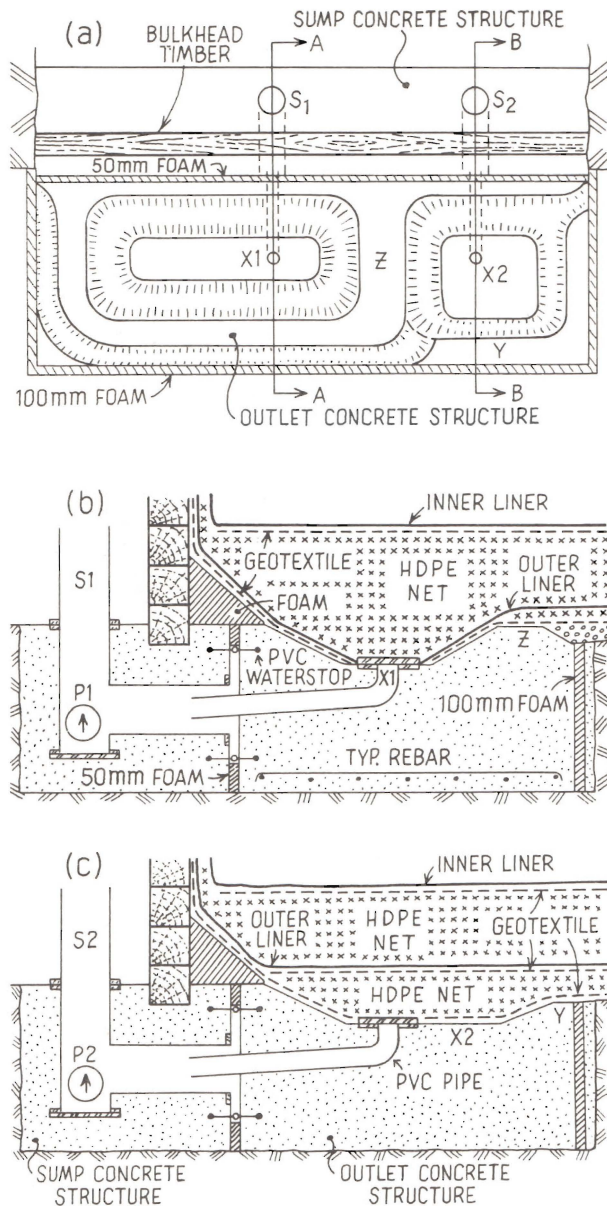


Fig. 3. (a) Plan view of the concrete structures embodying the clean and dirty outlets and sumps. S1 and S2 are the clean and dirty sump shafts which rise 20 m to the top of the reservoir. The 50 mm foam joint separates the two structures. The 100 mm foam joint on 3 sides of the outlet structure separate it from the rock. X1, X2, Y, and Z indicate various elevations of the outlet structure shown in section views A-A and B-B. (b) Section view A-A: internal drainage outlet, sump, and clean pump, P1. (c) Section view B-B: external drainage outlet, sump, and clean pump, P2. Indicated on the figures are the relative arrangements of the 2 HDPE geomembranes, HDPE drainage nets, geotextiles, bulkhead timbers, and foam-waterstop joints.

3. LINER MECHANICAL BEHAVIOR

3.1 Presentation of the Problem

Prior to first filling the reservoir, the double liner (including the intermediate plastic net spot-welded to the outer liner) is hung at the top and is subjected only to gravity forces. At this stage, the liner is in contact with the irregular walls of the cavity in a few points. Elsewhere, the liner is at a various distances from the wall.

The study described below was conducted to predict the behavior of the liner when subjected to water pressure and to recommend measures to alleviate stresses likely to be detrimental to the liner. Approximately 30 days are necessary to fill or empty the reservoir. The progressive application of water pressure on the liner during filling raised the following questions: (i) Would the water pressure be large enough to force the liner to follow the irregular shape of the walls? (ii) During the movement of the liner towards the walls, would the length of the liner increase (i.e., elongation of the liner) and/or would the top of the liner move downwards (provided it is not rigidly clamped)? and (iii) Would the liner withstand the stresses associated with the movements mentioned above? These questions are addressed in Section 3.2.

In all places where the liner would come in contact with the walls as a result of water pressure, another question arose: Would the liner burst through openings of the chain link mesh covering the walls? This question is addressed in Section 3.3.

Stresses likely to be caused by other mechanisms, such as gravity when the reservoir is empty, temperature difference between water and walls, and differential settlement between bottom and walls, were evaluated. These stresses appeared to be negligible as compared to the stresses generated by water pressures discussed in the next two sections.

In the analyses presented in the next two sections, the tensile behavior of the liner is the important characteristic to consider. The 2.5 mm (100 mils) thick geomembrane used for both liners had a tensile behavior typical for HDPE. The yield characteristics were approximately: 10% elongation, 20 MPa for the stress and, consequently, 50 kN/m for the force per unit width. The elongation at failure is of the order of 800%.

3.2 Liner Displacement Caused by Reservoir Filling

Survey of the Walls. To evaluate what would be the distance between the liner and the cavity walls, 52 vertical profiles of the walls were surveyed prior to liner installation. The 12 profiles surveyed on the East wall are shown in Fig. 4a. Also, the shape of the corners between walls and between wall-floor intersections were recorded. Typically, the radius of curvature at the corners was 0.6 to 0.9 m (2 to 3 ft). At the wall floor intersection, typical curvatures were 0.3 to 0.7m.

In the analysis, the likely location of the liner at the end of installation was determined by assuming that the liner was in contact with a number of protruding zones on the walls and was bridging the relative depressions between the protruding zones (Fig. 4b). The depressions were identified and recorded with their length, width, depth (relative to the assumed geomembrane location), and altitude. Also, the corners of the cavity were considered as depressions. This assumption proved to be correct, since during the installation it was not possible to place the liner in close contact with the wall at all points along the vertical corners.

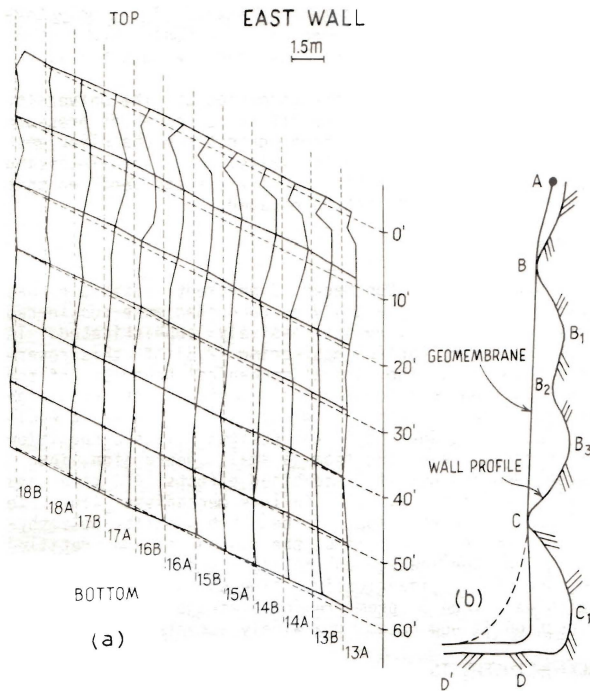


Fig. 4. (a) Surveyed shape of the East wall of the cavity after excavation. (b) Assumed shape of the geomembrane after installation.

Maximum Vertical Displacement. The first step of the analysis consisted of evaluating the maximum vertical displacement of the top of the liner during the first filling. Assuming that the liner would be pushed against the cavity wall by the water pressure, the predicted vertical displacement was the difference between wall length (RB1B2B3CC1D, in Fig. 4b) and liner length (BCD, in Fig. 4b). Values between 0.3 and 0.5 m (12 to 20 in.) were obtained. As a result, it was decided to hang the liner from adjustable chains instead of the traditional rigid clamping considered initially.

The predicted values of the vertical displacement mentioned above were believed to be upper boundaries because part of the movement of the liner towards the wall results from liner elongation rather than vertical displacement. However, vertical displacements observed after the first filling were slightly larger than predicted (max. observed value -0.6 m). Possibly, this could be due to the curved shape of the liner near the floor (CD' instead of CD in Fig. 4b). Also, depressions against which the liner exhibits elongation without vertical displacement are small depressions not shown in Fig. 4b. These small depressions were not considered in the evaluation of the wall profile length.

Pressure-Deflection Relationship. A preliminary study was conducted using the chart presented in (6). This chart had been established assuming that the liner would not move outside the depression area. This worst case situation gives the maximum liner elongation. The pressure-deflection analysis showed that the pressure exerted by 0 to 3 m (10 ft) of water would be sufficient to place the liner in contact with typical large depressions of the wall, but, in several cases, would cause yield of the liner. This analysis confirmed that it was important to attach the top of the liner in such a way that it could move as freely as possible.

To better model the behavior of the liner, an analysis was made, assuming friction between the liner and the wall (Fig. 5). The lengthy analysis cannot be reproduced here. However, it led to the chart presented in Fig. 5c. Using this chart for typical depressions of the wall showed that the water pressure necessary to push the liner in contact with the wall did not depend

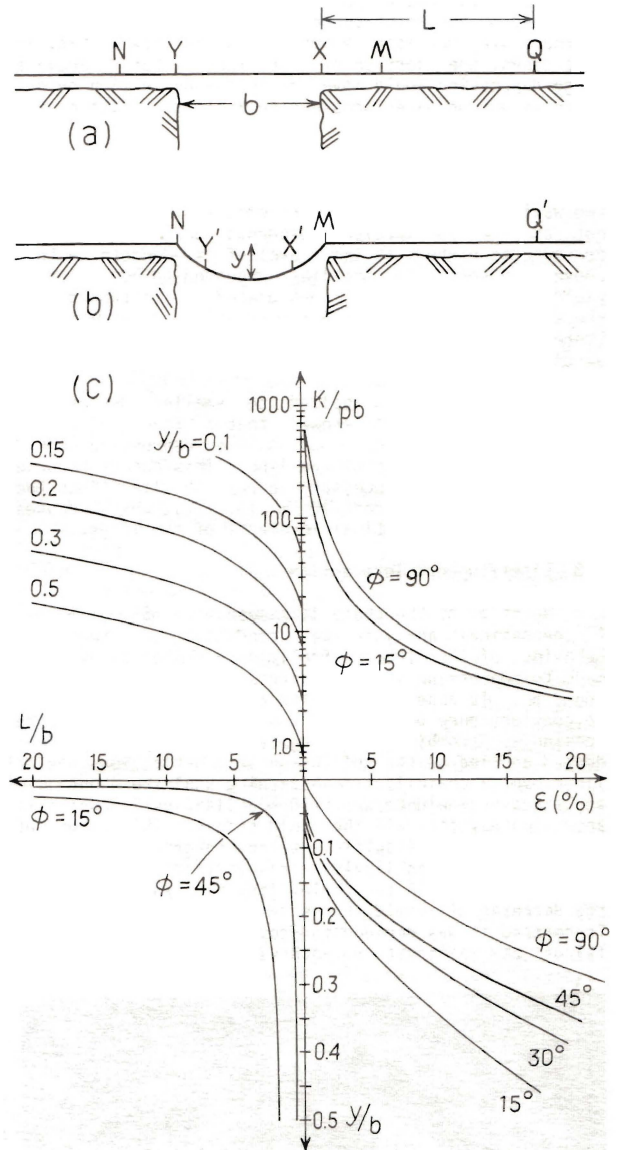


Fig. 5. Theoretical analysis of a geomembrane sliding with friction when pushed into a depression by water pressure: (a) Initial position of the geomembrane; (b) Final position of the geomembrane after sliding (beyond a certain length L, tensile stresses cannot overcome friction); (c) Chart. Knowing the geomembrane tensile stiffness (i.e., modulus x thickness), the water pressure p, and the width b of the depression, the chart gives the deflection y and the length L defined above (Note: for φ=90°, L=0).

significantly on the friction angle between the liner and the wall. Therefore, this analysis confirmed that a pressure of 0-3 m (10 ft.) of water would be sufficient to place the liner in contact with large depressions (typically 2 m wide) of the wall. Other findings of the analysis were as follows:

- A length of typically 2 to 4 m (6 to 13 ft) of liner could move towards a typical large depression (2-3 m), depending on the friction angle. Comparing this length to the above mentioned height of water shows that, in most cases, the portion of the liner located above a large depression would move toward the depression before there is enough water pressure to mobilize friction and block downward movements.
- The situation appeared more critical for the portions of liner located against vertical corners between two walls because horizontal movements of the liner are not as free as vertical movements and because water pressure is higher at the level of a depression than above. Stresses in the liner appeared to be close to yield stress if the liner was placed at distances from the wall of the order of 0.6 m (2 ft) or more. It was therefore judged extremely important to place the liner as close as possible to the wall in the corners.
- The same analysis applied to smaller depressions (typically 0.5 m wide) showed that a fraction of the movement of the liner towards the depression would result from elongation of the liner. This occurs because the height of water necessary to push the liner into the depression is of order 10 m (33 ft) which causes friction to block vertical movements of the liner.

3.3 Liner/Support Interaction

Openings of the chain link mesh were 55 mm x 55 mm. A theoretical analysis was conducted to study the behavior of the liner. Preliminary calculations were made to determine in what stress condition the liner would be. It appeared that the deflection was too large to consider pure bending and too small to consider pure tension. A combined bending-tension analysis was conducted and led to the conclusion that deflection of the liner over a chain link mesh opening would be about 3 mm and stresses would be about 10 MPa (1450 psi), which is approximately one-half the yield stress. This margin of safety was judged insufficient because of the potential additional stresses likely to result from elongation of the liner to reach the chain link mesh and because of the decrease of yield stress with time due to creep. Therefore, it was decided to cover the lower 3 m (10 ft) of the walls with polyethylene or other protective

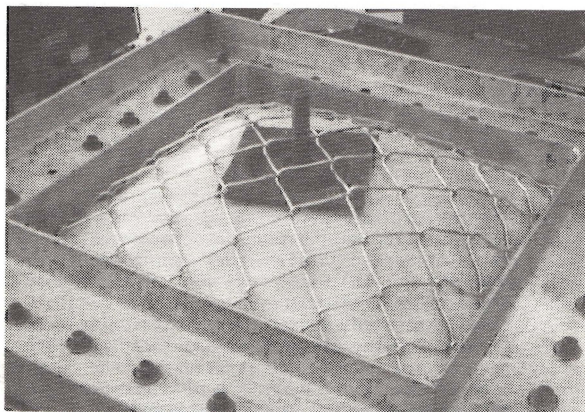


Fig. 6. Photograph of the testing apparatus.

plates placed on the chain link mesh. Also, polyethylene plates were placed on all rockbolts to prevent direct contact between the liner and the rock bolts.

Laboratory tests were conducted at The University of Michigan to evaluate the effect due to water pressure pressing the HDPE geomembrane against the chainlink mesh and rockbolts on the cavity walls. The actual testing apparatus is shown in Fig. 6. These tests verified that the HDPE liner would not be damaged.

CONCLUSION

When the Proton Decay Experiment reservoir was designed, some of the analyses conducted were considered very sophisticated, even excessively sophisticated. In fact, the very careful monitoring (2,3) of this reservoir shows that the actual mechanical behavior of the liner is even more complex than considered at the design stage. With the better knowledge gained today it would be possible to include in the design: (i) superposition of stresses resulting from a small depression into a large depression; (ii) combined effects of creep and yield; and perhaps, (iii) failure mechanisms linked to molecular reorientation of the high density polyethylene. On the other hand, the conceptual and detailed design of the leakage collection system would probably be done today identically. The use of plastic nets which was then a premiere for leakage collection and detection is now becoming a widely accepted practice.

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