Reservoir Lining for Pumped Storage Hydropower

Scoping Study of Geomembrane Lining Systems

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About HydroWIRES

In April 2019, WPTO launched the HydroWIRES Initiative* to understand, enable, and improve the contributions of hydropower and pumped storage hydropower (PSH) to reliability, resilience, and integration in the rapidly evolving U.S. electricity system. The unique characteristics of hydropower, including PSH, make it well suited to providing a range of storage, generation flexibility, and other grid services to support the cost-effective integration of variable renewable resources.

The U.S. electricity system is rapidly evolving, bringing both opportunities and challenges for the hydropower sector. While increasing deployment of variable renewables such as wind and solar have enabled low-cost, clean energy in many U.S. regions, it has also created a need for resources that can store energy or quickly change their operations to ensure a reliable and resilient grid. Hydropower (including PSH) is not only a supplier of bulk, low-cost, renewable energy but also a source of large-scale flexibility and a force multiplier for other renewable power generation sources. Realizing this potential requires innovation in several areas: understanding value drivers for hydropower under evolving system conditions, describing flexible capabilities and related tradeoffs associated with hydropower meeting system needs,

* Hydropower and Water Innovation for a Resilient Electricity System ("HydroWIRES")

optimizing hydropower operations and planning, and developing innovative technologies that enable hydropower to operate more flexibly.

HydroWIRES is distinguished in its close engagement with the DOE National Laboratories. Five National Laboratories—Argonne National Laboratory, Idaho National Laboratory, National Renewable Energy Laboratory, Oak Ridge National Laboratory, and Pacific Northwest National Laboratory—work as a team to provide strategic insight and develop connections across the HydroWIRES portfolio as well as broader DOE and National Laboratory efforts such as the Grid Modernization Initiative.

Research efforts of the HydroWIRES Initiative are designed to benefit hydropower owners and operators, independent system operators, regional transmission organizations, regulators, original equipment manufacturers, and environmental organizations by developing data, analysis, models, and technology research and development that can improve their capabilities and inform their decisions.

More information about HydroWIRES is available at https://energy.gov/hydrowires.

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Executive Summary

This report summarizes the results of a scoping study performed to research the use of geomembrane lining systems for pumped storage hydropower (PSH) reservoirs. The study consisted of a literature review for pertinent and publicly available information about geomembrane lining materials and their use in dams, reservoirs, and PSH as well as an assessment of the applicability of geomembrane lining systems to PSH reservoirs.

PSH power plants require water to be stored in reservoirs separated by an elevation difference to generate hydroelectric power during periods of electricity demand and then take excess energy off the grid by pumping water from the lower reservoir to the upper reservoir. When these reservoirs are excavated in or retained by earth or rockfill embankments, a seepage barrier is required to retain water within the reservoir. Traditionally these seepage barriers have been composed of cores of impervious soil or upstream linings constructed on the face of the embankments made from dense asphalt concrete (DAC) or reinforced concrete slabs. A more recent development is the use of geomembrane lining systems.

Geomembrane lining systems have been used for over 60 years in dams and reservoirs for multiple purposes, and using geomembrane lining materials in the design and construction of dams and reservoirs is well documented. The first application of geomembrane lining materials in a PSH reservoir found in the literature was for the 200 MW Mount Elbert pumped storage powerplant in Colorado, constructed by the United States Department of the Interior Bureau of Reclamation (USBR) in 1981, and the first PSH reservoir constructed using an exposed or uncovered geomembrane lining system identified in the literature was the 30 MW Okinawa Yanbaru seawater demonstration PSH project in Japan, which was completed in 1999. Newer PSH facilities have been designed and constructed using either covered or exposed geomembrane lining systems, including the 300 MW Mount Gilboa, 30 MW Calheta/Pico da Urze, 344 MW Kokhav Hayarden, and the 350 MW Abdelmoumen PSH projects. However, no new PSH facilities have been constructed in the United States using geomembrane lining systems since the Mount Elbert PSH powerplant, and we lack a body of knowledge in design, construction, and performance of these systems in PSH applications in the United States.

There is a number of design guidelines available for geomembrane lining systems for dams and reservoirs, and these guidelines provide pertinent and important design guidance to owners and engineers for the design of PSH reservoirs. These include guidelines provided by the International Commission on Large Dams (ICOLD), the American Water Works Association (AWWA), USBR, and Le Comité Français des Géosynthétiques (CFG). However, none of the design guidelines identified in this study focus on special design considerations for PSH reservoirs, which include water levels that fluctuate regularly between maximum and minimum elevations several times each day as well as a significant flow of water into and out of each reservoir during emptying and filling cycles, among others.

Furthermore, the review of regulatory guidelines for this study did not identify specific regulatory guidance related to the use and design of geomembrane lining systems for PSH reservoirs. The primary regulator for non-federally owned PSH reservoirs in the United States is the Federal Energy Regulatory Commission (FERC), and other agencies may be involved in the review and approval of PSH developments including the United States Army Corps of Engineers (USACE), USBR, and state dam safety agencies. Given the lack of specific regulatory guidance for geomembrane lining systems used in PSH reservoirs, close coordination and interaction with FERC and other regulatory agencies is required.

Several main takeaways have emerged from this study:

- Geomembrane lining systems are one of several lining systems that can be considered for the impervious lining of PSH reservoirs. Others include DAC and concrete linings, and the selection of one lining system over another must consider a number of factors.
- Given the variety of materials and factors that govern the design and construction of a geomembrane lining system, costs for the supply and installation of geomembrane lining systems should be expected to vary considerable from project to project and can be significant cost drivers for a given PSH project. The selection of a geomembrane lining system as the lining system for a PSH project is often based on factors other than costs, and in fact geomembrane systems may be more expensive than other options like DAC or concrete.
- The geomembrane used as the primary water barrier is only one part of a comprehensive lining system carefully designed to control seepage from a reservoir.
- There are a number of geomembrane materials available in the marketplace, and selection of a particular geomembrane material is subject to a variety of factors. There is not one material that can be considered superior in all respects.
- Detailed design of a geomembrane lining system will require involvement of a geomembrane lining manufacturer, and the owner and engineer will need to decide whether to select the manufacturer during preparation of the overall project design or leave final design details and selection of the manufacturer to the contractor.
- No PSH-specific design guidelines for geomembrane lining systems could be identified in the literature search conducted for this study.
- No PSH-specific regulations on the use of geomembrane lining systems could be identified in the literature search conducted for this study.

Topics for further study and investigation include the following:

- Expand the assessment of PSH liners to all lining systems (e.g., DAC and concrete).
- Perform a market assessment for potential liner applications.
- Further engage FERC and other relevant agencies to better understand regulatory guidance for geomembrane lining systems.
- Develop a cost model for pricing geomembrane lining system applications for PSH.
- Develop a preliminary reference design and cost assessment.

Acronyms and Abbreviations

AO	percentage of antioxidants in HDPE
ASTM	ASTM International (formerly American Society for Testing and Materials)
AWWA	American Water Works Association
CB	carbon black
CCL	compacted clay layer
CFG	Le Comité Français des Géosynthétiques
cm	centimeter (1/100 of a meter)
CPE	chlorinated polyethylene
CPER	reinforced chlorinated polyethylene
CSPE	chlorosulphanated polyethylene
DAC	dense asphalt concrete
DOE	Department of Energy
EC	engineering circulars
ELL	electrical leak location
EM	engineering manuals
EN	European Standards (European Norm)
EP	expanded polystyrene
ER	engineering regulations
EPC	engineer procure construct
EPDM	ethylene propylene diene terpolymer
ESC	environmental stress cracking
FERC	Federal Energy Regulatory Commission
FRR	flow rate ratio in relation to MI
GC	geocomposite
GCL	geosynthetic clay liner
GF	geofoam
GG	geogrid
GM	geomembrane
GN	geonet
GRI	Geosynthetics Research Institute
GSI	Geosynthetics Institute
GT	geotextile
HAC	hydraulic asphalt concrete
HDPE	high-density polyethylene
ICOLD	International Commission on Large Dams
in or "	inch

ISO	International Organization for Standardization
kg/m^3	kilogram per cubic meter
lb/ft^3	pound per cubic foot
LLDPE	linear low-density polyethylene
m	meter (unit of length in SI system)
m/s	meters per second (unit of velocity in SI system)
mm	millimeter (1/1000 of a meter)
mg/l	milligram per liter
mil	a unit used to measure geomembrane thickness (corresponds to 1/1000 of an inch)
MDPE	medium density polyethylene
MI	melt (flow) index
MW	megawatt
MWh	megawatt-hours
N	Newton (unit of force in SI)
NCTL	notched constant tension load test
NRCS	Natural Resources Conservation Service
PE	polyethylene
PET	polyester
PFMA	potential failure mode analysis
PP	polypropylene
PSH	pumped storage hydropower
PVC	polyvinylchloride
QUELTS	Queen's Experimental Liner Test Site
RCC	roller compacted concrete
RIDM	risk-informed decision-making
RPP	reinforced polypropylene
SI	International System of Units (aka metric system)
SQRA	semi-quantitative risk analysis
TRP	reinforced polyethylene
U.S.	United States
USA	United States of America
USACE	U.S. Army Corps of Engineers
USBR	U.S. Department of the Interior Bureau of Reclamation
UV	ultraviolet
WPTO	Water Power Technologies Office
WSB	water-saving basins

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1.0 Introduction

This report summarizes the results of a scoping study carried out to research the use of geomembrane lining systems in pumped-storage hydropower (PSH) reservoirs. This study consists of a literature review of pertinent and publicly available information concerning geomembrane lining systems and their use in PSH reservoirs and an assessment of the applicability of geomembrane lining systems to the unique operational requirements of PSH reservoirs.

The structure of this report includes this introduction to the study, a summary of the literature review carried out, a description of reservoir lining materials and practices identified in the literature, an assessment of the applicability of geomembrane lining systems for PSH reservoirs, and a summary of key findings and recommendations for further study. The core project team for the study consists of Argonne National Laboratory (Argonne), Oak Ridge National Laboratory (ORNL), and Stantec. The project team has collaborated with an Advisory Group consisting of industry experts as well as regulatory agencies and other stakeholders.

1.1 Research Rationale

PSH power plants require water to be stored in reservoirs separated by an elevation difference to generate hydroelectric power during periods of electricity demand and then take excess energy off the grid by pumping water from the lower reservoir to the upper reservoir. When these reservoirs are excavated in or retained by bedrock, earth, or rockfill embankments, a seepage barrier is required to retain water within the reservoir. Traditionally, these seepage barriers have been composed of cores of impervious soil, and upstream linings constructed on the face of the embankments are made from dense asphalt concrete (DAC) or reinforced concrete slabs.

A more recent development is the use of geomembrane lining systems for the upstream lining of PSH reservoirs. However, compared to other materials, geomembrane linings for PSH are new, and the body of knowledge in design, construction, and performance of these materials in PSH applications is less developed. As a result, the hydropower industry in the United States has an interest in developing such a body of knowledge so that planned PSH projects can more effectively consider geomembrane lining systems for PSH reservoirs, and engineers can more confidently design geomembrane lining systems for PSH applications.

1.2 Summary of Literature Review

1.2.1 Pumped-Storage Hydropower

PSH is a highly efficient way of storing electrical energy. The first PSH facilities originated in Italy and Switzerland at the end of the 19th century and the beginning of the 20th. The first PSH facility in the USA was built in 1931 by the Connecticut Electric and Power Company (Donalek, 2020) and pumped water from the Housatonic River to the storage reservoir near New Milford, Connecticut, which was located 70 m (230 ft) above the river. Since then, numerous PSH facilities have been built in the USA and around the world. The 2021 *Hydropower Market*

Report (DOE, 2021) lists 43 PSH plants in the USA, which account for 95% of the nation's utility-scale energy storage.

A PSH plant uses the water stored in an upper reservoir to produce energy as it flows downstream through a penstock (conduit) to a turbine. In open-loop PSH plants, the turbined water is released into a river. In closed-loop PSH plants, the turbined water is stored in a lower reservoir. The water in the upper reservoir is replenished by pumping water to the upstream reservoir either from the river (in open-loop PSHs) or from the lower reservoir (in closed-loop PSHs).

A PSH plant can be viewed as a battery that stores hydroelectric energy as the potential energy of the water in the upper reservoir, and it is regarded as a very efficient way of storing energy. Although the energy required to pump water to the upper reservoir is larger than the energy produced, the real importance of PSH plants is in their efficient storage capability. The overall round-trip efficiency of a PSH plant can be as high as 70%–85%. The ability of PSH plants to store energy efficiently proves to be extremely useful for addressing load variations on grids with a large number of heterogeneous electrical power generators. In recent years there has been a renewed interest in PSH plants because they can be used with other renewable energy resources, such as solar energy and wind energy, which are used to pump water to the upper reservoir. For more detailed information, the reader is directed to a number of references on PSH plants that are listed in Section 5.2 in the Appendix.

1.2.2 PSH Reservoirs

One or both of the upper and lower reservoirs of PSH plants can be natural watercourses or pre-existing waterbodies (open-loop PSH), or they can be manmade reservoirs constructed of concrete, roller compacted concrete (RCC), hardfill, or earth fill. Earth fill reservoirs are built by taking advantage of natural depressions in the topography to minimize the costs associated with excavation and construction of the surrounding embankments and dikes. They are generally surrounded by earthen embankments with sloping inner and outer faces designed to provide the designed storage volume, and they typically include various types of concrete inlet and outlet structures.

The standard design of a manmade water storage reservoir takes into account various criteria that ensure a safe, reliable and sustainable operation during its lifetime, such as durability, impermeability and weather resistance. The upper and lower reservoirs of PSH plants are generally subjected to cycles of rapid drawdown and filling cycles as the water is released from the upper reservoir into the lower reservoir to produce energy and pumped from the lower reservoir to the upper reservoir for storage. Therefore, for the reservoirs of PSH plants, slope stability during rapid drawdown and filling cycles is an important consideration. Conservation of the water used for energy production and protection of groundwater resources are also important criteria and require minimization of losses due to seepage. The Appendix (Section 5.3) provides a list of references on reservoirs and their construction, especially focusing on leakage, slope stability, and water conservation and evaporation.

Effective reservoir lining systems used by engineers to minimize the loss of reservoir volume by seepage include compacted clay, concrete, and asphalt concrete. It is important to note that throughout this document we will be using the term “lining system” or “liner system.” This is because the material used as a barrier layer is generally designed together with a drainage and support layer, which serves as foundation and safely collects leakage through the liner, and often with a cover layer, which protects the barrier material. The Appendix (Section 5.10) provides various references that review these traditional lining systems and provide comparisons.

1.2.3 Geomembrane Lining Systems

In the late 1960s and early 1970s, geomembrane systems started to be used as liners in reservoirs. They became a popular liner material because of their extremely low permeability and because their foundation and cover layers are thinner than the traditional systems.

Geomembranes are part of a larger group of materials called geosynthetics. ASTM D4439-20 defines a geosynthetic as “a planar product manufactured from polymeric material used with soil, rock, earth or other geotechnical related material as an integral part of a human made project, structure or system.” The term “geosynthetics” refers to a large variety of products, which are generally categorized as follows (after Cuelho, 2012): 1) geotextiles, 2) geogrids, 3) geonets, 4) geomembranes, 5) geosynthetic clay liners, 6) geofoam, 7) geocomposites, and 8) “geo-others.” All these materials have their specific applications. The present report focuses especially on geomembranes as liner materials. The Appendix (Section 5.4) lists useful references on polymeric materials in general with a special focus on geosynthetics.

According to ASTM 4439, a geomembrane is a very low-permeability synthetic membrane liner or barrier used with any geotechnical engineering-related material so as to control fluid (or gas) migration in a man-made project, structure, or system. ISO 10318-1:2015 defines a geomembrane as follows: “factory-assembled structure of geosynthetic materials in the form of a sheet in which the barrier function is essentially fulfilled by polymers” (ISO, 2015). Obviously, both definitions emphasize the use of geomembranes as a barrier material due to their extremely low permeability. While no material is completely impervious, the hydraulic conductivity of geomembranes ranges from 1×10^{-15} to 1×10^{-12} m/s. In comparison, a 0.5 m thick compacted clay liner made up of several layers less than 0.15–0.20 m thick offers a hydraulic conductivity of about 1×10^{-9} m/s (Science Direct, 2022; Cossu and Stegman, 2019).A geomembrane a few millimeters thick with proper support and cover layers, which may include natural materials such as soil, gravel, and stones as well as other types of geotextiles, offers hydraulic conductivity several orders of magnitude smaller than a 0.5 m thick compacted clay layer, which also needs support and cover layers. Muralikrishna and Manickam (2017) mention that according to national and international standards, the thickness of clay liner systems can be 1.0–1.5 m without the foundation and protective cover layers. Geomembrane liner systems generally are considerably less thick, which leads to a larger storage volume for the same amount of excavation.

Most geosynthetic materials are made from synthetic polymers, broadly called plastics (Koerner, 2016a). They are manufactured using various methods: by extrusion as thin contiguous polymeric sheets, by impregnation of geotextiles with asphalt, elastomer or polymer sprays, or as multilayered bitumen geo-composites. Different types of geomembranes commonly used in civil and geotechnical applications and as reservoir liners, differentiated by their chemical composition and manufacturing process, are available commercially:

- High-density polyethylene (HDPE), developed in 1941
- Linear low-density polyethylene (LLDPE), developed in 1956
- Polypropylene (PP), developed in 1957
- Polyvinylchloride (PVC), developed in 1927
- Polyester (PET), developed in 1950
- Expanded polystyrene (EP), developed in 1950
- Chlorosulphanated polyethylene (CSPE), developed around 1965
- Thermoset polymers, such as ethylene propylene diene terpolymer (EPDM), developed in 1960

Different types of geomembranes have different properties that make them suitable for specific applications (see Appendix Section 5.1 for general references). The properties of geomembranes and the methods of measuring these properties are of paramount importance from the point of view of design considerations and selection of the most suitable material for a specific project, taking into account both technical and economic considerations. The properties of geomembranes can be categorized as follows:

- *Physical* properties include thickness, texture, density, melt index, mass per unit area, water vapor transmission, solvent vapor pressure, and so on.
- *Mechanical* properties include tensile behavior, tear resistance, impact resistance, puncture resistance, interface shear, anchorage, stress cracking, and so on.
- *Endurance* properties define the resistance of geomembranes to various environmental factors such as ultraviolet light, radioactive degradation, bacteria, chemical substances, thermal extremes and thermal variations, oxidation, and synergistic effects.
- *Methods of lifetime prediction*, which aim to predict long-term behavior of the material using accelerated aging.

Koerner (2016a and b) discusses in detail the properties listed above and the standardized measuring techniques that are used to measure them. In the Appendix, Section 5.5.2 provides a list of references on the properties of geomembranes, and Section 5.5.3 references on the methods of measuring their properties. Mueller (2007) provides detailed information on the properties of HDPE as they relate to geotechnical applications and discusses measuring methods. These specifications, however, concern only the geomembrane and do not discuss specifications related to support or cover layers.

Koerner (2016b) presents some of the methods used in selecting and designing geomembrane liners for liquid containment reservoirs. The discussion covers geometric considerations, typical cross sections, selection of geomembrane material, geomembrane thickness, and side slope and anchor trench design. Additional references related to designing with geomembranes can be found in Section 5.5.4 of the Appendix, and references related to the use of geomembrane liners in water reservoirs are listed in Sections 5.7.1 and 5.7.2. Section 5.7.3 contains references for the use of geomembrane liners in hydropower plants.

Geomembrane liner systems may fail for a variety of reasons. Section 5.8 of the Appendix lists a number of references related to geomembrane liner failure modes, failure case histories, and mitigation methods. Detecting leaks that may occur due to aging, construction defects, animal activity, etc., is important to ensure the safe operation of the reservoir and to prevent the loss of stored water. In some cases, geomembrane liners are built with permanent leak detection systems to monitor the performance of the liner continuously. In other cases, temporary leak detection systems are installed to detect and localize suspected leaks. Both Koerner (2016b) and Mueller (2007) discuss leakage rates, leak detection methods, and repair methods for geomembrane liners. Additional references that discuss leak detection methods are listed in Section 5.5.5 of the Appendix. In addition, general references related to infiltration rates from reservoirs and water conservation methods can be found in Section 5.3.3 of the Appendix.

Currently, geomembranes are used for a variety of large-area lining purposes: dams, dikes, reservoirs, a variety of treatment basins (such as tailings ponds or leaching ponds in mineral and ore processing), landfill basal liners, landfill capping, sealing large areas for the containment and remediation of contaminated land, tunnel construction, canal construction, and large-area contiguous liners in industrial plants and road construction. In the USA, geomembrane liners are widely used for hazardous waste containment ponds. Mueller (2007) presents the history of geomembrane liners in waste landfill applications. The history of geomembrane liner use in the mining industry can be found in Breitenbach and Smith (2006). Section 5.7.4 of the Appendix lists a collection of references related to various environmental applications of geomembrane liners. Geomembranes are sometimes used as floating covers to protect water quality and to reduce evaporation, especially in the case of potable water reservoirs. Section 5.13 in the Appendix lists a number of references discussing evaporation from reservoirs and the use of geomembranes as floating covers.

Geomembranes are also used as sealing barriers for various hydraulic structures, such as dams. A list of references on the use of geomembranes in hydraulic structures can be found in Section 5.6 of the Appendix. Geomembranes are also used for repair and refurbishment of concrete structures, as reviewed by Nacer (2008). (See Section 5.11 in the Appendix.)

A review of both successful applications and failures of geomembrane systems in various projects provides invaluable information. Section 5.9 in the Appendix contains references that discuss case histories of geomembrane liner use in various types of projects. Giroud (2019) discusses the lessons learned from the case histories of reservoirs lined with geomembrane systems, including reservoirs of PSH plants. A keynote presentation by Giroud (2005) discusses geosynthetic engineering successes, failures and lessons learned. General references related to use of geomembranes can be found in Section 5.1 of the Appendix.

Only a few countries, such as France and the U.S., provide guidelines, specifications, and certifications standards for the use of geomembrane systems as barrier systems. The French Committee on Geosynthetics has published a general recommendations document, which is also available in English (see Comité Français des Géosynthétiques, 2017). The American Water Works Association (AWWA) has also published general recommendations for using geomembrane sealing system designs for potable water applications (AWWA, 1999 and AWWA, 2002). USBR provides detailed guidance on the use of geomembranes in embankment dams in their embankment dams design standards (USBR, 1992) *No. 13, Embankment Dams, Chapter 20, Geomembranes*. Lastly, the industry association International Commission on Large Dams (ICOLD) published an overview of the application of geomembrane sealing systems to various types of dams in ICOLD *Bulletin 135* (ICOLD, 2010). These guidelines are discussed in more detail in Section 3.2 of this report.

Zanzinger (2017) provides a comparative look at specifications, certifications and standards for geomembranes in the USA, France, Germany, and other countries and discusses the efforts for setting standards for environmentally critical applications in Africa. Section 5.16 in the Appendix lists references for geomembrane application specifications, certifications and standards. In the USA, the Geosynthetics Research Institute (GRI) has published a number of specifications, mostly related to the different types of geomembranes (see Section 5.16 of the Appendix).

The literature survey showed that there does not exist a comprehensive book or report or paper on geomembrane systems in reservoirs. The available information is scattered throughout a number of publications in different fields. Environmental applications for containing solid or liquid wastes are the most widespread use of geomembrane systems as barriers. Some books and conference proceedings on geomembranes, their properties, and design methods for various applications are listed in Section 5.14 of the Appendix. Koerner (2016a and b) and Mueller (2007) provide comprehensive outlooks on the use of geomembranes and their applications. Toepfer (2015) provides a field guide for ensuring the quality of geomembrane installations that covers the subgrade, geomembrane deployment, seams and their destructive and nondestructive testing, cover layers, and repairs.

Section 5.15 in the Appendix provides a partial list of major geomembrane companies and products as well as a limited list of references on installation guidelines.

1.3 Appendix Overview

The Appendix provides a categorized list of more than 350 references. The categorization, however, must be interpreted loosely because many of the references may belong to more than one category. Table 1-1 lists the number of references in each category.

Table 1-1 Categorized List of References Provided in the Appendix and the Number of References in Each Category.

	Heading	Number of References	Comments
5.1	General	6	
5.2	Pumped Storage Hydropower	14	
5.3	Reservoirs		
5.3.1	Leakage	3	
5.3.2	Slope Stability	12	
5.3.3	Water Conservation and Evaporation	11	
5.4	Geosynthetics: General		
5.4.1	Polymeric Materials	3	
5.4.2	Geosynthetics	1	
5.5	Geomembranes		
5.5.1	Geomembranes	4	
5.5.2	Properties of Geomembranes	5	
5.5.3	Testing. Monitoring and Performance of Geomembranes	36	
5.5.4	Designing with Geomembranes and Geosynthetics	37	
5.5.5	Detection of Failures	7	
5.6	Application of Geomembranes in Hydraulic Structures	17	
5.7	Geomembranes as Reservoir Liners	1	
5.7.1	Water Reservoirs	6	
5.7.2	Geomembranes for Potable Water Reservoirs	2	
5.7.3	Geomembranes for Hydropower Plants	7	
5.7.4	Geoenvironmental Applications	25	
5.8	Failures of Geomembrane Liners	16	
5.9	Case Histories	41	
5.10	Other Types of Liners		
5.10.1	Comparisons	5	
5.10.2	Compacted Clay Liners	10	
5.10.3	Asphaltic Concrete Liners	9	
5.10.4	Other Types of Liners	1	
5.11	Geoliners and Concrete Structures	1	
5.12	Specifications	1	
5.13	Geomembranes as Floating Covers		
5.13.1	Evaporation from Reservoirs	2	Websites referenced lead to multiple documents
5.13.2	Geomembranes as Floating Covers	4	
5.14	Books	8	
5.15	Documents from Industry		
5.15.1	Companies and Products	14	
5.15.2	Installation Guides	6	
5.16	Specifications Certifications and Standards	17	
5.17	Other Applications of Geomembranes	1	

1.4 Overview of Liner Application and Deployment

Geomembrane lining systems have been used for over 60 years in dams and reservoirs for multiple purposes, including reservoirs for flood control, recreation, agriculture, hydropower generation, storage, mine tailings storage, and liquid waste storage. The use of geomembrane lining materials in the design and construction of dams and reservoirs for these purposes is well documented, and geomembrane lining systems are widely accepted for uses in waste storage and water storage. More recently, geomembrane lining systems have been incorporated into hydropower dams and PSH reservoirs, and a growing list of hydropower facilities in operation around the world have successfully used geomembrane lining systems in their design (ICOLD, 2010).

Although PSH reservoirs and other water storage reservoirs share many of the same features and design, construction, and operation considerations, the main way in which PSH reservoirs differ from those of other reservoirs is that water levels in PSH reservoirs fluctuate regularly to the extremes of the reservoir water level ranges. Water levels in PSH reservoirs can fluctuate by more than 20 meters several times each day. This adds additional complexity to the design of PSH reservoirs:

- Most water storage reservoirs are designed to withstand conditions of rapid water drawdown in the event of uncontrolled water releases, but such drawdown events are considered uncommon and extreme. For PSH reservoirs, rapid drawdown of the reservoir is considered a normal loading condition. Both the dam and water seepage barrier must be designed to withstand this loading condition as a matter of normal operation. The geomembrane lining system is also exposed to a potential buildup of uplift forces beneath the lining if reservoir water builds up behind the lining due to leaks. In addition, the lining and supporting layers are exposed to frost via contact with below freezing air temperatures during winter months.

- Since PSH reservoirs are essentially filled and emptied multiple times each day, there is a significant flow of water into and out of each reservoir. These flows can be up to tens of thousands of cubic feet per second and impart significant flow velocities and shears on the floor and adjacent side slopes of the reservoir and dam. The geomembrane lining system can be exposed to higher forces than typically experienced in normal water or waste storage reservoirs.

For these reasons, the use of geomembrane lining systems in PSH reservoirs is less common. The first use of geomembrane lining materials in a PSH reservoir found in the literature was for the 200 MW Mount Elbert pumped storage power plant in Colorado, constructed by USBR in 1981. The geomembrane lining system was a covered system installed to mitigate seepage issues stemming from the initial clay lining designed and constructed at the reservoir forebay (see Section 3.4.1 of this report for more information). The first PSH reservoir constructed using an exposed geomembrane lining system identified in the literature was for the 30 MW Okinawa Yanbaru seawater demonstration PSH project in Japan. The project was completed in 1999 and consisted of an upper reservoir that incorporated an HDPE geomembrane lining system (see Section 3.4.2 of this report for more information). Several newer PSH facilities have been designed and constructed internationally using geomembrane lining systems for the seepage barriers of one or both reservoirs (see Section 3.4 of this report for information on some of these facilities); however, as of the writing of this report, no new PSH facilities have been constructed in the United States that have used geomembrane lining systems.

Given the scarcity of new PSH development in the United States, domestic industry knowledge of the design and construction of geomembrane lining systems for PSH reservoirs is lacking.

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2.0 Reservoir Lining Materials and Practices

2.1 Reservoir Lining Technologies

The following sections provide a brief overview of different reservoir lining systems and materials.

2.1.1 Why and When a Reservoir Liner is Needed

Surface reservoirs are built to store water, or other liquids and solids, for specific purposes. Regardless of the purpose, the efficient and reliable containment of the stored material with a minimum of loss to the environment is the intrinsic property of a reservoir.

In this document, we primarily focus on surface water reservoirs that store water as part of a PSH facility. The water stored in the reservoir is a precious resource. Since it can be converted back to electrical energy with a very high round-trip cycle efficiency (about 70%-85%), the stored water can be considered stored electrical energy. Thus, conserving stored water reliably and efficiently and minimizing losses due to infiltration is an important criterion in the design of surface reservoirs for PSH plants.

Surface reservoirs are generally built by excavating soil or bedrock. In some cases, the designers may take advantage of natural depressions in the topography to minimize the excavation amount. Often, the excavated material can be used to build embankments around the reservoir to create the reservoir volume with a minimum amount of excavation. In all cases, however, the characteristics, especially the hydraulic conductivity, of the soil or bedrock layer forming the bottom and the sides of the reservoir control the water holding capability of the reservoir and the amount of water that will be lost due to infiltration through the bottom and sides of the reservoir. Reducing seepage from the reservoir has purposes other than just conserving stored water:

- It reduces the risk of piping (internal erosion) around the reservoir and prevents downstream slope instability due to the elevated phreatic surface (and decreased effective stress) in the downstream portion of an embankment projecting above the ground surface.
- It improves the stability of the side slopes to withstand repeated rapid filling and drawdown cycles.
- Depending on the geotechnical characteristics of the substrate on which the reservoir is built, it protects against potential weakening due to seepage water (for example in karstic areas).
- It protects against the potential contamination of the ground water.

Often the site where the reservoir will be built is dictated by criteria other than the type of the soil or the bedrock on which it is built; thus, it becomes necessary to line the reservoir bottom and side slopes with a barrier system with a very low hydraulic conductivity in order to minimize the losses due to seepage, to protect the reservoir embankments against piping, and to protect the groundwater. The decision of whether the installation of a liner system is necessary begins with the hydraulic conductivity of the material in which the reservoir volume is created. Table 2-1 lists typical ranges of hydraulic conductivity (permeability) for four soil groups, identified based on the percentage passing through a No. 200 sieve and the plasticity index (PI) (NRCS, 2009).

Table 2-1 Categorization of Soils Based on Their Estimated Range of Permeability (adapted from NRCS 2009)

<table>
<tr><th colspan="3" rowspan="2"></th><th colspan="5">Soil Permeability Group Number</th></tr>
<tr><th>I</th><th colspan="2">II</th><th>III</th><th>IV</th></tr>
<tr><td rowspan="13">Unified Soil Classification System (USCS)[1] groups and their occurrences in soil types I to IV</td><td>GW</td><td>Well-graded gravel, fine to coarse gravel</td><td>Always</td><td colspan="2">Never</td><td>Never</td><td>Never</td></tr>
<tr><td>GP</td><td>Poorly graded gravel</td><td>Always</td><td colspan="2">Never</td><td>Never</td><td>Never</td></tr>
<tr><td>GM</td><td>Silty gravel</td><td>Sometimes</td><td colspan="2">Usually</td><td>Sometimes</td><td>Sometimes</td></tr>
<tr><td>GC</td><td>Clayey gravel</td><td>Never</td><td colspan="2">Sometimes</td><td>Usually</td><td>Sometimes</td></tr>
<tr><td>SW</td><td>Well-graded sand, fine to coarse sand</td><td>Always</td><td colspan="2">Never</td><td>Never</td><td>Never</td></tr>
<tr><td>SP</td><td>Poorly graded sand</td><td>Always</td><td colspan="2">Never</td><td>Never</td><td>Never</td></tr>
<tr><td>SM</td><td>Silty sand</td><td>Sometimes</td><td colspan="2">Usually</td><td>Sometimes</td><td>Sometimes</td></tr>
<tr><td>SC</td><td>Clayey sand</td><td>Never</td><td colspan="2">Sometimes</td><td>Usually</td><td>Sometimes</td></tr>
<tr><td>ML</td><td>Silt (inorganic)</td><td>Never</td><td colspan="2">Usually</td><td>Sometimes</td><td>Never</td></tr>
<tr><td>CL</td><td>Lean clay (inorganic)</td><td>Never</td><td colspan="2">Sometimes</td><td>Usually</td><td>Sometimes</td></tr>
<tr><td>CL-ML</td><td>Lean clay and silt</td><td>Never</td><td colspan="2">Always</td><td>Never</td><td>Never</td></tr>
<tr><td>MH</td><td>Elastic silt (inorganic)</td><td>Never</td><td colspan="2">Sometimes</td><td>Usually</td><td>Sometimes</td></tr>
<tr><td>CH</td><td>Fat clay (inorganic)</td><td>Never</td><td colspan="2">Never</td><td>Sometimes</td><td>Usually</td></tr>
<tr><td colspan="3">Percentage fines</td><td><20</td><td>≥20</td><td><20</td><td>≥20</td><td>≥20</td></tr>
<tr><td colspan="3">PI (plasticity index)</td><td>5</td><td>≤15</td><td>≥5</td><td>16≤ PI ≤30</td><td>>30</td></tr>
<tr><td colspan="2" rowspan="2">Estimated hydraulic conductivity (m/s)</td><td>Low</td><td>3×10^{-5}</td><td colspan="2">5×10^{-8}</td><td>5×10^{-10}</td><td>1×10^{-11}</td></tr>
<tr><td>High</td><td>2×10^{-2}</td><td colspan="2">5×10^{-6}</td><td>1×10^{-11}</td><td>1×10^{-9}</td></tr>
</table>

[1] ASTM Method D–2488 has criteria for use of index test data to classify soils by the USCS.

Table 2-1 provides a rough guide, based solely on the type of soil, but does not account for various other factors that affect the permeability of the soil. Normally, soils with higher dry density have a smaller percentage of voids and lower hydraulic conductivity. In some cases, however, clayey soils with low permeability may form blocks with cracks due to desiccation. These cracks form preferential pathways for water that increase permeability by orders of magnitude. The chemical composition of the soil and its interaction with the stored water may also modify its hydraulic conductivity. The anisotropy of the hydraulic conductivity of a soil layer, i.e., different hydraulic conductivity in horizontal and vertical directions, is another factor that must be considered. All these factors must be carefully considered when making engineering decisions about reservoir design and decisions about the liner system.

Figure 2-1, based on the data from Freeze and Cherry (1979) and NRCS (2009), shows the approximate limits of specific or intrinsic permeability, k, and hydraulic conductivity, K, for various types of soils and rocks, which are based on Freeze and Cherry (1979). Permeability, k, and the hydraulic conductivity, K, can be converted from one to the other assuming that the flowing liquid is water as shown in the figure, where μ and ρ are the absolute viscosity and density of water at 20°C (68°F).

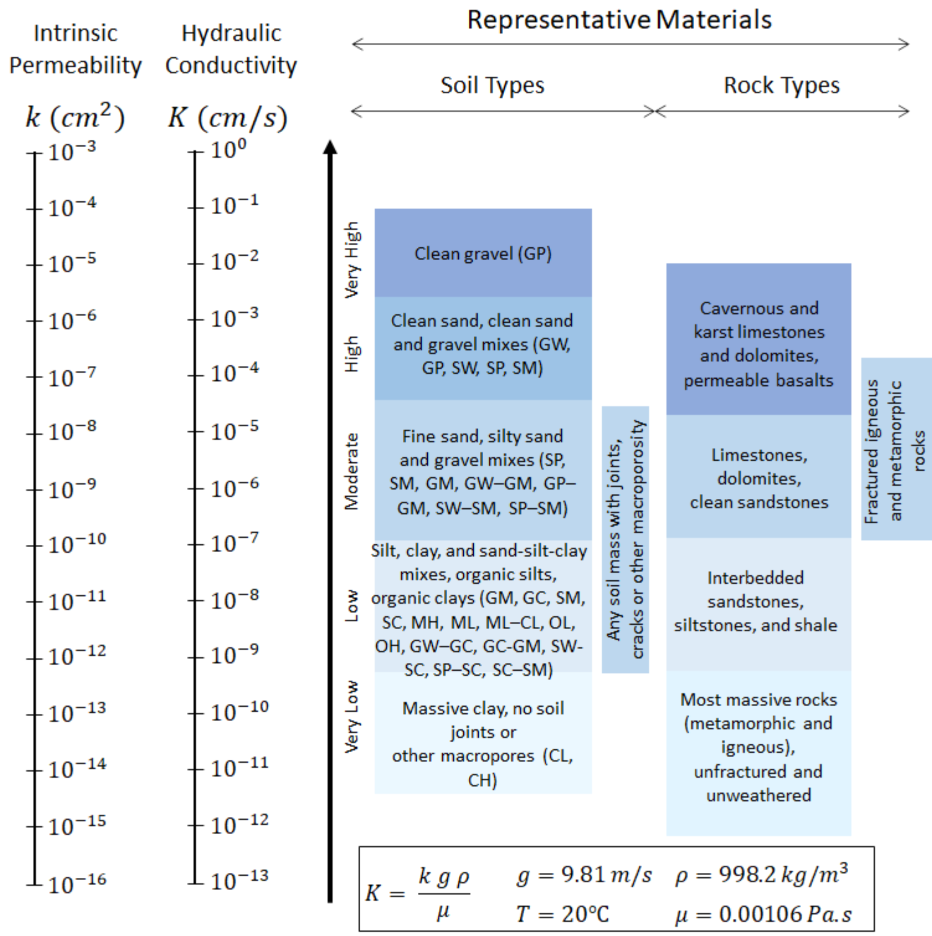


Figure 2-1 Range of Values of Permeability and Hydraulic Conductivity and Permeability.

2.1.2 Reservoir Lining Systems

A reservoir lining system is generally composed of multiple layers, at least one of which is the barrier layer that provides containment by minimizing the infiltration losses into the ground and into the embankment structures. The detailed design of the reservoir lining system is project specific and takes into account various factors, such as the geotechnical and chemical properties of the reservoir area and the embankments designed to create the reservoir volume, the type of barrier material used and type of liquid to be stored, the cost, the availability of materials, constructability, etc. Therefore, this discussion will only focus on some general principles.

The simplest type of reservoir lining system is composed of three layers:

1. *Barrier Layer:* The essential element of the system is the barrier layer, which has a very low hydraulic conductivity that reduces seepage and ensures containment of the liquid stored. The selection of barrier material takes into account various factors such as hydraulic conductivity, type of liquid stored, geometry of the reservoir, physical and mechanical properties of the barrier material, endurance properties, lifetime, the local availability and the costs associated with the barrier liner material, its installation, and the operational and maintenance costs. The type of material used as the barrier layer has evolved over time in parallel with the scientific and technological development of engineering materials. This will be discussed in detail later in this section.

2. *Support Layer:* The support layer—a barrier layer with low permeability—is not typically directly laid over the excavated foundation material or substrata for various reasons. Generally, a support layer is constructed over the foundation and the barrier layer is placed over this support layer. The support layer allows proper installation of the barrier layer, but may have also other functions, such as capturing and safely draining away any leaks from the barrier layer and/or preventing frost heave.

3. *Protective Cover:* Often it is necessary to build a suitable protective cover layer over the barrier layer. This protective layer minimizes the damage to the barrier layer due to natural factors or human activity. Depending on the type of barrier material used, the natural factors may include desiccation of the barrier material, deterioration due to ultraviolet (UV) light, lifting of the barrier material by wind shear, etc. Human and animal activity during low reservoir levels and vandalism may damage an exposed geomembrane.

There are no standard guidelines for the design of surface reservoir liners. The designs are site specific and vary depending on the barrier material used. The engineering design of a liner system has to take into consideration that the properties of the barrier material used in the liner system may be subject to defects during manufacturing or placement. The properties of the barrier material also evolve with time, and defects may appear due to various factors (temperature variations, stresses, animal activity, chemical processes, etc.). For this reason, the real engineering design of the liner system may involve more layers with different functions to provide durability and a long service life. The layers may also include instrumentation to monitor and quantify leakages from the barrier layer. A good liner system is one designed to operate safely and reliably under all operating conditions. The liner system design should also allow for and facilitate repairs as much as possible.

Figure 2-2 shows the conceptual design for a generic multi-layer reservoir liner system for storing non-rigid waste based on Appendix C in *Liners for containing pollutants, using synthetic membranes* (Government of Western Australia, Department of Water, 2013). The attributes for different layers are also listed in the figure. Although the liner system shown in Figure 2-2 is designed for storing non-rigid wastes, the design considerations are also valid to a large extent for surface water PSH reservoirs.

Top

Liner Material		Attributes
Upper granular protective layer		• Protects liner/drain system during contained material deposition • Minimizes drying out of compacted soil liners • Permits recovery of residue and minimizes risk of liner damage • Prevents UV light damage to PVC liners
Subsoil drains over primary liner		• Aids depressurization of primary liners • Enhances solid residue consolidation • Enhances fluid recovery from slurries • Normally requires protection against fine solid intrusion using fabric wrap
Primary liner (select one)	Compacted clay	• Hydraulic permeability less than 10^{-9} • Normally it will not deteriorate when in contact with stored material
	High density polyethylene (HDPE)	• Abrasion and ultra-violet light resistant • Heat welded or lapped mechanical joints
	Polyvinyl chloride (PVC)	• Will stretch before shearing in disturbed ground • May lose plasticity with time • Susceptible to UV deterioration (unless treated) • Unsuitable for retention of some chemicals such as petroleum products, which cause liner deterioration
	Composite synthetic	• Plastic coated outer/fiber reinforced mesh inner liner • Polypropylene mesh encasing bentonite
Under drainage/ monitoring layer (select one)	Granular layer with drainpipes to collector pit	• Permits seepage monitoring • Permits seepage recovery • Depressurizes secondary liners
	Geo-textile mesh or net separator, drains to collector pit	• Ease of installation • Takes up little volume
Secondary liner (select one of first two)	Compacted soil	• Natural or imported low permeability soil (less than 10^{-9} m/s when compacted)
	HDPE, PVC or composite synthetic	• As given above for primary liner
	Geo-textile underlay	• Used as necessary to separate incompatible soils or prevent perforation of synthetic liners.
Liner sub-base: External underdrainage layer		• Used as necessary to separate incompatible soils or prevent perforation of synthetic liners.
Natural soil base layer		• Surface cleared of vegetation, free of stones exceeding 25 mm diameter and any other material that may cause damage to a liner • Clean soil fill, well-graded and compacted to provide a firm unyielding foundation sufficient to permit the movement of vehicles and welding equipment without causing rutting or other deleterious effects • Layer should be greater than two meters above maximum wet season water table

Bottom

Figure 2-2 Materials Typically Used in Non-Rigid Waste-Containment Systems (Government of Western Australia, 2013).

The generic liner system shown in Figure 2-2 involves two barrier (liner) layers: a primary liner and a secondary liner, with an underdrainage and monitoring layer in between. This barrier layer redundancy is typically specific to waste containment reservoirs because of concerns about contamination of the ground water. However, depending on the geotechnical characteristics of the area in which the reservoir is constructed, if leakages due to defects in the reservoir layer have the potential to affect the safety of the reservoir, it is not uncommon to have a double liner system for a water storage reservoir as well. Examples of double liner systems will be given and discussed in other chapters.

Two principal types of barrier material are listed for the primary liner layer: compacted clay and geosynthetic materials. Although there are undoubtedly many choices, Figure 2-2 shows three types of geosynthetic liners. HDPE and PVC are geomembranes, and the composite synthetic is an assembly of at least two geosynthetic products: a geomembrane and a geotextile. The advantages and disadvantages of these different materials are briefly mentioned in the Attributes column.

Although absent from Figure 2-2, other barrier materials that may be considered, and that have been used for water storage and PSH reservoirs, include dense asphalt concrete (DAC) and reinforced concrete panels, like those as used in concrete-faced rockfill dams (CFRD). In the 1980s and early 1990s, geomembrane liners were thought to have a service life of less than 100 years, while a compacted clay liner was believed to have a longer service life. This view is no longer held, because it has been shown that a single conventional clay layer, typically 1 m thick, can be damaged by root penetration, burrowing animals, and crack formation caused by desiccation. It has also been shown that the aging process of certified and properly installed geomembranes is so long that it is no longer a design consideration.

The secondary liner layer may use the same type of material as the primary liner layer. The secondary liner layer may be placed over a geotextile underlay, which helps redistribute loads to the underlying linear sub-base and prevents the perforation of the secondary geomembrane.

The underdrainage or monitoring layer sandwiched between the primary and secondary liners may be equipped with sensors for detecting any leakage from the primary liner. The causes of leakage from barrier layers will be discussed in more detail in the next section.

For waste containment, Rowe (2012b) recommends the use of a much simpler composite liner system consisting of a geomembrane over a clay liner, which can be either a compacted clay liner or a geosynthetic clay liner. The geomembrane is usually a high-density polyethylene with a thickness ranging from 1.5 to 2.5 mm. According to the data collected by Rowe (2012a and b), the composite liners with a geosynthetic clay liner perform better than composite liners with compacted clay liners.

2.1.3 Liner Systems with Different Barrier Materials

The following sections describe the liner systems that are most commonly used for water reservoirs, including hydropower and PSH reservoirs.

2.1.3.1 Compacted Clay Liners

Compacted soil liners have long been used as barrier layers for liquid containment in ponds and reservoirs. They are generally made of fine-grained, natural mineral materials having low permeability. Typically, these are soils containing a high percentage of clay. In Table 2-1, these soils are inorganic clays of the low to medium plasticity (CL), inorganic clays of high plasticity (CH), and clayey sands (SC) USCS soil classes. (See also ASTM D-2487.) CL and CH classes would have 50% or more material with a particle size smaller than a No. 200 sieve (<75 μm), and SC class soils have a 50% or more coarse material smaller than a No. 4 sieve (4.75 mm) mixed with at least 12% fines smaller than a No. 200 sieve.

The soils to be used as compacted clay liner can be excavated *in situ* or excavated in a borrow pit in another location and brought to the site. Usually, various standard soil tests need to be performed to ascertain the suitability of the soil to be used as liner. The property of highest importance is, of course, the permeability, but the design engineers and contractors may also specify limits on the percentage of fines and the percentage of gravel content. Atterberg tests determine the moisture content of the fine-grained soil transitions between different phases (liquid limit, plastic limit, and plasticity index) and can be used to assess shear strength, to forecast settling, and to identify potentially expansive soils. These tests help to ascertain that the soil conforms to the specifications. However, a qualified construction quality assurance (CQA) inspector's observation is necessary for the soil to be approved for use as liner material for a specific project (Daniel, 1993).

Although in some cases soils can be used directly, often they need to be processed to break down large clods and to remove larger particles, stones and impurities. Non-uniform soils may be homogenized. The water content of the soil may also need to be adjusted before it is compacted to construct the barrier layer: drying it if it is too wet and wetting it if it is too dry. In some cases, other materials, such as bentonite, may be added to the processed soil. The processed material is inspected and approved by CQA and construction quality control (CQC) inspectors before the placement of the liner.

A compacted clay layer requires a suitable subgrade layer that will provide adequate support for compaction during the placement of the barrier layer. The subgrade layer should be properly prepared to avoid soft spots and compacted to provide a firm and smooth surface, especially if a geotextile is going to be used between the subgrade and the compacted clay layer. The final preparation of the surface is verified and approved as part of the normal CQA process.

Processed material approved by CQA and CQC inspectors is placed on top of the subgrade (and the geotextile if it is used) at the bottom and sides of the reservoir in lifts of appropriate thickness compacted according to specifications.[*] The thickness of the lifts must be carefully controlled and kept at the specified value in order to ensure that the compaction energy is properly transmitted to the bottom of the lift (Daniel, 1993). Requirements and specification for compaction during the construction process are beyond the scope of this introductory text and can be consulted in Daniel (1993) and various books on geotechnical engineering. Various tests are performed to ensure the proper compaction of the liner, such as the standard Proctor test (ASTM D-698) and modified compaction (ASTM D-1557). Generally, CQA requires that the water content and dry unit weight of the samples from the field-compacted soil be comparable to the laboratory compaction tests. Such tests and several others are standard requirements of CQA and CQC (Daniel, 1993).

It is important that after the compaction of each lift and after the completion of the entire liner, the compacted clay liner must be protected from desiccation or freezing due to high or low temperatures, respectively.

NRCS-Arizona (2010) provides additional information on compacted clay liners, such as criteria for limiting seepage, liner thickness, liner protection, side slope angles for stability, and the procedures for installation and basis of acceptance.

* Layer placement is also affected by the slope. For slopes 4H:1V or flatter, the compacted clay can be placed in layers parallel to the slope. For steeper slopes this is generally not possible, and the clay needs to be placed in horizontal layers progressing up the slope.

Compacted clay liners may be combined with an overlying geomembrane to form a composite liner, commonly used for lining waste ponds or landfills. Additional discussion on composite liners is provided in the following sections.

2.1.3.2 Dense Asphaltic Concrete

Dense asphaltic concrete or DAC (WALO, n.d.) has been used for lining reservoirs since the 1930s. DAC is a robust, impermeable, and non-toxic lining system composed of bitumen and a continuously graded aggregate. It is stable when placed hot and and uncompacted to enable reliable compaction on slopes up to 1V:1.6H (Wilson, 2017). DAC has a number of advantages: Its high density makes it resistant to chemical, biological, or mechanical attacks, and it can be thin enough to be flexible under pressure and strong enough to withstand the stress of varying volumes of water being pumped in and out every day. It has been used for new projects as well as for rehabilitation of existing structures. However, DAC requires an appropriate source of good sand and granular material for its manufacturing, and while it is a robust liner system, its success is dependent on many parameters and on good, well-aligned workmanship during its construction.

A short introduction to the technology of asphaltic linings for storage reservoirs can be found in Geiseler (1996). The different component layers of the standard design of asphaltic concrete are shown in Figure 2-3. Note that the figure on the right represents a double lining system and that this sandwich-like layered construction can be expensive. Some characteristics of DAC lining systems are listed below:

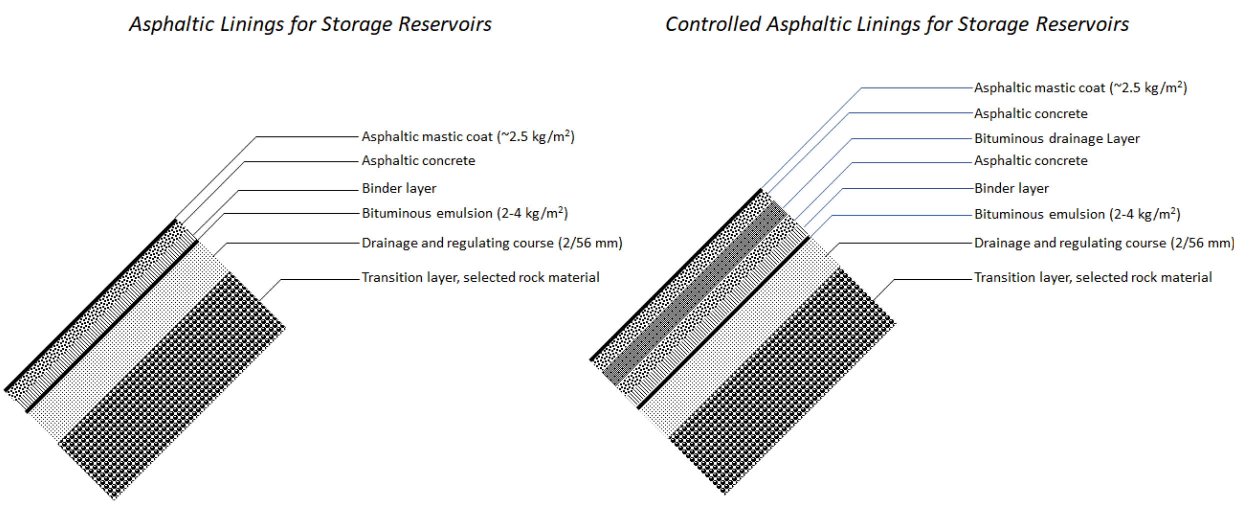


Figure 2-3 Standard Designs of Asphaltic Concrete Linings for Reservoirs (Geiseler, 1996).

- The substructure consists of crushed material having a thickness of at least 30 cm on the slopes and 20 cm at the bottom. The grain size should not be larger than 56 mm, and the material should contain some finer material.

- The non-bituminous drainage layer is typically sprayed with a cationic bituminous emulsion having a bitumen content of 60%.

- The binder layer is a mixture of crushed aggregates of different sizes and typically has a thickness of at least 7-8 cm. The grain size distribution is adjusted to have a void ratio of 10%-15%.

- The asphaltic concrete, which is the waterproofing layer, is a mixture of crushed aggregates having a maximum size of 11.2 mm and natural sand, mineral filler, and bitumen. The bitumen content must be adjusted to have sufficient coating on the grains while ensuring the stability of the slope. The proportion of round natural sand must be sufficient to ensure good workability of the mixture.

- A mastic seal coat is applied on the surface to protect the impervious asphaltic concrete layer from UV light, which causes embrittlement of the bitumen in conjunction with oxygen. During the service life of the reservoir, the mastic seal coat must be regularly checked and repaired and replaced as needed.

Examples of reservoirs constructed or refurbished using asphaltic concrete can be found in Strabag (1996).

2.1.3.3 Liner Systems Using Geosynthetics

Standard Terminology for Geosynthetics (ASTM, 2020) defines geosynthetics as planar products manufactured from polymeric material used with soil, rock, earth, or other geotechnical related material as an integral part of a human-made project, structure or system.

According to ISO 10318, a geomembrane is a "factory-assembled structure of geosynthetic materials in the form of a sheet in which the barrier function is essentially fulfilled by polymers" (ISO, 2015).

Most of the geosynthetic materials are made from synthetic polymers, which are broadly called plastics (Koerner, 2016a). The geosynthetic industry uses a few types of commercially available polymers:

- High-density polyethylene (HDPE), developed in 1941

- Linear low-density polyethylene (LLPDPE), developed in 1956

- Polypropylene (PP), developed in 1957

- Polyvinylchloride (PVC), developed in 1927

- Polyester (PET), developed in 1950

- Expanded polystyrene (EP), developed in 1950

- Chlorosulphanated polyethylene (CSPE), developed around 1965

- Thermoset polymers, such as ethylene propylene diene terpolymer (EPDM), developed in 1960

Mueller (2007) provides a short description of the history of the use of geomembranes as liners. His book notes that the use of HDPE geomembranes as liners began in the 1960s with minor projects. The first use of HDPE geomembranes in landfill liners was reported in 1977 by F.W. Knipschild. In 1985, the A. Gruber company in Linz, Austria (later AGRU Geomembrane), built a new manufacturing facility in its Waldneukirchen plant to produce the first 5 m wide PE liner system (AGRU, n.d.). This was followed by the construction of a similar facility by A. Schlütter in Kempen-Tönisberg to make Carbofol geomembrane. In the 1980s, the use of HDPE geomembranes expanded considerably. The developments in Germany and the USA influenced developments in other countries. In 1996, the First Germany/USA Geomembrane Workshop was held to discuss the state of the art and unresolved issues.

The first geosynthetics conference took place in 1977 in Paris. Since then, geosynthetics have become widely popular. They are extensively used in civil engineering and geotechnical engineering for various kinds of large area lining purposes: dams, dikes, reservoirs, all kinds of treatment basins (such as tailings ponds or leaching ponds in mineral and ore processing), landfill basal liners, landfill capping, sealing large areas for the containment and remediation of contaminated land, tunnel construction, canal construction, large-area contiguous liners in industrial plants, and road construction.

Mueller (2007) reports that as of 2007, 2 to 4 million square meters of HDPE geomembrane were being installed annually, and about a dozen major suppliers were competing for an international market of at least 100,000,000 m^2 annually. At the time, the price of installed geomembrane per square meter on the German market averaged €3 to €4 per millimeter thickness.

Geotextiles

ASTM 4439 defines a geotextile as a "permeable geosynthetic composed solely of textiles." Geotextiles can further be classified as follows:

- Woven geotextile, which can be further classified into three subtypes
  - Woven monofilament
  - Woven multifilament
  - Woven slit film
- Non-woven geotextile, which can be further classified as
  - Needle punched
  - Heat bonded
- Knitted geotextile

Geogrids

ASTM D4439 defines a geogrid as a geosynthetic formed by a regular network of integrally connected elements, with apertures greater than ¼ inches to allow interlocking with surrounding soil, rock, earth, and other materials to function primarily as reinforcement.

Geogrids are further classified as follows:

- Extruded geogrid
- Bonded geogrid
- Woven geogrid

Geonets

ASTM D4439 defines a geonet as a geosynthetic consisting of integrally connected parallel sets of ribs overlying similar sets at various angles for planar drainage of liquids and gasses.

Geomembranes

ASTM 4439 defines a geomembrane as a very low permeability synthetic membrane liner or barrier used with any geotechnical engineering related material so as to control fluid (or gas) migration in a man-made project, structure, or system. They are manufactured in several ways:

- Thin contiguous polymeric sheets
- Impregnation of geotextiles with asphalt, elastomer or polymer sprays
- Multilayered bitumen geocomposites

Since geomembranes have a very low permeability, ranging from 1×10^{-15} to 1×10^{-12} m/s or several times more impervious than a clay liner, they are primarily used to contain liquids or gases.

Geosynthetic Clay Liners

Geosynthetic clay liners (GCLs) are composed of sodium or calcium bentonite bonded to a layer or layers of geosynthetic, which can be a geotextile or geomembrane. The bentonite is sandwiched between two geotextile layers, and the assembly is bonded by adhesive or by needle punching or stitching (Buazza, 2002). Needle punched GCLs are manufactured by pushing fibers through the top geotextile, the bentonite, and the bottom geotextile layers, and they rely on entanglement and friction to hold the assembly together. Stitched GCLs are manufactured by sewing the assembly in parallel rows. With the advantage of being very thin, they are often used as a replacement for compacted clay layers.

Geofoam

ASTM 4439 defines geofoam as a block or planar rigid cellular foamed polymeric material used in geotechnical engineering applications.

Geocomposites

ASTM 4439 defines geocomposites as a product composed of two or more materials, at least one of which is a geosynthetic. Some of the common combinations are the following:

- Geotextile and geonet
- Geotextile and geogrid
- Geotextile and drainage pipes
- Geonet and erosion mat

Geo-others

The world of geosynthetics is continuously changing, and new products are becoming commercially available for use in civil engineering and geotechnical applications as well as in other fields. Geo-others is the catch-all name for a wide range of newly developed products that don't fit in other categories, such as polymeric anchors, wrapped floor cells, encapsulated soil cells, threaded soil masses, and others (Koerner, 2016a).

Geosynthetics can be categorized under six functions. Table 2-2 summarizes the primary functions of all geosynthetic types. The function of interest for reservoir liners is containment, which is only provided by three geosynthetic types: geomembrane (GM), geosynthetic clay liner (GCL), and geocomposite (GC).

Table 2-2 Primary Functions of Different Geosynthetic Types

Type of Geosynthetics	Primary Function of the Geosynthetics Type				
	Separation	Reinforcement	Filtration	Drainage	Containment
Geotextile (GT)	Yes	Yes	Yes	Yes	
Geogrid (GG)		Yes			
Geonet (GN)				Yes	
Geomembrane (GM)					Yes
Geosynthetic clay liner (GCL)					Yes
Geofoam (GF)	Yes				
Geocomposite (GC)	Yes	Yes	Yes	Yes	Yes

2.2 Types and Characteristics of Geomembranes

Geomembranes and other types of geotextiles are made of polymeric products derived from ethylene and its byproducts. Table 2-3 shows the chemical formula and Lewis structure of various polymeric products commonly used to manufacture geomembranes and other geotextiles.

The chemical composition and molecular structure of the polymeric products used in making a geomembrane are important, as they define the geomembrane's characteristics: its physical properties, mechanical properties, endurance properties, and service lifetime. They also define whether the material is thermoplastic or thermoset. Geomembranes are generally made of thermoplastic materials, because they can be softened by melting and heat-welded to form seams. Thermoset materials are rarely used as geomembranes because they can be seamed only by using chemicals or special adhesive tapes, and they are difficult to repair. Some other polymeric products may initially be thermoplastic but become thermoset with aging due to the vulcanization process. Different polymeric products also show different resistance to the UV degradation and oxidation that make them brittle. Geomembranes may swell by absorbing the liquids with which they come into contact, which can cause large wrinkles that lead to stress concentration and work against the peripheral weld between the loose and anchored liner. This may cause the welds to experience significant peel stresses (Peggs et al., 2004). Different additives added to the resins of geomembranes can make them more resistant to these processes.

The geomembrane sheets used to line a reservoir may have a smooth or textured surface. Textured geomembranes, called single-textured surface or double-textured surface geomembranes, have one or both sides roughened with asperities, respectively. The asperities added to roughen the surface help to increase the friction coefficient between the geomembrane and the underlying and/or overlying layers, which is especially important when the liner system is placed on slopes. In textured geomembranes, depending on the manufacturing method, the asperities may be randomly distributed in pattern and size. Some manufacturers also produce what they call “column point anti-skid geomembrane,” or “pillar geomembrane,” with uniformly distributed raised points on one side or both sides (Shandong, 2022).

Three methods are used to manufacture textured geomembranes (WasteAdvantage, 2014): co-extrusion (blown film), structuring or patterning (calendered extrusion), and impingement. Currently, only the first two methods are used in North America. Below, these three manufacturing methods are briefly described together with their advantages and disadvantages.

- The *co-extrusion (blown film)* method uses a single calendered extrusion during which a certain amount of nitrogen gas is injected into the molten polyethylene. As the extruded material leaves the die, the nitrogen gas expands to create a roughened surface with a random pattern. This process has several disadvantages:
    - The texturing produced by the expansion of the injected nitrogen gas is often acceptable, but its exact geometry cannot be controlled, and the properties from roll to roll may be different. The variations of core thickness and texture across and along the geomembrane require a greater number of tests to be performed to account for the uncertainty.

Table 2-3 Polymeric Products Derived from Ethylene and Its Byproducts (adapted from Figure 1.2 in Koerner, 2016, Vol. 1). In the last column final products used in the manufacturing of geosynthetics are highlighted. Chemical formula and Lewis structure are provided unless the name refers to a group of multiple products.

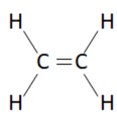
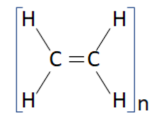
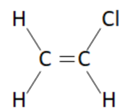
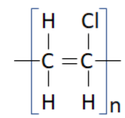
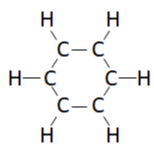
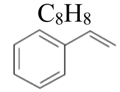
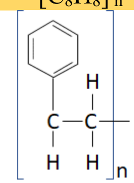
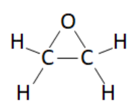
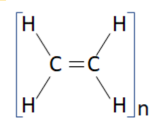
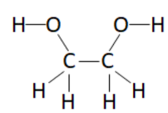
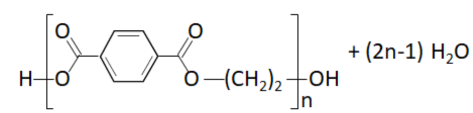
Initial Product	Added Chemical	Intermediate Product	Process	Final Product
Ethylene C2H4 or H2C=CH2 			Polymerization	Polyethylene (PE) and copolymers $[H_2C{=}CH_2]_n$ 
	+ Chloride Cl^-	Vinyl chloride $H_2C{=}CHCl$ 	Polymerization	Polyvinyl chloride (PVC) $[H_2C{=}CHCl]_n$ 
	+ Benzene (alkylation) C_6H_6 	Styrene C_8H_8 	Polymerization	Polystyrene (PS) $[C_8H_8]_n$ 
	+ Oxygen (oxidation) O	Ethylene oxide C_2H_4O 	Polymerization	Polyethylene (PE) $[H_2C{=}CH_2]_n$ 
		Ethylene glycol $C_2H_6O_2$ 	Polymerization	Polyester or Polyethylene terephthalate (PET) 

Table 2-3 (Cont.)

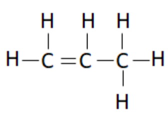
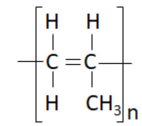
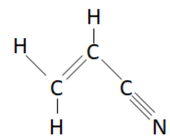
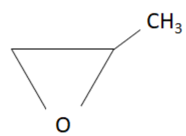
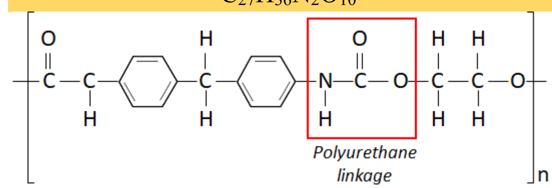
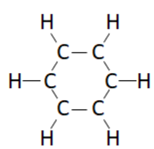
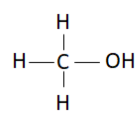
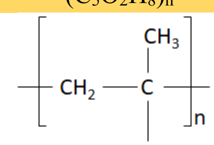
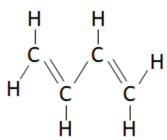
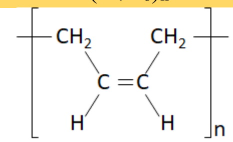
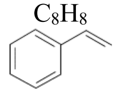
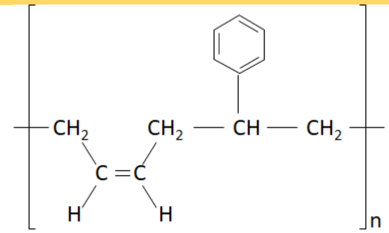
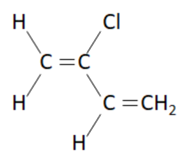
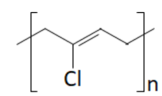
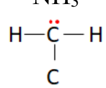
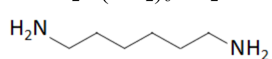
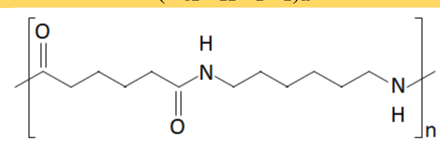
Initial Product	Added Chemical	Intermediate Product	Process	Final Product
Byproduct Propylene C_3H_6 			Polymerization	Polypropylene $(C_3H_6)_n$ 
	Ammonia NH_3	Acrylonitrile C_3H_3N 	Polymerization	Acrylic fiber, plastic, rubber [Multiple products]
	Oxygen (oxidation) O	Propylene oxide C_3H_6O or CH_3CHCH_2O 	Polymerization	Urethane (polyurethane) foams $C_{27}H_{36}N_2O_{10}$ 
	+ Benzene (alkylation) C_6H_6 	Cumene then phenol and acetone	Polymerization	Phenolic resins (or phenol formaldehyde resins) [Multiple products]
		↓ + Methanol CH_3OH 		
		Methacrylates [Multiple products]	Polymerization	Polymethylmethacrylate $(C_5O_2H_8)_n$ 

Table 2-3 (Cont.)

Initial Product	Added Chemical	Intermediate Product	Process	Final Product
Byproduct butadiene 			Polymerization	Polybutadiene $(C_4H_6)_n$ 
	+ Styrene C_8H_8 		Polymerization	Styrene butadiene rubber (SBR) $C_{12}H_{24}$ 
	+ Chloride Cl^-	Chloroprene C_4H_5Cl or $CH_2{=}CClCH{=}CH_2$ 	Polymerization	Neoprene rubber 
	Ammonia NH_3 	Hexamethylene diamine $H_2N(CH_2)_6NH_2$ 	Polymerization	Nylon 6,6 or polyamide (PA) $(C_{12}H_{22}N_2O_2)_n$ 

- The height of the asperities may be increased by increasing the amount of nitrogen injected into the mixture. This may even result in improved shear resistance in some cases, but it requires heavier and larger diameter rollers to work with the increased amount of polyethylene that must be used, and the increased nitrogen may have the adverse effect of causing variations in the core thickness.

- The asperities created by this process produce a VELCRO®-like surface, requiring a slip sheet in between it and nonwoven geotextiles or GCLs during installation in order to avoid snagging.

- *Structuring or patterning* (calendered extrusion) is a more recent manufacturing method in which the geomembrane extruded by a flat die is immediately passed through patterned rollers while it is still hot. Thus, this method of manufacturing uses a two-step extrusion process: a calendered extrusion using smooth rollers followed by a passage through patterning rollers. The asperities are imprinted as the inverse of the pattern embossed into the second rollers. The structuring or patterning method removes the uncertainties and randomness of the co-extruded process and offers several important advantages:

    - Since the geomembrane is calendered, the core membrane has a uniform thickness over the entire sheet and the uniformity of the texture (asperities and the pattern) is completely controlled both along the width and the entire length of the geomembrane roll.

    - This manufacturing method ensures a uniform and consistent pattern quality not only within the roll but also from one roll to another. This removes the uncertainties during the various tests for a project, such as the thickness measurement and direct shear testing, and reduces the required number of tests. The engineer can be assured that the test sample represents all the rolls manufactured with the same pattern.

    - It is possible to leave the edges of the patterned roller smooth to produce smooth edges on the geomembrane to facilitate the welding process.

    - Different patterned cylinders may be used on different sides of the geomembrane to control the desired asperities based on the requirements of different applications.

- *Impingement* is the process of manufacturing textured geomembranes by spraying molten polyethylene onto a smooth geomembrane in a secondary process that follows the manufacture of the smooth geomembrane. During spraying, the edges of the extruded sheet can be protected to keep them smooth to facilitate welding. In the U.S., the impingement method was used in the 1990s but was later abandoned due to the higher cost of the secondary process. It is still being used in Europe.

2.2.1 Properties of Geomembranes and Measuring Techniques

Measurement and testing of properties are important and serve multiple purposes:

- Testing provides information on the geomembrane's properties for use in both the design process and the selection of the most technically and economically appropriate geomembrane for the specific purpose at hand. These properties are crucial in predicting the behavior of the geomembrane in the various conditions and with the types of loads it will experience during its service life.

- Testing confirms that the geomembrane selected for a specific task meets certain quality standards in manufacturing, handling and transport to the job site, placement on the job site, and service life.

The list of tests for geomembranes taken from Mueller (2007) is provided in Table 2-4.

Table 2-4 Tests for HDPE Geomembranes (from Mueller, 2007).

Property Measurement (Test Procedure)	Performance Test	Quality Assurance Test	Identification Test	Durability Test
Surface condition, appearance		Y		
Homogeneity of cross section		Y		Y
Carbon black content	Y	Y		Y
Carbon black dispersion	Y	Y		
Skew	Y	Y		
Waviness	Y	Y		
Thickness	Y	Y		
Density		Y	Y	
Melt flow rate (MFR) and change in MFR	Y	Y	Y	
Dimensional stability	Y			
Permeability to hydrocarbons				
Melting enthalpy and point			Y	
Oxidation stability (OIT)			Y	
Tensile properties		Y	Y	
Behavior under planar deformation (burst test)	Y			
Tear resistance	Y			
Resistance to static puncture	Y			
Resistance to dynamic puncture	Y			
Flexibility in low temperatures	Y			
Relaxation behavior	Y			
Seam quality (peel, tensile test)	Y	Y		
Resistance to chemicals	Y			Y

Table 2-4 (Cont.)

Property Measurement (Test Procedure)	Performance Test	Quality Assurance Test	Identification Test	Durability Test
Stress crack resistance	Y			Y
Resistance to thermo-oxidative degradation	Y			Y
Long-term behavior under combined stressing	Y			Y
Weathering resistance	Y			Y
Microbe resistance	Y			Y
Root penetration resistance	Y			Y
Resistance to rodent damage	Y			Y
Stress crack resistance of textured	Y			Y
Geomembranes (long-term tensile test)	Y			Y
Adhesion of texture particles (long-term shear strength test)	Y			Y
Friction (shear box test)	Y			

Understanding the tests listed in Table 2-4 and how to use the information they provide is extremely important. Using the results of these tests without fully understanding the limits of the information they provide, or their correct interpretation, may lead to errors. Mueller (2007) insists on this issue and gives a few examples of pitfalls when interpreting and using the results of these standard tests.

One example is interpreting the results of the multiaxial tension test (burst test), which provides information about deformation due to a planar stress state and is measured as an important property of a geomembrane. Mueller points out that the strain at the time of the yield, i.e., at the time the geomembrane bursts, should not be interpreted as the permissible strain limit for the geomembrane. This is because the long-term behavior of the geomembrane is defined by its stress crack resistance, which is measured by another test (see Table 2-4). Another is the strain at the breaking point of textured geomembranes, which may be affected by the stress concentration due to surface texturing. Thus, Mueller (2007) points out, for textured geomembranes, the strain at break test cannot be used for comparing different types of geomembranes. The results of the test should only be used for quality control purposes.

Tests for physical, mechanical and other properties of geomembranes may be conducted during different phases of the design and construction of a geomembrane liner (Swan, 2019): 1) during the design phase for site specific testing, 2) during pre-construction material selection/qualification, 3) during the manufacturing of the geomembrane for manufacturing quality control and quality assurance, 4) during construction for construction quality control and quality assurance, and 5) for forensic analysis after a failure. These tests are usually requested by 1) design engineers during the design phase, 2) contractors and installers during the construction phase, 3) manufacturers during research and development for product development and manufacturing, 4) owners and their representatives at all phases, 5) regulators, and 6) legal representatives following a failure.

Reviewing all the tests available for geomembranes would be too lengthy and is outside the scope of the present document. In the following sections, only selected standard tests will be briefly described and discussed.

2.2.2 Physical Properties of Geomembranes

Physical properties of geomembranes—the characteristics of the material resulting from its manufacturing and as received at the job site—are of paramount importance for design considerations and selection of the most suitable material from both the technical and the economical points of view. Figure 2-4 shows a selection of the physical properties of geomembranes required for their designed use in civil engineering applications as pond or reservoir liners.

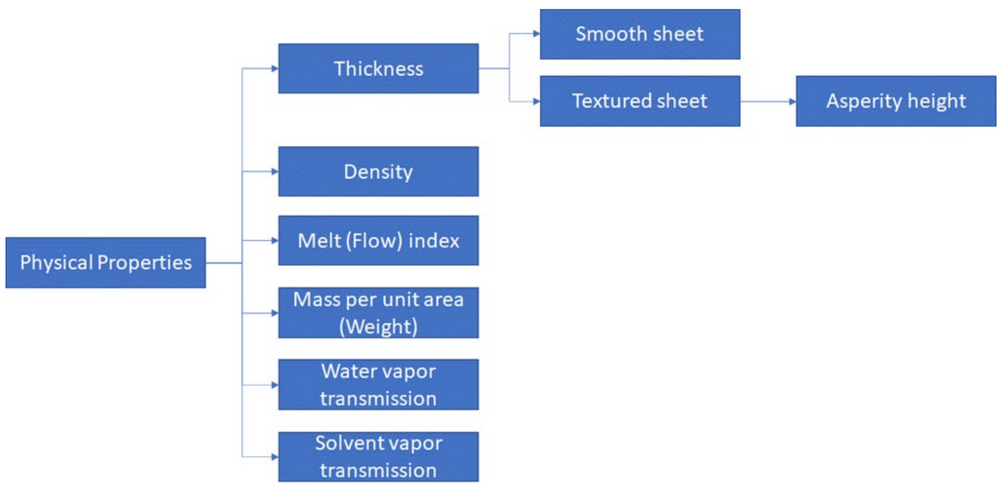


Figure 2-4 Relevant Physical Properties of Geomembranes for Pond or Reservoir Liners.

2.2.2.1 Thickness

Geomembranes are thin sheets of flexible thermoplastic polymeric materials that may have a smooth or textured surface. The definition of the thickness and the method of its measurement depends on the surface characteristics of the geomembrane.

A smooth or non-textured geomembrane ideally has a constant thickness, leaving aside small deviations due the manufacturing process, and a single value can be used to define the thickness of the sheet. Textured geomembranes do not have a constant thickness due to the asperities created on one or both sides to increase the friction between the geomembrane and the adjacent layer, so the thickness cannot be defined by a single value. The core thickness and the heights of the asperities must be measured separately.

A deadweight micrometer with a spring-loaded presser foot or point (see Figure 2-5) is used to measure the thickness of smooth geomembranes and the core thickness of textured geomembranes, respectively.

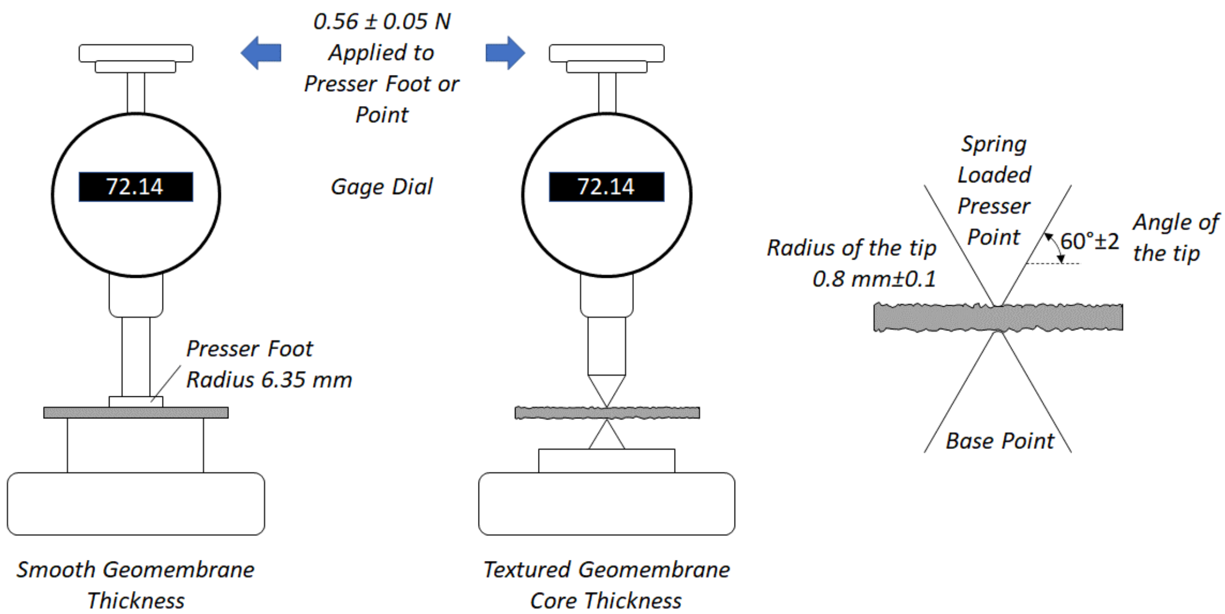


Figure 2-5 Deadweight Micrometers with a Spring-Loaded Presser Foot (Left) and Point (Middle) Used for the Measurement of Thickness of Smooth and Textured Sheet Geomembranes, Respectively (ASTM D5994 Thickness Textured Membranes video).The illustration on the right shows the detail of the conical point.

Measuring the thickness of a smooth geomembrane is described by ASTM 5199, GRI GM13, and ISO 09863 and uses a deadweight micrometer with a circular presser foot (see Figure 2-5 left). First the micrometer is zeroed by bringing the presser foot onto the base plate, then the smooth geomembrane is placed flat on the base plate and the presser foot is brought in contact with the sample to read its thickness. For each geomembrane roll, 10 samples across its width are measured, and the average of these readings is taken as the thickness of the roll. The measured thickness should be equal to the nominal thickness, and the difference between the lowest individual reading and the nominal thickness should be less than or equal to 10%.

Measuring the core thickness of a textured geomembrane is described by ASTM 5994: The deadweight micrometer with a conical presser point is used (see Figure 2-5 middle and right illustrations), with a second conical tip on the base plate located exactly under the presser point. First the micrometer is zeroed by bringing the presser tip to the tip attached to the base plate, then the textured geomembrane is placed onto the bottom tip and the presser tip is brought in contact with the sample to read its core thickness. For each geomembrane sample, three readings of the core thickness are made by placing the points in the “valleys” between the asperities. The smallest reading is recorded as the core thickness for the sample. For each geomembrane roll, 10 samples across the roll width are measured, and the average of the minimum readings from these ten samples is considered the thickness of the roll. The difference between the measured roll thickness and the nominal thickness should not be more than 5%, and the difference between the lowest individual reading and the nominal thickness should be less than or equal to 15%.*

For textured geomembranes, the height of the asperities must be measured as well as the core thickness. The height of the asperities can be measured using a micrometer as described in ASTM D7466 (see Figure 2-6). The pointed stylus of the micrometer has a diameter of 1.3 mm. First, the gauge is zeroed by placing the setting block of the micrometer on a base plate (a smooth, flat, hard surface), then the geomembrane is placed on the base plate and the micrometer is placed on the geomembrane with the stylus positioned in a “valley” between the asperities to measure the asperity height. For double-sided textured geomembranes, the procedure is repeated with the other face up. It is recommended to measure every second roll in 10 locations across the roll width. At least three measurements are made at a single location, and the maximum value is recorded. The minimum average height of an asperity should be more than or equal to 0.254 mm (10 mil).†

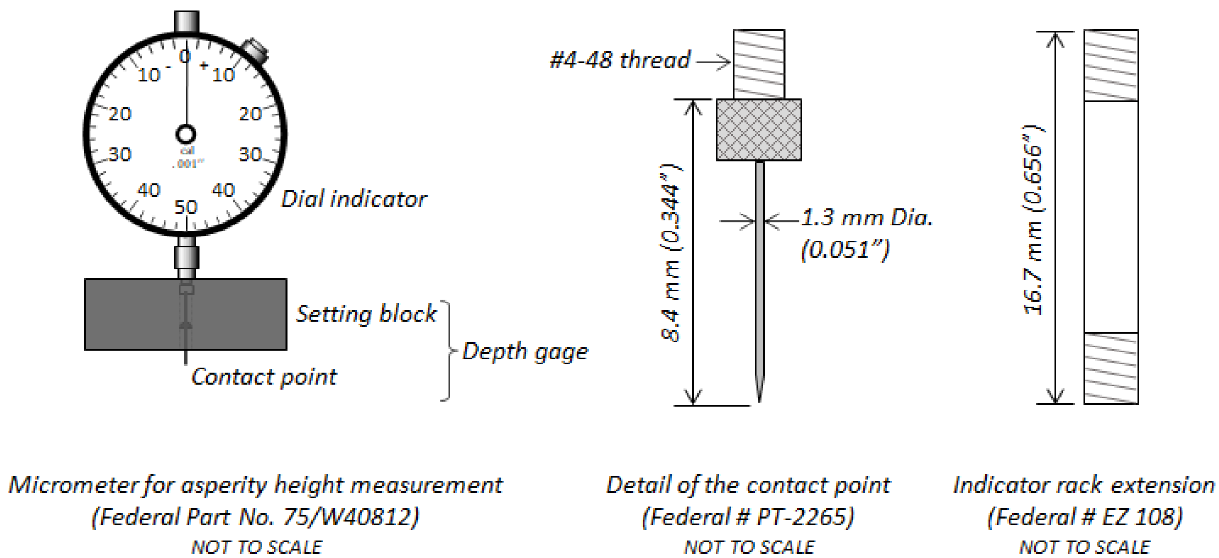


Figure 2-6 Micrometer to Measure the Asperity Height of Textured Geomembranes (Koerner, 2016c).

Koerner (2016) also mentions the methods for measuring the height of the asperities using profilometry (Dove and Frost, 1996) and three-dimensional topography (Ramsey and Youngblood, 2009a and b) as shown in Figure 2-7. Yesiller (2005) discusses the issues arising when measuring core thickness and asperity height using a micrometer.

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* A video tutorial for measuring the core thickness of textured geomembranes is available from the Geosynthetics Institute (GSI) at https://www.youtube.com/watch?v=Bs4fBtE6EOs.

† A video tutorial for measuring the asperity height of textured geomembranes is available from GSI at https://www.youtube.com/watch?v=IVB7uDgu7zA.

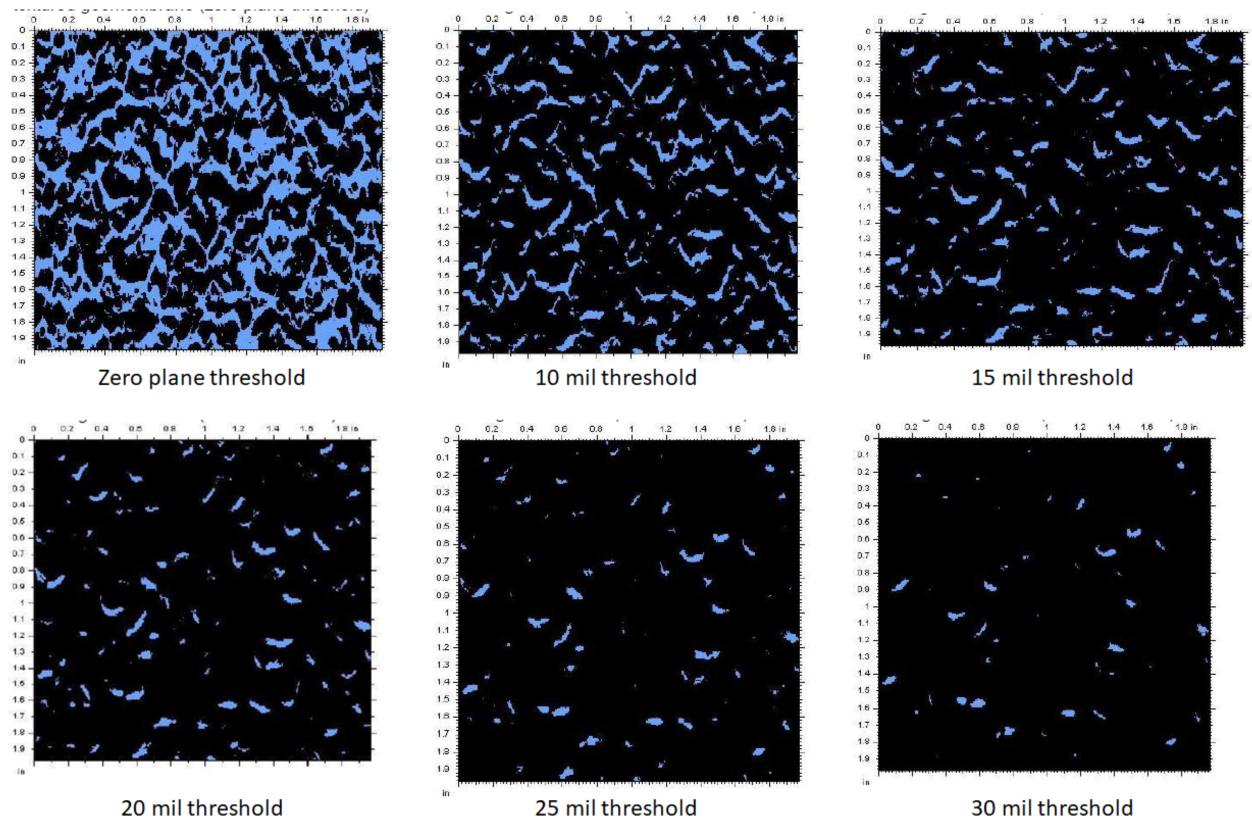

Figure 2-7 3D Topological Characterization of a Textured Geomembrane Shows the Contours of the Asperities of a Textured Geomembrane at Different Threshold Heights (Ramsey and Youngblood, 2009b). The sample size is 2" × 2".

2.2.2.2 Density

The density of the geomembrane depends on the chemicals from which it is made—HDPE, LLPDPE, PP, etc. The densities of all geomembranes fall within the range 0.85 to 1.5 g/cc. The standard test methods for measuring geomembrane density are defined by ASTM D792 and ISO R1183. Both methods are based on the Archimedes' principle: A body immersed in a fluid experiences an upthrust equal to the weight of the fluid displaced. The density is therefore obtained by dividing the weight of the geomembrane in air by its weight in water.

ASTM D1505 defines a more accurate but more tedious method of measurement of the density of a plastic by the density gradient technique. In this method, the density is measured by observing the level to which the test specimen sinks in a liquid column with a known vertical density gradient. This method may achieve an accuracy of 0.002 g/cc if properly executed.

ASTM requires HDPE geomembranes to have a density of at least 0.941 g/cc. Commercially available HDPE geomembranes are made from polyethylene resin with a density range of 0.934 to 0.938 g/cc called medium density polyethylene (MDPE). In order to achieve a higher density in line with the ASTM standards for HDPE, carbon black and some antioxidant chemicals are used during manufacturing, so HDPE is in fact MDPE with some additives. The formulated density of HDPE is then calculated using the following formula.

$$\rho_f = \rho_r + 0.0044\,(CB + AO) \tag{1}$$

in which ρ_f (g/cc) is the formulated density, ρ_r (g/cc) is the resin density, CB (%) is the percentage carbon black in the mixture, and AO (%) is the percentage of antioxidants.

The standard test method for determining the percentage of carbon black content in polyethylene compounds using the "muffle-furnace technique" is described in ASTM 4218.

Classification of PE resins according to the density range is as follows: HDPE (0.941-0.965 g/cc), MDPE (0.926-0.940 g/cc), LLDPE (0.915-0.925 g/cc), LDPE (0.910-0.915 g/cc), and VLDPE (0.880-0.910 g/cc). Increasing the density of the resins used to manufacture a PE geomembrane by means of additives has important consequences on its various properties.[*] Increasing the density of a PE geomembrane increases its crystallinity, tensile strength (at yield), stiffness, chemical resistance and abrasion resistance while decreasing its impact strength, stress crack resistance, permeability, and processability.

2.2.2.3 Melt (Flow) Index

The melt flow index or melt index (MI) reflects the ability of a polymer to flow in its molten state. To control the uniformity and processability of the polymer, manufacturers measure MI of both the original polymer and the polymer with additives used for manufacturing the geomembrane. MI index depends on the chemical properties of the polymer resin and its density, which is modified by additives. The standard test for measuring MI is described by ASTM D1238. The polymer is brought to the molten state by heating in a furnace, and then it is extruded under constant force (weight) through a hole at the bottom of the test apparatus. The weight of the molten polymer extruded in 10 minutes gives the MI value. All other things being the same, a higher MI value indicates a lower polymer density and a lower molecular weight.

The test sometimes produces two different weights: $W_1 < W_2$. Assuming that the MI values obtained using these two weights are MI_{W_1} and MI_{W_2}, respectively, the flow rate ratio (FRR) is calculated as follows:

$$FRR = \frac{MI_{W_2}}{MI_{W_1}} \tag{2}$$

A higher FRR value is interpreted as a wider molecular weight distribution. The MI and FRR are important parameters for quality control and quality assurance of polyethylene resin batches and manufactured geomembranes.

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[*] https://www.linkedin.com/pulse/properties-polyethylenes-geomembranes-shahab-jafarzadeh

2.2.2.4 Mass Per Unit Area (Weight)

The standard method for the measuring the mass per unit area (weight) of a geomembrane is given by ASTM D1910. The mass per unit area is measured in units of g/m^2 and is obtained by weighing a sample with a known surface area.

2.2.2.5 Water Vapor Transmission

Although polymeric geomembranes are manufactured to provide an impervious barrier, they still allow a measurable, albeit very small, amount of diffusion, like any other material. Different measurement methods can be used. The European standard NF-EN 14150 measures the diffusion of water through a 200 mm diameter sample subjected to 100 kPa differential water pressure. This is a difficult procedure, as the amount of water passing through the tested sample is very small even over very long times. The ASTM E96 procedure is easier: It monitors the weight loss of water in an aluminum cup sealed by the geomembrane specimen being tested. The relative humidity difference between the cup (100%) and the outside air humidity is kept constant, and weight loss over time is monitored. An alternative method of the same test is performed by placing a desiccant into a dry aluminum cup sealed by the geomembrane. The outside humidity is kept constant, and the weight of the cup is monitored over time. The increase in weight gives a measure of the water vapor transmission. The time required for the test varies from 3 to 40 days. Details of this method can be found in Koerner (2016b).

2.2.2.6 Solvent Vapor Transmission

Solvent vapor transmission is an important parameter when the geomembrane is used for reservoirs containing chemicals other than water. The test is similar to ASTM E96, but the cup is filled with the solvent, and the transmission of its vapors through the geomembrane is measured. A more detailed discussion can be found in Koerner (2016b).

2.2.3 Mechanical Properties of Geomembranes

Mechanical properties of geomembranes generally focus on the resistance of the geomembrane to tensile stresses. Standards tests have been devised to measure these mechanical properties of geomembranes in the laboratory under controlled conditions. Figure 2-8 summarizes the tests relevant to the mechanical properties of geomembranes.

It is important to note that the standard tests to measure the mechanical properties of geomembranes can be categorized in two groups:

- *Index tests* or *in-isolation tests* use a standard sample of the geomembrane alone to measure its mechanical properties: These tests are solely on the mechanical properties of the geomembrane in isolation from underlying or overlying material.

- *Performance tests* are designed to test the geomembrane together with the soil layers to be used below and/or above it. These tests try to imitate the field conditions where the geomembrane is subject to tensile forces together with the underlying and/or overlying geomaterials (soil, gravel, etc.). The adjacent soil layers used in the tests can be standard soils or a soil sample taken from the site.

This section briefly summarized the different types of tests that measure the mechanical properties of geomembranes. It is important to note that the tests described in this section test the geomembrane samples for different phenomena individually. The geomembrane placed in the field, however, is subject to the simultaneous effects of multiple phenomena. Therefore, engineering judgment must be exercised when using these test results, considering that the test results may also include scale effects.

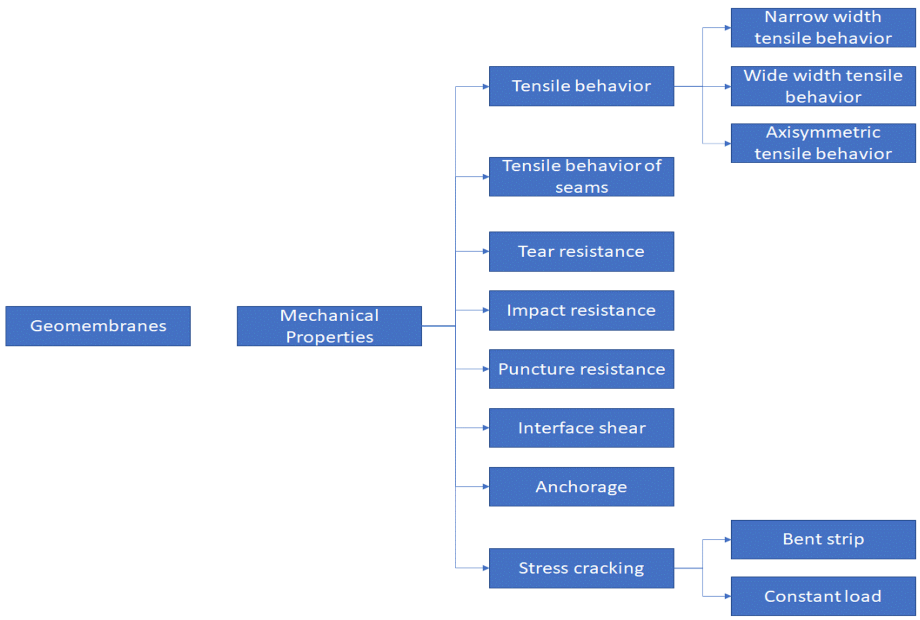

Figure 2-8 Testing Relevant Mechanical Properties of Geomembranes.

2.2.3.1 Tensile Behavior of Geomembranes

Tests of tensile behavior are so-called index or in-isolation type tests. They are mostly used for quality control of the manufactured geomembrane. The test is performed using a uniaxial tension machine, and a narrow strip of geomembrane is used as a sample. The sample of geomembrane placed between two clamps is slowly extended until it breaks. The machine measures the elongation of the sample and the force used to extend it.

The total length of the sample from end to end is L_o. The sample is generally wider at each end for easier clamping. The narrow section of the sample between the wider ends has a length L_G, which is called the gage length. If at a given time the elongated length of the tested gage section is L, and the force applied to it is F_n, the strain is computed as $\varepsilon = \Delta L/L_G$, where $\Delta L = L - L_G$, and the corresponding stress is computed as $\sigma = F_n/(w \times t)$, where w is the width of the strip and the t is the thickness of the geomembrane.

The machine records the strain and the corresponding tensile force until the sample breaks. The plot of stress versus strain for the test provides important properties: 1) maximum stress and strain, 2) ultimate stress and strain at failure, and 3) Young's modulus of elasticity. Maximum stress is the ultimate stress for PVC and LLDPE, yield stress for HDPE, and stress at scrim break for CSPE-R. The characteristics of the sample and the test method depend on the type of geomembrane, as shown in Table 2-5 (Koerner, 2016b). Note that since the original width and thickness are used in calculating the stress, the results do not reflect the true stress.

Table 2-5 Uniaxial Tensile Test Methods for Different Types of Geomembranes (adapted from Koerner, 2016b).

Test and Details	HDPE	LLDPE, fPP	PVC	fPP, EPDM-R, CSPE-R, EIA-R
ASTM test method	D6693	D6693	D882	D751
Sample shape	Dog bone	Dog bone	Strip	Grab
Sample width, w (mm)	6.3	6.3	25	100 (25 grab)
Sample length, L_0 (mm)	115	115	150	150
Gage length, L_G (mm)	33	33	50	75
Strain rate (mm/min)	50	500	500	300
Strength	F_n	$\sigma = F_n/(wt)$	$\sigma = F_n/(wt)$	F_n
Strain (mm/mm)	$\Delta L/L_G$	$\Delta L/L_G$	$\Delta L/L_G$	$\Delta L/L_G$
Young's modulus	Slope of the graph	Slope of the graph	Slope of the graph	Slope of the graph

Figure 2-9 compares the typical stress versus strain curves for different geomembrane types using narrow and wide samples. The narrow sample tests are based on the methods described in Table 2-5, and the wide sample tests are based on ASTM D4885.

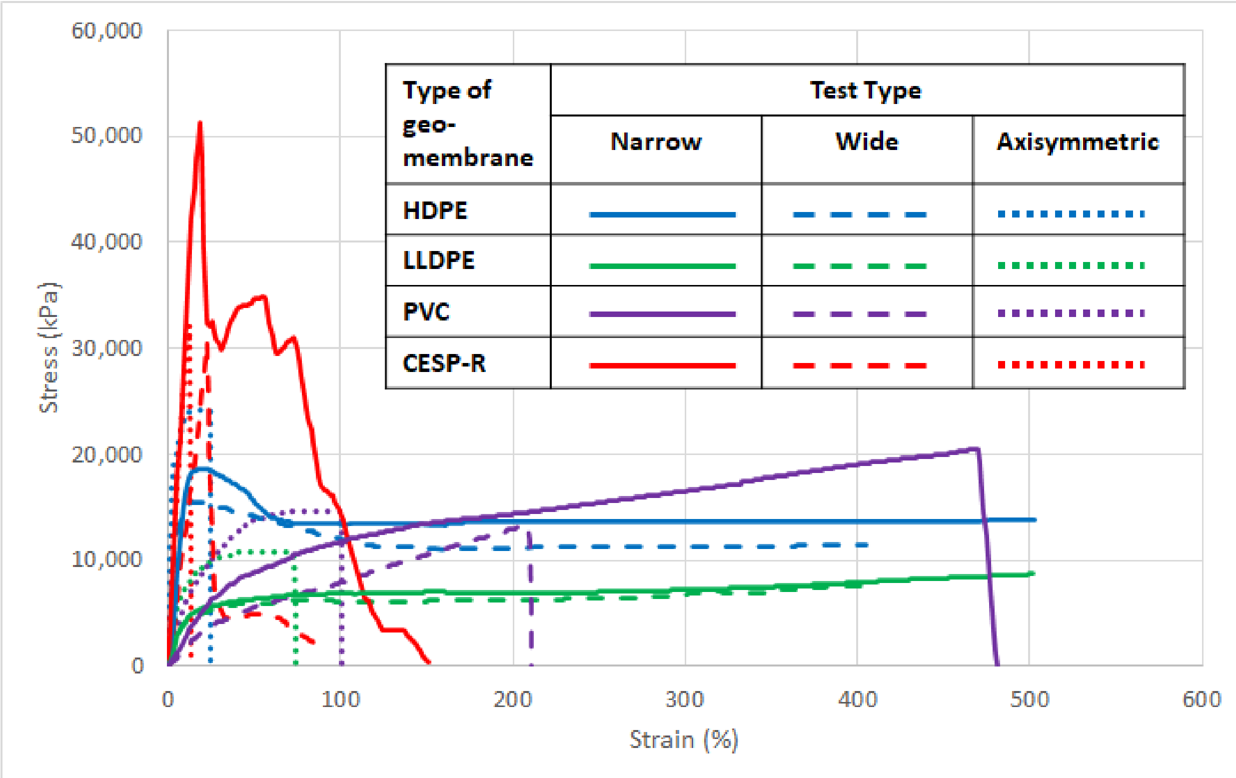

Figure 2-9 Comparison of Typical Stress-Strain Curves Obtained for Different Types of Geomembranes Using Narrow and Wide Samples in Uniaxial Testing (In-Plane Tensile Forces), and in Axisymmetric Testing (Out-of-Plane Tensile Forces) (Koerner, 2016b).

The narrow width of the sample is an important shortcoming of the index test described above. As the sample is elongated, i.e., the strain increases, the width of the sample becomes significantly smaller. To address this weakness, tests with wider samples have been developed. ASTM D4885 describes the method for a test with a width of 200 mm. These wide sample tests, however, require several hours to complete at the recommended strain rate of 1.0 mm/min, so they are considered performance tests.

The tests described above are uniaxial: They test the strength of the geomembrane against tensile forces in the plane of the geomembrane. However, geomembranes may also be subjected to out-of-plane tensile forces due to deformation of the subgrade or due to the pressure of the gases accumulated underneath it. Axisymmetric tensile behavior tests have been developed for testing the tensile strength of geomembranes when subjected to out-of-plane forces. The axisymmetric tensile strength test is described in ASTM D5716. In this test, a circular sample of the geomembrane is placed over a large circular container. A cover is placed on the container, over the sample, and tightly sealed. Initially the sample is like a flat membrane between the container and the cover, but as water or air is gradually introduced from the top of the cover, the geomembrane begins to take the shape of an inverted dome. The pressure is increased until the geomembrane dome bursts. The testing apparatus records the deflection of the center point of the bulging geomembrane and the corresponding pressure from the beginning of the test to the failure at the end. The stress and strain are calculated using this data. The details of the calculations and the equations can be found in Koerner (2016b).

2.2.3.2 Tensile Behavior of Seams

When geomembrane is installed in the field, adjacent rolls are overlapped and welded together to form a continuous barrier material. Various methods are used to seam the geomembranes: Wedge welding, extrusion welding, and chemical bonding are the most common. The type of seam depends on the type of the geomembrane (see Section 2.3.2.1 for more on seams).

Several standard methods have been developed to test the strength of geomembrane seams:

- *Shear tests* consist of pulling apart a representative section of the seam joining two geomembranes using a uniaxial tensile test machine. ASTM D6392, ASTM D882, and ASTM D751 describe these types of shear tests. The shear test is generally assumed to be a performance test.

- In a *peel test*, the representative section of the seam joining two geomembranes is pulled apart using a testing device. The device grips the end of one of the geomembranes and the overlapping part of the other geomembrane and pulls them apart. ASTM D6392, ASTM D882, and ASTM D413 describe peel tests for seams. The peel test is generally assumed to be an index test.

In both cases, the sample is 250 mm wide. The tests clearly show that the seams are weaker than the original material. The peel test in particular yields significantly lower strength values.

2.2.3.3 Tear Resistance

Different standard tests are available for testing the tear resistance of geomembranes. All the tests are performed using a universal test machine:

- ASTM D1004 method uses a sample die-cut to a standard shape and dimensions with a 90° angle in the center (see Figure 2-10). The sample is pulled by the grips of a uniaxial tensile test machine. The tear begins at the tip of the 90° wedge and propagates until the sample breaks.

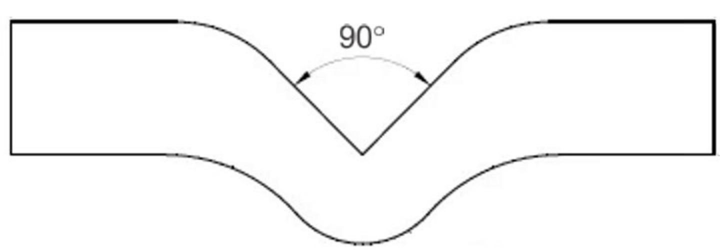

Figure 2-10 Tear Resistance Test Sample.

- ASTM D 2263 uses a trapezoidal-shape sample. A small cut is made in the middle of the long edge of the trapezoidal sample. The long side of the sample is placed between the grips of a uniaxial tensile machine and the shorter side remains slack. The tear begins at the small cut and propagates to the other side until failure.

- ASTM D624 and ISO 34 measure tear resistance using three different sample shapes (trouser-shaped sample, angle test sample, and crescent shape with a nick) by pulling them in a dual or single column universal testing machine.

Several other tests use different techniques to measure tear resistance:

- ASTM D1424 describes a method of determining the tear resistance of geomembranes using an Elmendorf-type falling pendulum apparatus.

- ASTM D5884 is used for geomembranes with an internal textile reinforcement and uses a tongue tear method to determine the tear resistance. This test is recommended for testing scrim reinforced geomembranes.

- ASTM D751 describes a series of tests, including a tear test, for measuring the tear resistance of coated fabrics with one layer of geomembrane and at least one layer of fabric.

The tear resistance of thin geomembranes is quite low (18 to 130 N). This emphasizes the importance of extreme care in their handling. Thicker geomembranes have a higher tear resistance.

2.2.3.4 Impact Resistance

After it is placed in the field, the geomembrane is subject to potential damage due to falling objects or during the placement of the cover soil. A number of tests have been developed to measure resistance of geomembranes to the impact from objects. ASTM D1709 and ISO 13433 use a free-falling "dart,"[*] ASTM D3029 uses a falling weight,[†] and ASTM D1822, ASTM D746, and ASTM D3998 are pendulum-type tests.[‡] The Spencer impact tester described by ASTM D3420[§] is a special type of pendulum test that is commonly used. It is performed by using an Elmendorf tear tester (ASTM D1424) with a Spencer impact attachment piece. Detailed information about these methods and the typical values obtained for different types of geomembranes can be found in Koerner (2016b).

2.2.3.5 Puncture Resistance

Stones and other angular or pointed debris that remain on the surface of the subgrade layer or within backfill materials create the potential for geomembrane punctures. Puncture tests determine the index value of the puncture resistance of geomembranes. The standard procedure described by ASTM D4883 can be performed by a universal testing machine for

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[*] See a video of the dart impact test at https://www.youtube.com/watch?v=fnCXop5QwRk or https://www.youtube.com/watch?v=gOoItehUeck.

[†] See a video of the falling weight impact tester at https://www.youtube.com/watch?v=0UTBntfPaSE.

[‡] See a video of ASTM D-1822 impact tensile test at https://www.youtube.com/watch?v=wIYH7SJmH9k.

[§] See a video of the Spencer impact tester at https://www.youtube.com/watch?v=v6NhmDxzdf8.

tensile/compressive testing. The sample of the geomembrane is clamped between two metal pieces with a 45 mm hole at the center. The machine pushes an 8 mm diameter rod with a flat bottom but beveled rim onto the center of the sample at a constant rate of 300 mm/min until it is punctured. The force at the puncture is recorded as the puncture resistance of the geomembrane. Puncture resistance for thin geomembranes ranges from 50 to 500 N, and for thin reinforced geomembranes it ranges from 200 to 2,000 N. The tests clearly show that for the same geomembrane thickness, placing a geotextile on one or both sides significantly increases the puncture resistance.

2.2.3.6 Interface Shear

Interface shear measures the frictional resistance between the geomembrane and the adjacent soil layer and is an important property for designing liner systems with geomembranes. The tests are performed using a direct shear box, schematically illustrated in Figure 2-11. The sample to be tested is bonded to a wooden block, which is placed in the upper half of the direct shear box. The soil specimen, compacted to a specified density and moisture content, is placed in the lower half of the direct shear box. ASTM D5321 recommends a box size of 300 mm × 300 mm for all types of soils unless a smaller box can be justified. Koerner (2016b) considers that for tests with fine grained soils such as sands, silts or clays, a smaller box of 100×100 mm can be used.

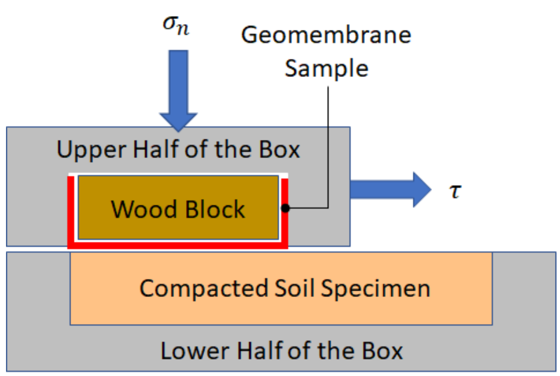

Figure 2-11 Schematic Illustration of the Direct Shear Box for Measurement of Interface Shear Stress (Koerner, 2016b).

In this test, after the specimens are prepared, as shown in Figure 2-11, a constant normal stress σ_n is applied to the top of the upper half. The upper half is then pulled or pushed over the lower half by the apparatus, which measures and records the applied shear stress and the displacement of the bonded geomembrane. The test is repeated three times with new samples using different constant normal stresses. The resulting curves of shear stress versus displacement are then used to plot the Mohr-Coulomb failure envelope. The intercept and slope of the straight line fitted to the Mohr-Coulomb failure line provides the required interface shear parameters, as follows:

$$\tau = c_a + \sigma_n \tan \delta \qquad (3)$$

where τ is the shear strength of the geomembrane against the opposing soil, σ_n is the applied normal stress, c_a is the adhesion or apparent cohesion between the geomembrane and the soil, and δ is the friction angle between the geomembrane and the soil. In order to estimate efficiencies, a direct shear test must be carried out for determining soil-to-soil shear resistance by placing a compacted soil sample in the upper half of the box. The Mohr-Coulomb failure envelop for these tests would be the following:

$$\tau = c + \sigma_n \tan\phi \tag{4}$$

where τ is the shear strength of the soil, c is the apparent cohesion of the soil and ϕ is the friction angle of the soil. The percentage efficiencies for adhesion and the friction angle are then calculated:

$$E_c = (c_a/c)\,100 \tag{5}$$

$$E_\delta = (\tan\delta\,/\tan\phi)\,100 \tag{6}$$

In liner systems with geomembranes, the geomembrane creating the barrier layer is sometimes used with a geotextile. For these types of designs, the interface shear parameters between the geomembrane and the geotextile can also be determined using the shear box method.

The direct shear method measures the peak interface shear stress. Tests that use a ring-shear device can generate larger shear displacements, and they show residual stresses significantly lower than those obtained by the direct shear box and demonstrate the polishing action at the interface during large displacements. The details can be found in Koerner (2016b). In addition to these tests, other methods can be used for determining the interface shear (Swan, 2019): the tilt table method, the pullout interface method, and the triaxial shear test.

ASTM 5321, approved in 1992, describes the standard test method for determining shear strength described above. ASTM D6243, approved in 1998, describes the standard test method for determining the internal and interface shear resistance of geosynthetic clay liners using the direct shear method. ASTM D7702 is the standard guide to evaluating direct shear results involving geosynthetics.

2.2.3.7 Anchorage

The edges of the geomembrane, at the top of the sloped sides, are generally secured by sandwiching the geomembrane between the subgrade and the cover layer. Figure 2-12 shows different types of designs for anchoring the geomembrane at the top of the side walls. Anchorage tests are designed to test the behavior of the geomembrane sandwiched between two layers. Small-scale laboratory experiments use a 200 mm wide sample. One side of the sample is embedded between two channels, and the other end is attached to a uniaxial testing machine. The surfaces of the channels are fitted with a material representing the roughness of the materials sandwiching the geomembrane. The channels are pressurized using a hydraulic jack. The end of the geomembrane attached to the uniaxial tensile testing machine is pulled to determine the anchorage depth needed to mobilize the strength of the geomembrane (also called the mobilization distance). The mobilization distances for HDPE, PVC and CSPE-R (scrim reinforced geomembranes) range from 50 mm to 300 mm. The mobilization distance represents the distance required to dissipate the normal forces in the geomembrane, and it is quite small. The standard test procedure is described in ASTM D6706 and ISO 13430.

Large-scale anchorage tests are carried out using a larger sample (1,000×1,000 mm) to avoid scale effects. These tests are described in Koerner (1990 and 1991).

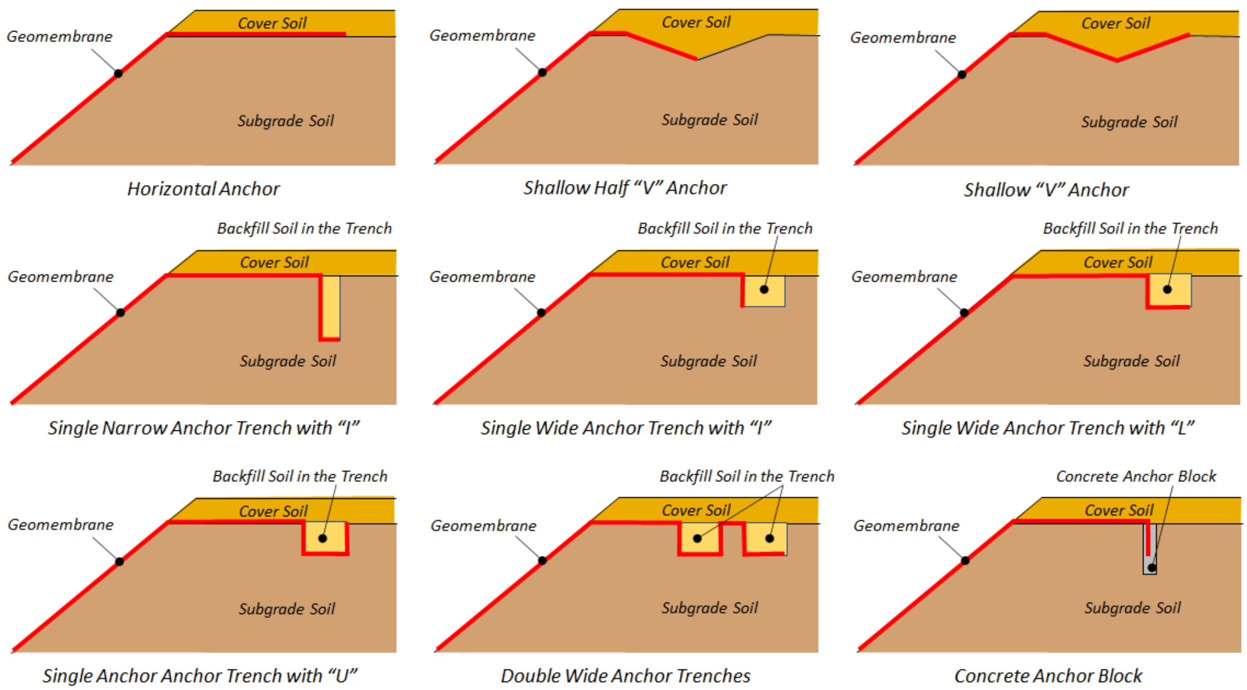

Figure 2-12 Types of Geomembrane Anchorage Designs (Lajevardi, Briancon, and Dias, 2014).

2.2.3.8 Stress Cracking

Stress cracking is defined by ASTM as "an external or internal rupture in a plastic caused by tensile stress less than its short-time mechanical stress." There are several standardized tests for measuring stress-cracking behavior of geomembranes:

- Bent-strip stress cracking test: This test is only applicable to HDPE geomembranes with a semicrystalline material structure. The standard procedure for the test is described in ASTM D1693. This test is no longer used.

- Constant load stress cracking test: This test is also called a notched constant tension load test (NCTL) and the standard procedure is described in ASTM D3597 (ISO 16700). In this test, a dumbbell-shaped sample is placed in a wetting agent (usually Ipegal 630) at 50°C under a constant tensile load corresponding to a known percentage of the yield stress. The test is performed at different percentages of yield stress to measure the time in hours for the geomembrane to transition from ductile to brittle behavior. Koerner (2016b) recommends a transition time of 150 hrs or more when selecting a geomembrane.

- Single-point stress cracking test: The NCTL test is the standard test for stress cracking. However, it takes considerable time to develop the full curve of yield stress versus failure time to determine the transition time from ductile to brittle behavior. ASTM D5397 describes a faster test for quality control applications called single-point notched constant tension load test (SP-NCTL). This test is the same as the NCTL test except that the test is only carried out for 30% of yield stress. If the sample does not fail in 300 hrs, it is concluded that the geomembrane will have a transition time of 150 hrs or more.

2.2.4 Endurance Properties of Geomembranes

The tests for the endurance properties of geomembranes investigate the behavior of the molecular structure of the geomembrane that will cause it to become brittle and modify its stress-strain behavior. Various types of endurance tests for geomembranes are summarized in Figure 2-13. These are only briefly described with reference to standard ASTM methods. The details can be found in related ASTM documents. Koerner (2016b) also provides a short discussion of each method.

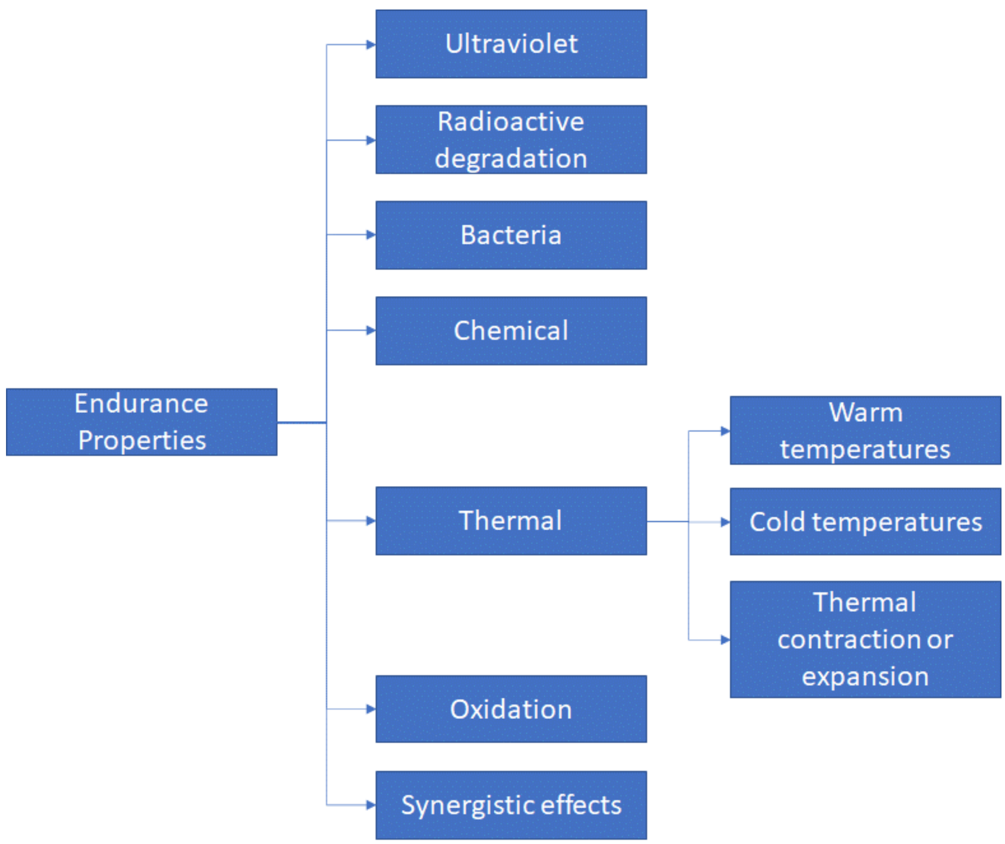

Figure 2-13 Relevant Endurance Properties of Geomembranes.

2.2.4.1 Ultraviolet Light

Because it has a short wavelength, UV light can penetrate the geomembrane and cause chain scission, bond breaking, and surface degradation. ASTM G26 (fluorescent tube method) and ASTM D4355/ISO 4893 (xenon arc method) are used to simulate the effects of long-term exposure to UV light. ASTM D1435, ASTM D3334, and ASTM D5970 describe outdoor weathering tests. For exposed liners, it is possible to obtain 20-year or longer warranties from the manufacturers of CSPE-R and HDPE geomembranes.

2.2.4.2 Radioactive Degradation

Radioactivity greater than 10^6 rads has the potential to cause degradation of the geomembrane by chain scission. This is mostly relevant to waste containment applications.

2.2.4.3 Biological Degradation

A large variety of organisms living in the subgrade and cover soil may sometimes be damaging to geomembrane liners.

- ASTM G21 measures the resistance of the geomembrane to fungi.
- ASTM G22 measures the resistance of the geomembrane to bacteria.

2.2.4.4 Chemical Degradation

Geomembranes' chemical resistance is mostly of concern for waste storing applications. Numerous tests exist but will not be discussed here. A summary can be found in Koerner (2016b).

2.2.4.5 Thermal Degradation

Depending on where they are installed, geomembranes are subjected to variations in temperature that cause the material to go through cycles of contraction and expansion. The thermal expansion and contraction coefficient is different for different types of geomembranes. ASTM D2102 and ASTM D2259 measure contraction, and ASTM D1042 and ASTM D1204 measure expansion. For polyethylene geomembranes, thermal expansion and contraction coefficients range from 11 to 13×10^{-5}/°C for HDPE to 15–25×10^{-5}/°C for LLDPE. Polypropylene geomembranes have a lower value of 5–9×10^{-5}/°C. Plasticized PVC geomembranes have a thermal expansion and contraction coefficient ranging from 7 to 25×10^{-5}/°C. Details can be found in ASTM documents and Koerner (2016b).

2.2.4.6 Oxidation

When a geomembrane is exposed to air, oxygen combines with free radicals in the polyethylene chain to form hydroperoxy, which can propagate and gradually lead to chain scission. This problem is less of a concern when the geomembrane is always underwater, because there is insufficient oxygen on the surface of the geomembrane. However, the water level in closed-loop PSH reservoirs fluctuates with the cycles of power generation (upper reservoir emptied and lower reservoir is filled) and pumping (upper reservoir is filled and lower reservoir is emptied). Manufacturers are now adding various types of antioxidants during the manufacture of geomembranes to prevent oxidation. This continues to be a research and development area. There are two standard tests for geomembrane resistance to oxidation:

- The standard oxidative induction time (standard OIT) test is described by ASTM D3895 or ISO 11357.
- The high-pressure oxidative induction time (high-pressure OIT) test is described by ASTM D5885.

2.2.5 Lifetime Prediction for Geomembranes

Lifetime prediction methods aim to predict the long-term behavior of a material using accelerated aging. Generally, the following methods are available for interpreting the test data:

1. *Stress limit testing* is widely used by the HDPE pipe industry in the U.S. to provide the hydrostatic design basis stress.
2. *The rate process method* is used in Europe for pipes and geomembranes. It is similar to stress limit testing.
3. *The Hoechst multi-parameter approach* utilizes biaxial stresses and stress relaxation for lifetime prediction. The test samples can have seams.
4. *Arrhenius modeling*, introduced by Mitchell and Spanner (1984), combines compressive stress, chemical exposure above, oxidation below, elevated temperatures, and long testing times in a single apparatus for lifetime prediction.

2.2.6 Standard Tests for Geomembranes

Table 2-6 presents a summary of index and performance tests that are used to measure the properties of geomembranes. Some of these tests have been described briefly. Additional information can be found in related ASTM documents, Koerner (2016a and b), Muller (2007), and the references cited.

Table 2-6 List of Index and Performance Tests to Measure Various Properties of Geomembranes.

Measured Property	Test Type	Applicable to	ASTM Method	GRI Method	ISO Method	Other
Physical Property						
Thickness	Deadweight micrometers with a spring-loaded presser foot	Smooth	ASTM 5199	GRI GM13	ISO 09863	
Core thickness	Deadweight micrometers with a spring-loaded presser conical point	Textured	ASTM 5994			
Asperity height	Micrometer with a stylus	Textured	ASTM D7466			
	Profilometry					See Dove and Frost (1996)
	3D topography					See Ramsey and Youngblood (2009a and b)
Density	Archimedes' principle		ASTM D792		ISO R1183	
	Density gradient technique		ASTM D505			
Melt (flow) index	Forcing heated polymer from the bottom of the test apparatus		ASTM D1238		ISO 1133	
Mass per unit area (weight)	Weight of the sample with a known surface area		ASTM D1910, ASTM D1593		ISO 9864	
Water vapor transmission	Weight loss of water in an aluminum cup sealed by the geomembrane		ASTM E96			
Solvent vapor transmission	Similar to water vapor transmission but using a selected solvent		ASTM E97			
Mechanical Property						
Tensile behavior	Narrow sample tests	HDPE, LLDPE, fPP	ASTM D6693			
		PVC	ASTM D882			
		fPP, EPDM-R; CSPE-R, EIA-R	ASTM D751			
	Wide sample tests		ASTM D4885			ISO 10319

Table 2-6 (Cont.)

Measured Property	Test Type	Applicable to	ASTM Method	GRI Method	ISO Method	Other
Tensile behavior of Seams	Shear test		ASTM 6392, ASTM D882, ASTM D751			
	Peel test		ASTM D6392, ASTM D882, ASTM D413			
Tear resistance	Uniaxial tensile test using a sample with a 90° notch		ASTM D1004			
	Uniaxial tensile test using a trapezoidal sample		ASTM D2263		ISO 13434	
	Uniaxial test using three different sample shapes		ASTM D624		ISO 34	
	Elmdorf type falling pendulum		ASTM D1424			
	Tongue tear method		ASTM D5884			
	Used for combination of geomembrane and a geotextile		ASTM D751			
Impact resistance	Falling dart method		ASTM D1709		ISO 13433	
	Falling weight method		ASTM D3029			
	Pendulum type		ASTM D1822, ASTM D746, ASTM D3998			
Impact resistance	Elmendorf tear tester fitted with Spencer impact tester (pendulum type)		ASTM D3420			
Puncture resistance	Pushing a rod through a geomembrane in a universal testing machine		ASTM D4883			
Interface shear	Direct shear method using a shear box	Geomembranes	ASTM 5321			
	Direct shear method using a shear box	Compacted clay geomembranes	ASTM D6243			

Table 2-6 (Cont.)

Measured Property	Test Type	Applicable to	ASTM Method	GRI Method	ISO Method	Other
	Direct shear method using a shear box	Geosynthetics	ASTM D7704			
	Torsional ring shear apparatus					
	Cylinder ring shear					See Stark and Poeppel (1994) and Moss (1999)
	Tilt table					
	Pullout					
	Triaxial shear					
Anchorage	Pullout		ASTM D6706		ISO 13430	
Stress cracking	Bent strip	No longer used	ASTM D1693			
	Constant load		ASTM D3597		ISO 16700	
	Single point constant load		ASTM D5397			
Endurance						
Ultraviolet light	Fluorescent tube method		ASTM G26			
	Xenon arc method		ASTM D4355		ISO 4893	
	Generally relevant for waste containment					
Radioactive degradation						
Biological degradation	Resistance to fungi		ASTM G21			
	Resistance to bacteria		ASTM G22			
Chemical degradation	Different methods based on the chemical used		ASTM D5322		ISO 175	
Oxidation	Standard oxidative induction time (Standard OIT)		ASTM D3895		ISO 11357	
	High-pressure oxidative induction time (high-pressure OIT)		ASTM D5885			

Table 2-6 (Cont.)

Measured Property	Test Type	Applicable to	ASTM Method	GRI Method	ISO Method	Other
Lifetime Prediction						
Accelerated aging	Stress limit testing					
	Rate process method					
	Hoechst multi-parameter approach					
	Arrhenius modeling		ASTM F1980		ISO 11607-1	
In Relation to Density						
Carbon black content			ASTM D1603		ISO 6964	
Carbon black dispersion			ASTM 5596		ISO 11420	

2.3 Design of Geomembrane Liner Systems

2.3.1 General Approach and Terminology

2.3.2 Types of Liner Systems

A liner system consists of multiple layers, each serving a purpose. It is impossible to review all types of cross sections of liner systems, which are different from one project to another based on specific requirements.

One of the first decisions in selecting a liner system is to decide whether the topmost liner will be covered by a protective layer or left exposed to water and atmosphere. A cover layer protects the liner from outside elements such as ozone, UV light, temperature extremes, ice formation, and wind. The cover layer also prevents damage to the geomembrane layer due to animal activity and vandalism.

Reservoirs for PSH facilities are subject to repeated rapid drawdown and rapid filling during operation. Covered liners can provide an advantage because they are protected from the repeated cycles of rapid drawdown and filling. The protective cover layers, however, must be properly designed to avoid sloughing due to the rapid drawdown and filling. This requires proper grading of the material and ensuring sufficient friction between the protective layer and the geomembrane.

Figure 2-14 schematically shows typical cross sections of different types of liner systems for liquid impounding reservoirs. The illustrations in the left column depict liners without a protection layer, and the illustrations in the right column correspond to liners with a protective layer. Cross sections (a) and (b) in the first row of Figure 2-14 represent typical liner systems with a single liner, i.e., a single layer of geomembrane with and without a cover, respectively. These are the simplest liner system designs, in which the liner is directly placed over a properly prepared support layer that has a smooth, leveled surface free of large, pointed elements that can damage the geomembrane. These designs do not provide an explicitly designed drainage underlayer (underdrain). If the underlying embankment materials do not drain freely, an accumulation of gases and liquid leakage can lift the geomembrane (see Section 2.6).

The liner system in illustration (c) in Figure 2-14 has a geotextile underliner between the unprotected liner and the support layer. Koerner (2016b) provides several reasons why the geotextile underliner is considered good design practice:

- The geotextile layer placed over the support layer provides a clean working area when placing the geomembrane layer and welding the seams.

- The geotextile underliner also protects the geomembrane layer against punctures caused by the irregularities in the support layer due to the weight of the support layer, the water pressure, and the weight of vehicular traffic during the construction phase.

- The geotextile underliner increases the frictional resistance between the geomembrane liner and the support liner, which is particularly important for preventing excessive
- stresses on the geomembrane placed on side slopes and the geomembrane entering an anchorage trench.

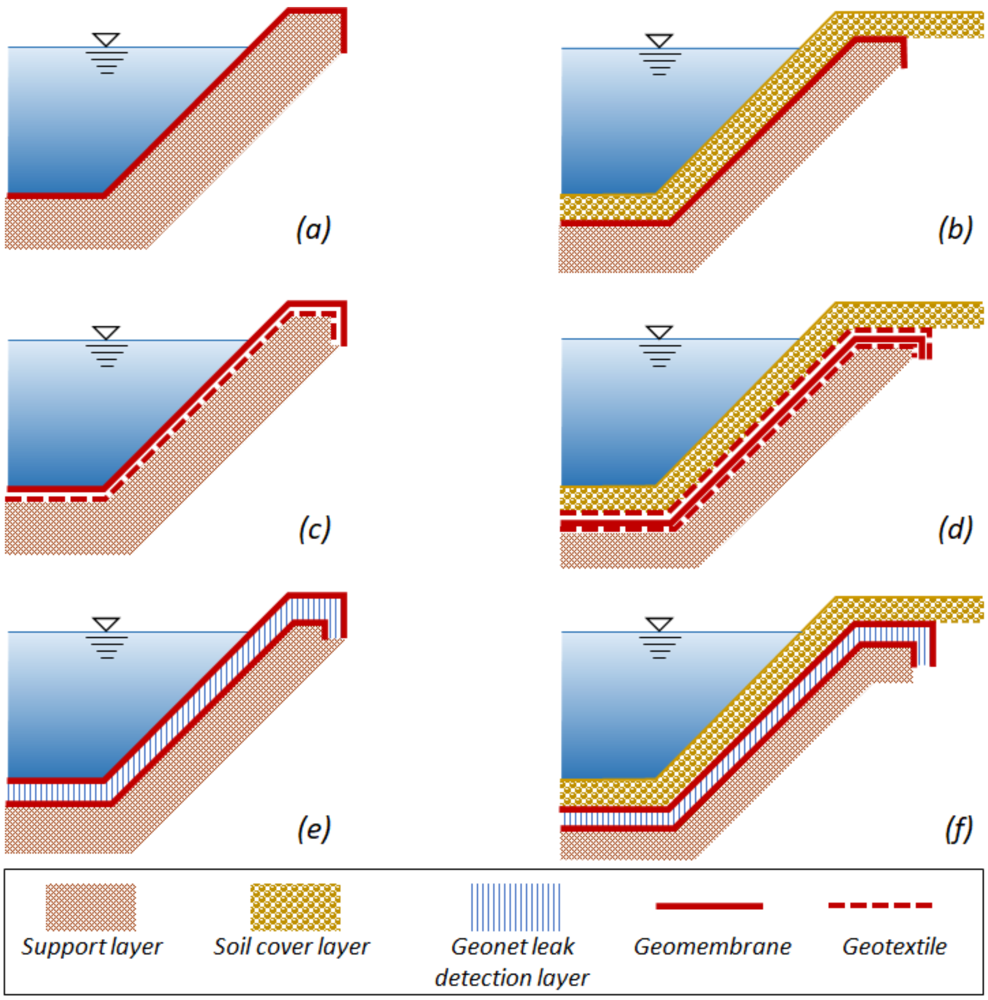

Figure 2-14 Types of Geomembrane Liner Systems for Liquid Impounding Reservoirs. According to Koerner (2016b): (a) single unprotected liner, (b) single liner protected by a soil cover layer, (c) single unprotected liner with geotextile underliner, (d) Single liner with geotextile underliner and overliner protected by a soil cover layer, (e) unprotected double liner with geonet leak detection between liners, and (f) double liner with geonet leak detection between liners and soil covering, which may or may not have geotextile or geogrid layers as veneer reinforcement (Koerner, 2016b).

- The biodegradation of organic material in the subsurface soil layers may cause gases to accumulate under the liner. A rising water table in the subsurface layer may also cause air to be expelled from the soil and accumulate under the geomembrane. A properly selected geotextile may help to evacuate these accumulated gases and water laterally from the sides. Nonwoven needle-punched geotextiles, geonets, or drainage geocomposites with sufficient transmissivity can be used for this purpose.[*]

The liner system in illustration (d) in Figure 2-14 uses also a single liner layer but is designed with a protective cover layer. The single liner is sandwiched between geotextile underliner and overliner layers. The geotextile overliner has a function similar to that of the geotextile underliner in illustration (c): It increases the friction between the geomembrane liner and the protective layer, and it also protects the geomembrane against puncturing from large pointed, angular materials that may be present in the protective layer.

Illustrations (e) and (f) in the last row of Figure 2-14 represent unprotected and protected liner system designs with a double liner and a geonet leak detection layer in between. It should be noted that these double liner systems can also be designed with underliner and overliner layers made of geotextile or geogrid to serve as reinforcement and to increase friction between liners and adjacent subsurface or protective layers.

Giroud and Bonaparte (1989a) use a slightly different terminology and classification for discussing the calculation of leakages, as shown in Figure 2-15. All the liners in Figure 2-15 are unprotected; however, it is possible to add a protective layer to their design.

They define a "double liner" system as a liner system which includes two barrier layers, i.e., liners (geomembranes), with a drainage layer in between to detect, collect, and remove the liquid that may leak from the topmost liner. Their "single liner" system is defined as a liner system with only a single barrier layer. Giroud and Bonaparte (1989a) clarify their definition by stating that two geomembrane layers placed directly one on top of the other without a drainage layer in between cannot be called a double liner system. In a double liner system, the liner at the top is called a top liner or primary liner, and the one below is the bottom liner or the secondary liner. The leakage collection layer between the top and bottom liners is part of the leakage detection, collection, and removal system and may include collector pipes and sumps. Its function is to capture leakage from the defects in the top liner and facilitate its flow to a collection point for evacuation.

For liquid containment ponds where the geomembrane support layer is made up of low-permeability soils, a single liner system where a geomembrane is placed directly over a compacted layer is not desirable. The head over the geomembrane layer is generally large (on the order of several meters at least) and leakage from defects in the geomembrane would accumulate between the low-permeability compacted clay layer below and the geomembrane layer at the top. The accumulation of the leakage could cause uplift forces to push the geomembrane away from

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[*] See Example 5.7 on page 480-481 in Koerner (2016b).

the compacted clay subgrade. This would have the effect of causing a further increase in the leakage rate and cause “whales” or “hippos.”

For liquid-containing ponds where the geomembrane support layer is low-permeability soil, double liner systems, such as those shown in illustrations (e) and (f) in Figure 2-14 and illustrations on the right side of Figure 2-15, are generally preferred. The general idea is to contain the leakage in a drainage layer sandwiched between the upper geomembrane layer and a second impervious layer, which can be a lower geomembrane layer, a GCL layer, or a compacted clay layer. The final choice will be dictated by the specific conditions of the project.

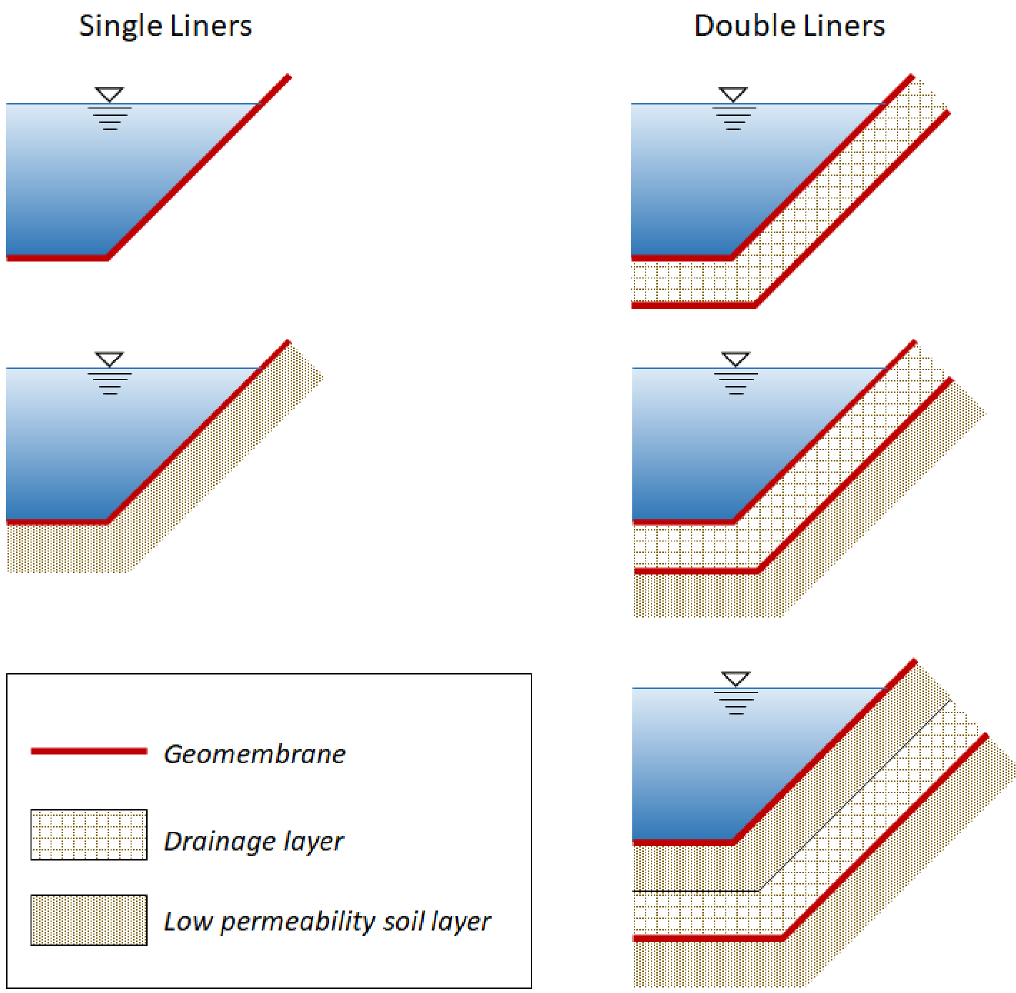


Figure 2-15 Liner System Examples with Single and Double Liners According to Giroud and Bonaparte (1989a).

2.3.2.1 Materials Used in Geomembrane Liner Systems

The selection of a geomembrane for a specific project requires the consideration of numerous factors (Rohe, n.d.): material type, manufacturing process, slope stability (smooth versus textured geomembranes), tensile strength, elongation at failure, failure at yield, multiaxial stress, stress cracking, oxidation, usable welding types, flexibility, chemical compatibility, regulations, etc. In addition, conditions expected during construction (rain, wind, hot and cold temperatures, etc.) and during service (temperature, UV light, etc.) are also important considerations. The selection of the resin (see Table 2-3) should take into account the material to be contained, which is water in the case of reservoirs for PSH applications. For these types of applications, Koerner (2016b) recommends at least 20 years of service life for uncovered geomembranes. The U.S. Department of the Interior's Bureau of Reclamation (USBR, 2018) estimates the typical useful life of uncovered geomembranes used in dam projects to be between 10 and 20 years. The useful life of a covered geomembrane is significantly longer but, given the type of composition (resin, antioxidants and other additives), it remains vulnerable to service temperatures.

Information on the useful life of different types of geomembranes can be found in Koerner (2016b, Section 5.1.5) and Koerner et al. (2011). In these publications, the useful life of the HDPE geomembrane is separated into three stages. Stage A is the so-called oxidative induction time (OIT), which exponentially decreases with higher service temperature (200 years at 20°C to 45 years at 40°C). Stage B is from OIT to the onset of degradation (30 years at 20°C to 10 years at 20°C). Finally, Stage C is the time to reach a 50% degradation of mechanical properties, also called half-life. It also depends on service temperature (208 years at 20°C to 13 years at 40°C). Thus, the half-life (sum of the durations of stages A, B, and C) varies from 446 years at 20°C to 69 years at 40°C. The references given can be consulted for the useful life of other types of geomembranes.

Different resin chemistry, density, and additives also play a role in defining the service life, but in all cases, service life exponentially decreases with increasing service temperature. This is an especially important issue in landfill applications, due to the potential generation of heat by the waste. Jafari et al. (2014) extensively discuss the service life of HDPE geomembranes subjected to elevated temperatures. They point out that in the U.S. there is no regulation of the required service life of a HDPE geomembrane, even for landfill applications.

The thickness of the geomembrane must be decided on by considering the tensile forces it will be subjected to in case of a deformation of the subgrade. This will be discussed in detail in Section 2.4.2. Regulations may also impose a minimum thickness. For example, NRCS (2017) defines the minimum thickness of HDPE and LLDPE liners as 30 mil (0.762 mm or 0.030 inches). For geosynthetic clay liners, a bentonite mass per unit area of 0.75 lbs/ft^2 is recommended (AGRU America, n.d.).

Many geomembranes are commercially available for use as barrier material in liner systems. These are reviewed below, and their advantages and disadvantages are briefly discussed.

Polyethylene (PE) geomembranes are thermoplastic materials. Six types of polyethylene geomembranes are widely used as liners in surface water reservoirs:

1. *High-density polyethylene (HDPE)* is the most widely used and most popular geomembrane for liners of water storage surface reservoirs. It has several advantages, such as low material cost, durability, resistance to ozone and ultraviolet light, high tensile strength relative to its weight, resistance to higher temperatures, being food safe (can be used for drinking water storage), etc. Since HDPE is UV resistant, the liner system can be designed without a cover layer, if necessary. For a given thickness, HDPE geomembranes are relatively more rigid than others. An important advantage of HDPE geomembranes is that they are available in more thicknesses than other types of geomembrane material types. Commercially, HDPE geomembranes are available in thickness ranging from 40 mil to 120 mil (1.016 to 3.048 mm or 0.040 to 0.120 inches). At the lower range, 40 mil, HDPE requires extra care in the finish of the subgrade surface. The installation of HDPE geomembranes at the job site is sensitive to temperature and poor weather conditions. HDPE geomembranes develop an oxidized layer with time. During repairs, the oxidized layer must be cleaned off before repair welding is performed.

2. *Linear low-density polyethylene (LLDPE)* is also a geomembrane type widely used in liners of surface water reservoirs. It is manufactured from virgin polyethylene resins, has a rubber-like flexibility, and provides excellent resistance to low temperatures and UV light. It is durable and has a long service life. It can be laid on the subgrade with an almost skin-tight finish. Because of its higher flexibility, it is particularly suited for installation on subgrades prone to differential settlement. Commercially, LLDPE is available in thicknesses in the range of 20 mil to 80 mil (0.508 to 2.032 mm or 0.020 to 0.080 inches) and is available in a smooth, textured, or single-side textured finish. Geomembranes 40 mil and 60 mil thick can be welded with either extrusion or wedge welding. Its narrow operating range for heat sealing is a disadvantage.

3. Chlorosulphonated polyethylene (CSPE) is a thermoplastic material. Geomembranes made from CSPE are always scrim-reinforced and called CSPE-R. They are often used uncovered because of their excellent resistance to UV light. Their resistance to puncture and tear is only fair. Although initially a thermoplastic material, CSPE becomes thermoset with time. Thus, although initially extrusion or hot wedge welding methods can be used, after the geomembranes become thermoset only solvents can be used for creating seams.

4. *Chlorinated polyethylene (CPE)* is a polyethylene in which chlorine is substituted for some hydrogen atoms. CPE may have a chlorine content from 34% to 44%. This type of geomembrane has good UV and chemical resistance (AZO Materials, n.d.). It is flexible and has high tear strength. It is inherently difficult to ignite but has a high gas permeability. It is often used as an impact modifier for PVC or compounded with LDPE or HDPE film to improve toughness. CPE geomembranes are used as pond liners and for agricultural applications.

5. *Reinforced chlorinated polyethylene (CPER)* is similar to CPE, but a reinforcing material is used. It was successfully used by the USBR as early as the 1970s (USBR, n.d.). The USBR's first reservoir lining project was the placement of a 45 mil (1.14 mm) CPER liner at the Mt. Elbert Forebay project (see Section 3.4.1) to address excessive seepage

through the compacted earth lining (Comer, 1994). The choice of a CPER liner, however, was mainly based on its availability to meet the construction schedule.

6. *Reinforced polyethylene thermo-responsive polymer* geomembranes are similar to reinforced polypropylene (RPP) geomembranes, with the difference that a polyethylene material is used instead of polyester. TRP geomembranes have UV light stability and good resistance to chemicals and low temperatures. They are used for long-term water containment and industrial waste applications (GSSB, n.d.).

Polyvinyl chloride (PVC) is a thermoplastic material manufactured from vinyl, plasticizers, and stabilizers. PVC is a rigid material. Addition of plasticizers (primary and secondary) makes it soft and suitable for use as a geomembrane. PVC geomembranes have excellent resistance to extreme weather conditions, punctures, abrasion, tears, and various contaminants, but they have poor resistance to UV light. Therefore, for a long service life, PVC geomembranes require a cover layer. They are extensively used as liners and/or covers in waste impoundments. They are resistant to differential settlement and are easy to repair with the help of a special repair kit.

Ethylene propylene diene monomer (EPDM) is a flexible, lightweight, durable geomembrane with a rubber-like texture. It has excellent resistance to extreme weather conditions, UV light, and punctures. EPDM is an elastomeric material, which is thermoset. It cannot be welded by melting with the use of heat; welding of seams is accomplished using an adhesive or tape system. EPDM is ideal for extreme weather conditions and puncture resistance and is used in linings for dams.

Reinforced polypropylene (RPP) geomembranes are made of UV-stabilized and polyester-reinforced polypropylene copolymer with a nylon scrim. It is a highly flexible and durable geomembrane with good chemical resistance properties. It is, however, quite expensive (Ait, 2021) compared to HDPE and PVC liners that offer the same qualities of longevity and chemical resistance.

Butyl rubber and *ethylene-propylene rubber (EPDM)* have good resistance to UV light and oxidation but are difficult to seam.

2.3.2.2 Seams and Seaming

Geomembranes are generally transported to a site as rolls or panels and assembled by seaming using different types of welds. The type of weld that can be used depends on the type of the polymer material used in manufacturing the geomembrane. The types of geomembranes used as barrier materials in liner systems were briefly reviewed above, and their chemical compositions are listed inTable 2-3. Depending on the chemical composition and characteristics, there are five usual welding methods to seam geomembrane rolls or panels:

1. *Hot air welding:* This method uses a small lightweight welding apparatus that blows hot air between the overlapping surfaces of the two geomembranes that are being seamed. The hot air melts the surfaces of the geomembranes, which are then pressed together to form a waterproof, tight bond. Hot air welding is particularly suitable for detail work at corners, around pipes, etc.

2. *Extrusion welding:* An extrusion welder compatible with the geomembranes being seamed together forces a plastic rod through a heated barrel and deposits it as a molten bead along the edge of the upper geomembrane. At the same time, hot air is blown to soften the geomembrane layers. A Teflon shoe presses over the molten bead to form a strong bond between the bead and the two superposed geomembrane layers.

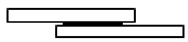
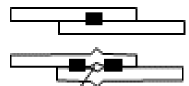
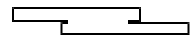
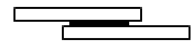
3. *Wedge welding:* There are two types of wedge welding: single wedge and split wedge. In both cases, the principle is the same: The interface of the two overlapping geomembranes is softened by a heated metal wedge. As the apparatus is pulled along the seam a roller presses the softened interfaces to form a strong bond. For thinner geomembranes, a single wedge is used to create a single line of seam. The width of the seam is at least 25 mm (~1 in). For thicker geomembranes, a split wedge can be used to create two lines of seam with air in between them. Wedge welds are used only for straight run welds. They are not suitable for detail work or curves. Wedge welded geomembranes always require finishing work.

4. *Chemical welding:* Chemical welding requires that the geomembranes to be seamed together can be softened by a special solvent. The solvent is simply applied between the overlapping layers using a brush. The surfaces of the two geomembranes are dissolved by the solvent and they bond when they are pressed against each other. Chemical welding is used for PVC and urethane-based geomembranes. Chemical welds have a long curing time, which may delay destructive and nondestructive testing of the seams during construction. Also, the quality of chemical welding can be affected by low temperatures and high humidity.

5. *Adhesives:* Adhesives are applied between the overlapping geomembranes. An adhesive does not dissolve the surfaces of geomembranes, but it contains a bonding agent that attaches to both layers (EPI, n.d.). Thus, at the end of the curing time, the applied adhesive remains as a separate bonding material between the geomembranes.

Table 2-7 summarizes the types of welds that can be used with different geomembrane types. USBR (2018) provides a detailed discussion of geomembrane welding and seaming.

Table 2-7 Types of Welds Used for Field Seaming Different Geomembrane Types.

Geomembrane Type	Hot-Air Welding	Extrusion Welding	Wedge Welding	Chemical Welding	Adhesive Welding
					
HDPE	Y	N	Y	N	N
LLDPE	Y	Y	N	N	N
PVC	Y	N	Y	Y	Y
CSPE	Y	N	Y	Y	Y
Urethane	Y	Y	N	Y	Y
EPDM	N	N	N	N	Y
Polypropylene	Y	N	Y	N	N

Various types of standard methods have been developed for testing the integrity of the welded seams (Stessel, 1998, and Peggs, 1990): destructive and nondestructive tests in the laboratory and in the field. The destructive tests require cutting out a section containing the seam (Peggs, 1990 and Rogbeck et al., 1994) to be tested in the laboratory, which means that the cut-out in the geomembrane must subsequently be repaired. The standard shear and peel tests are described in Section 2.2.3.2, and some of the available nondestructive tests are described in Section 2.5).

2.3.2.3 Underlayers and Protective Layers

In the liner systems shown in Figure 2-15, the low-permeability soil layer is constructed of clays, silty clays, clayey sands, and silty sands. The coefficients of permeability and hydraulic conductivity for different types of geo-materials are summarized in Figure 2-1. Table 2-8 shows the range of hydraulic conductivity values for selected low-permeability soil types, some of which are listed above. The highest maximum conductivity value in this table is 1.80×10^{-5} ft/s (1.80×10^{-5} m/s) for clayey sand. The dry and submerged densities for selected low-permeability soils are listed in Table 2-9 .

Table 2-8 Hydraulic Conductivity Range for Selected Low-Permeability Soils (StructX, n.d.).

Soil Description	Hydraulic conductivity, k			
	U.S. Customary (ft/s)		SI (m/s)	
	Min	Max	Min	Max
Inorganic silty fine sand/clayey fine sand, slight plasticity	1.64×10^{-8}	3.28×10^{-6}	5.00×10^{-9}	1.00×10^{-6}
Silty sand	3.28×10^{-8}	1.64×10^{-5}	1.00×10^{-8}	5.00×10^{-6}
Clayey sand	1.80×10^{-8}	1.80×10^{-5}	5.50×10^{-9}	5.50×10^{-6}
Inorganic silt, high plasticity	3.00×10^{-10}	1.64×10^{-7}	1.00×10^{-10}	5.00×10^{-8}
Silt, compacted	2.30×10^{-9}	2.30×10^{-7}	7.00×10^{-10}	7.00×10^{-8}
Inorganic clay/silty clay/sandy clay, low plasticity	1.60×10^{-9}	1.64E-07	5.00×10^{-10}	5.00×10^{-8}
Organic clay/silty clay, low plasticity	1.64×10^{-8}	3.28×10^{-7}	5.00×10^{-9}	1.00×10^{-7}
Organic clay, high plasticity	1.60×10^{-9}	3.28×10^{-7}	5.00×10^{-10}	1.00×10^{-7}
Inorganic clay, high plasticity	3.00×10^{-10}	3.28×10^{-7}	1.00×10^{-10}	1.00×10^{-7}
Clay	3.28×10^{-11}	1.54×10^{-8}	1.00×10^{-11}	4.70×10^{-9}
Clay, compacted	3.28×10^{-10}	3.28×10^{-9}	1.00×10^{-10}	1.00×10^{-9}

Table 2-9 Dry and Submerged Densities of Some Low-Permeability Soils (StructX, n.d.).

Soil Description	Dry Density, ρ_s				Submerged Density, ρ_{ss}			
	U.S. Custom. (lb/ft^3)		SI (kg/m^3)		U.S. Custom. (lb/ft^3)		SI (kg/m^3)	
	Min	Max	Min	Max	Min	Max	Min	Max
Silt, uniform/inorganic	81	136	1297	2179	51	73	817	1169
Silty sand	88	142	1410	2275	54	79	865	1265
Sandy or silty clay	100	147	1602	2355	38	85	609	1362
Clay	94	133	1506	2130	31	71	497	1137
Organic silt	87	131	1394	2098	25	69	400	1105
Organic clay	81	125	1297	2002	18	62	288	993

Detailed requirements for subgrade design and construction with geomembranes are provided by FGI (2010). This online document provides photographs of acceptable and unacceptable subgrade conditions. Some geomembrane manufacturing and installation companies also provide guidelines for the preparation of the subgrade (Layfield, 2019, and Solmax, 2020). It is important that the prepared surface that will be in contact with the geomembrane is uniform, well-compacted, and free of any sharp stones. The surface should not have any protruding objects. Clay lumps sticking out of the surface should be avoided. The compaction should create a firm layer to support the weight and traffic of the construction equipment without surface deformation. The traffic over the subgrade must be kept to a minimum, and any deformation observed after the passage of a construction vehicle must be repaired. The compaction around pipes and concrete structures requires special care to avoid differential settlement that can cause the failure of the liner system.

The leakage collection layers are constructed using high-permeability materials. Sands, gravels, or a mixture of the two are typically used for drainage layers. It is also possible to use synthetic materials (also called synthetic transmission media) such as thick needle-punched nonwoven geotextiles, geonets, geomats, and corrugated or waffled plates (Giroud and Bonaparte, 1989a). The underliner drainage system should have a bottom slope of at least 1% for ponds (NRCS 2017). The leakage collection layers may be equipped with leakage monitoring sensors, collector pipes, etc. NRCS (2017) does not recommend a drainage layer or a venting system under a GCL in a composite liner system. A discussion of the design of drainage layers can be found in Koerner and Koerner (2015).

NRCS (2017) recommends a minimum thickness of 12 inches of cover over the geomembrane to protect it from weathering and damage. The maximum allowable particle size of cover soil is 3/8 inch for geomembrane liners and 1/2 inch for GCLs. The stability of the cover layer should be checked under all operational and exposure conditions, such as rapid drawdown (see Sections 2.4.4 and 2.4.5).

A typical geomembrane and drainage specification can be found in AGRU (2020).

2.3.2.4 Texture and Reinforcement Options

It is important to note that, when selecting a geomembrane, the designer should also consider high-level options regarding texture and reinforcement of the geomembrane. While a detailed discussion of these options is beyond the scope of this report, a brief discussion follows:

- Textured geomembranes offer increased friction between the textured surface and the adjacent material, so they are often used on steep slopes. Non-textured and textured geomembranes were briefly introduced and discussed in Section 2.2, and the manufacturing processes were briefly mentioned. Measuring the core thickness and the height of the asperities for textured geomembranes was discussed in Section 2.2.2.1.

- Unreinforced geomembranes are manufactured using either an extrusion process or a calendered process. They derive their strength from the core thickness of the geomembrane, which may vary from 5 mil to 200 mil (0.107 mm to 5.08 mm) for extruded geomembranes, and from 10 mil to 80 mil (0.254 mm to 2.032 mm) for calendered geomembranes. They are deployed as rolls and are welded in the field. Exposure to sunlight may cause expansion and lead to wrinkles and cracking.

- Reinforced (coated or laminated) geomembranes are high-performance geomembranes. Coated geomembranes consist of a synthetic fabric coated with a polymeric material. They are manufactured with molecular bond between the fabric and the polymeric coating material. Laminated reinforced geomembranes use a loosely woven scrim rather than fabric. Reinforced geomembranes derive their strength from the fabric used as reinforcement. Thicknesses for reinforced geomembranes are in the 20 to 60 mil range (0.508 to 1.524 mm). Reinforced geomembranes are ideally suited for use as liners on steep slopes because they do not slide down under their own weight, unlike unreinforced geomembranes. The reinforced geomembranes can be prefabricated in the factory in modules that fit different parts of the reservoir. This helps to reduce field seaming and avoid installation errors.

Geosynthetics (2021) provides comparative product data for five different geomembrane products from 15 different manufacturers. Table 2-10 provides the list of companies and the geomembrane products that are included in this comparative product study. The comparison considers base polymer, dimensional properties, physical properties, tensile properties, additives, etc. For each geomembrane types multiple products from each company are included. The details can be viewed in the original reference, which is available online.

Table 2-10 Companies and Their Commercial Products

Company	Poly-ethylene non-HDPE	Poly-ethylene HDPE	Poly-propylene	PVC and EPDM	Reinforced
AGRU America Inc. www.agruamerica.com	Y	Y			
Atarfil S.L. www.atarfil.com	Y	Y	Y		Y
E2-E Squared (formerly EPT) www.e2techtextiles.com	Y		Y	Y	Y
HUITEX www.huitex.com	Y	Y			
Layfield Environmental Containment www.layfieldgroup.com	Y	Y			Y
Raven Engineered Films Inc. www.ravengeo.com	Y	Y	Y	Y	
Solmax International Inc. www.solmax.com	Y	Y			
Cooley Group www.cooleygroup.com			Y	Y	Y
Plastatech Engineering Ltd. www.plastatech.com				Y	Y
BTL Liners www.btlliners.com					Y
Burke Industries www.burkeind.com					Y
Inland Tarp & Liner, LLC www.inlandtarp.com					Y
InterTape Polymer www.itape.com					Y
Owens Corning www.owenscorning.com/rhinomat					Y
Seaman Corporation www.xr-technology.com					Y

2.3.3 Calculation of Leakage for Geomembrane Liner Systems

As a material, the geomembrane is virtually impervious. The permeation of liquid through the geomembrane due to Fick's diffusion is extremely small and generally can be ignored. That does not mean that the geomembranes in a liner system do not leak. Although the losses due to permeation of the liquid are small, geomembranes are not perfect and may contain defects created during the manufacturing process or during construction on the job site. Under the pressure of the stored volume, defects in the geomembrane, such as holes and cuts, can cause loss of the stored material.

Obviously, the size of the hole plays a role in the rate at which the liquid can pass through it in a given time period. The volume of liquid lost through a defect in the geomembrane during a unit of time is called the leakage rate.

Giroud and Bonaparte (1989a) note that due to its thinness, even if only leakage due to the permeation of water is considered, the breakthrough time for a geomembrane is only on the order of a few weeks, while the breakthrough time for a low-permeability 3 ft compacted clay layer without any cracks is on the order of 10 years or more. However, the presence of the geomembrane over the compacted clay layer in composite liners reduces the leakage rate—in fact, analytical and numerical calculations show that the presence of a geomembrane over a compacted clay layer reduces the leakage rate by several orders of magnitude.

Giroud and Bonaparte (1989a and 1989b) provide separate formulae to calculate the leakage rate for geomembrane liners (i.e., a geomembrane alone over a drainage layer) and composite liners (a geomembrane with an underlying layer of low-permeability compacted clay). These formulae are briefly summarized below.

2.3.3.1 Leakage Rate Through a Geomembrane Liner

In calculating the leakage rates for geomembrane liners, Giroud and Bonaparte (1989a) distinguish between different sized holes. In Giroud (1984), a pinhole refers to a small hole created during the manufacturing process. The pinhole is assumed to have a diameter significantly less than the thickness of the geomembrane. They provide the following expression for leakage rate due to pinholes in the geomembrane:

$$Q = \frac{\pi \rho g h_w d^4}{128 \eta T_g} \tag{7}$$

This expression was derived based on the Poiseuille equation assuming that the pinhole can be seen as a pipe. In this equation, Q (m^3/s) is the leakage rate through pinhole defect in the geomembrane, h_w (m) is the liquid head on the geomembrane, T_g (m) is the thickness of the geomembrane, ρ (kg/m^3) and η ($kgm^{-1}s^{-1}$) are the density and dynamic viscosity of the stored liquid (water in the case of PSH reservoirs*), and $g = 9.81$ (m/s^2) is the gravitational acceleration.

Giroud (1984) distinguishes holes from pinholes as having a diameter equal to or larger than the thickness of the geomembrane. The calculation of the leakage rate through a hole requires consideration of the permeability of the overlying and underlying layers.

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* For water at 20°: $\rho = 1000$ (kg/m^3) and $\eta = 10^{-3}$ (kg/m/s).

Assuming that the geomembrane is between two infinitely pervious media, Giroud and Bonaparte (1989a) use Bernoulli's equation through an orifice to provide an expression for the leakage Q through a hole.

$$Q = C_B a\sqrt{2gh_w} \tag{8}$$

In this equation a (m^2) is the area of the hole, and C_B is a dimensionless coefficient that depends on the entrance condition defined by the edges of the hole. Assuming sharp edges, $C_B = 0.6$.

Geomembranes placed in the field may develop defects due to various factors, such as defective seams, damage to geomembranes due to machinery and human traffic, puncturing by stones and other sharp or pointed objects in the subgrade or the cover layer, tensile failure, environmental stress cracking, faulty connections between the geomembrane and penetrating pipes or concrete structures, etc. Giroud and Bonaparte (1989a) provide statistical data on the density of such defects. However, the data is old and probably does not reflect current conditions, considering the advances in technologies for placing and seaming geomembranes on the field. Touze-Foltz (2001) provides detailed data on geomembrane defects based on electrical leak-location system data (see Section 2.3.4). Table 2-14 in Section 2.6 shows the results of a statistical analysis of the defects in geomembrane liners that was performed by Nosko and Touze-Foltz in 2000 (Mueller, 2007). Nevertheless, for the purposes of design, following Giroud and Bonaparte (1989a), one can assume one hole per 4000 m^2 (roughly one acre) surface area of geomembrane liner and use two different hole sizes for calculations:

- A hole size of 1 cm^2 (0.16 in^2) is recommended for sizing the components of the lining system and the design of the leakage detection, collection, and removal layers (spacing of collector pipes and their diameter) and system.

- A hole size of 3.1 mm^2 (0.005 in^2) is recommended for evaluating the performance of the liner system.

2.3.3.2 Leakage Rate Through a Composite Liner

In composite liner systems, the calculation of the leakage rate is more complex and depends on whether the geomembrane is in perfect contact with the underlying low-permeability soil layer and overburden pressure or not. Giroud and Bonaparte (1989b) provide a detailed discussion of the topic with both two- and three-dimensional analytical analysis (vertical and radial flow, respectively) and present the results of laboratory model tests. They propose four different empirical formulations, which are summarized in Table 2-11. In these formulations, k_s (m/s) is the hydraulic conductivity of low-permeability soil underlying the geomembrane, and H_s (m) is the thickness of the low-permeability soil layer.

Table 2-12 from Giroud and Bonaparte (1989a and b) shows the leakage rates for geomembrane liners and composite liners computed for cases of diffusion (permeation), a small hole, and a large hole for different liquid heads on top of the geomembrane (0.03, 0.3, 3, and 30 m). The leakage rates for different cases were calculated using equation (7), equation (8), and the equations listed in Table 2-11. The assumptions used for various parameters in the equations are listed at the bottom of the table.

Table 2-11 Equations to Calculate Leakage Rate Through Composite Liners in Different Conditions (taken from Giroud and Bonaparte 1989b).

Conditions Assumed	Equation	Remarks
Absolute minimum (vertical flow)	$Q = \dfrac{k_s a (h_w + H_s)}{H_s}$ $R = d/2$	Assumes a perfect contact between the geomembrane and the underlying low-permeability soil. R is the radius of the hole.
Perfect contact (approximate value given by radial flow)	$Q = \pi k_s h_w d$ $R = unknown$	Provides an approximate value of the leakage rate assuming a perfect contact between the geomembrane and the underlying low-permeability soil.
Excellent contact (empirical equations from model tests)	$Q = 0.7 a^{0.1} k_s{}^{0.88} h_w$ $R = 0.5 a^{0.05} k_s{}^{-0.06} h_w{}^{0.5}$	Derived by combining analytical analyses and experimental results. It is assumed to correspond to excellent field conditions.
Absolute maximum (free flow due to a large space between geomembrane and low-permeability soil)	$Q = C_B a \sqrt{2 g h_w} = 0.6 a \sqrt{2 g h_w}$ $R = 0.39 d (2 g h_w)^{0.25} k_s{}^{-0.5}$	Same as equation (8) because it is assumed that, due to the large space between the geomembrane and the low-permeability soil, the same leakage rate through a hole in a geomembrane can be assumed.

Table 2-12 Generalized Leakage Rates Through Geomembrane Liner with Defects (from Giroud and Bonaparte, 1989a and b).

Liner System	Leakage Mechanism	Leakage rate in liter per hectare per day			
		Liquid head on top of geomembrane			
		0.03 m	0.3 m	3 m	30 m
Geomembrane alone (between two sand layers)	Diffusion	0.01	1	1	1
	Small holes	300	1,000	1,000	1,000
	Large holes	10,000	30,000	30,000	30,000
Composite liner (poor field conditions, i.e., wavy)	Diffusion	0.01	1	1	1
	Small holes	0.8	6	6	6
	Large holes	1	7	7	7
Composite liner (good field conditions, i.e., flat)	Diffusion	0.01	1	1	1
	Small holes	0.15	1	1	1
	Large holes	0.2	1.5	1.5	1.5

Note: The calculations assume a frequency of one hole per 4000 m^2 (roughly an acre). Small and large holes have an area of 3.1 mm^2 (0.005 in^2) and 1 cm^2 (0.16 in^2), respectively. The HDPE geomembrane is assumed to have a thickness of 1 mm (40 mils). The liquid is water. For composite liners, it is assumed that the low-permeability underlying layer has a thickness of 0.9 m (about 3 ft) and a hydraulic conductivity of 10^{-9} m/s.

2.3.3.3 Additional Consideration for Evaluating Leakage Rates

While equation (7), equation (8), and the equations given in Table 2-11 by Giroud and Bonaparte (1989b) can be used for design purposes, a number of more recent studies have become available on the topic of leakage through composite liners, especially in waste containment facilities and landfills.

Rowe (2012a) discusses short- and long-term leakage through composite liners and observes (Rowe, 2012b) that, in the case of composite liners for landfills, the leakages observed in practice are orders of magnitude (10 to 10,000 times) larger than those calculated using available equations that assume direct contact and a specified number of holes per hectare. Touze and Barroso (2008) provide a table of empirical equations developed for estimating the flow rate through composite liners due to defects in the geomembrane layer. These equations are valid for hydraulic head ranges from 0.03 to 3.0 m.

Studies have shown that wrinkles formed on the geomembranes due to a high coefficient of thermal expansion and high bending stiffness (Giroud, 2016) can lead to a large number of defects and affect the rates of leakage through composite liners. Warm temperatures and solar radiation play a role in the formation of wrinkles. According to the experiments performed at Queen's Experimental Liner Test Site (QUELTS) in Godfrey, Ontario, Canada (Rowe et al., 2012c), the range of geomembrane surface temperatures favorable to formation of wrinkles is 20°C–40°C higher than the ambient temperature (Rowe, 2018).

A sketch for calculating leakage through a composite liner system in the presence of a wrinkle is shown in Figure 2-16. Note that the wrinkle has caused the geomembrane to lose contact with the underlying low-permeability layer, which can be a compacted clay layer (CCL) or a geosynthetic clay liner (GCL), over a width of $2b$.

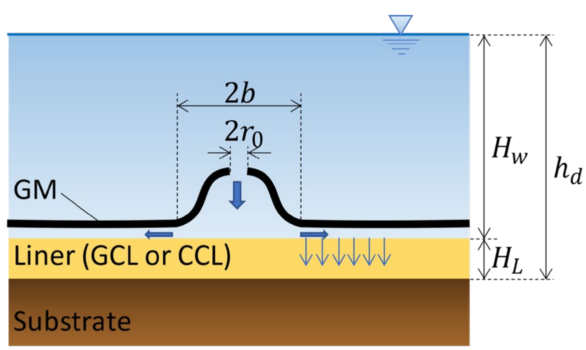

Figure 2-16 Calculating Leakage Through a Composite Liner System Due to Wrinkles in the Geomembrane (adapted from Rowe, 2012b).

Rowe (1998) provides the following equation for calculating the leakage through a composite liner with wrinkles in the geomembrane:

$$Q = \frac{L\,[k_s\, 2b + 2\,(k_s\, H_L\, \theta)^{0.5}\, h_d]}{H_L} \tag{9}$$

where Q is the leakage discharge through membrane in liters per hectare per day, L is the wrinkle length in meters, k_s is the hydraulic conductivity in m/s of the geomembrane liner, b is the half width of the wrinkle in meters, H_L is the thickness of the liner (CCL or GCL) under the geomembrane in meters, and θ is the transmissivity in m^2/s between the geomembrane and the liner (CCL or GCL). The variable h_d represents the head loss across the liner in meters. Assuming that the hydrostatic head over the liner is h_w meters, leakage is calculated by the following:

$$h_d = h_w + H_L \tag{10}$$

Additional information about the performance of geomembrane liners, especially composite liners for waste containment and landfill applications, can be found in Rowe (2005). GCLs are often used to replace the compacted clay layers in composite liners. Rowe et al. (2014) present the results of field observation on GCLs on slopes due to moisture degradation. Nasiri and El-Zein (2015) discuss the effects of the defects on reducing the desiccation potential of compacted clay layers in composite liner systems. Benson (2009) addresses the modeling of the performance and degradation of cover layers and composite liners. The field performance of exposed composite geomembrane liner systems was studied by Rentz (2015).

2.3.4 Statistical Data and Allowable Leakage Rates

A memorandum by Peppersack (2015) specifies the permissible seepage rates for unlined reservoirs in different soil groups (Unified Soil Classification System) based on the Alabama Agricultural Experiment Station Bulletin 599.

- SM (silty sand, sand silt mixtures) = 0.2 ft/day (6.1 cm/day)
- SC (clayey sands, sand clay mixtures) = 0.007 ft/day (0.21 cm/day)
- ML (inorganic silts—very fine sands, silty, or clayey fine sands) = 0.02 ft/day (0.61 cm/day)
- CL (low to medium plasticity clays) = 0.003 ft/day (0.091 cm/day)
- CH (high-plasticity clays) = 0.0003 ft/day (0.0091 cm/day).

Note that the values given above need to be multiplied by the surface area of the reservoir to obtain the permissible loss of volume due to seepage during a day. The seepage rate of 0.2 ft/day for the SM soil group is assumed to be the maximum allowable seepage rate from an unlined reservoir. According to the memorandum, the reservoirs constructed in the following types of bed material are likely to exceed this limit and thus will always require a liner:

- GW (well-graded gravels and gravel-sand mixtures)
- GP (poorly graded gravels and sandy gravel mixtures with little or no fines)
- GM (silty gravel and poorly graded gravel/sand-silt mixtures)
- GC (clayey gravels and poorly graded gravel-sand-clay mixtures)
- SW (well-graded sands and gravelly sands with little or no fines)
- SP (poorly graded sands and gravelly sands with little or no fines)

Koerner and Koerner (2009) review U.S. state regulations on allowable leakage rates from both liquid impoundments and wastewater ponds. They show that the allowable leakage rates from one state to another show large differences.

Touze-Foltz (2001) analyzed the defects of geomembrane liners based on electrical leak location system data. She reported a mean defect density of 2.8 defects/ha (1.13 defects/acre) after the installation of the geomembrane and 11.9 defects/ha (4.82 defects/acre) after placement of granular protection layer.

Based on interviews with various experts on defects in geomembrane layers, Giroud and Touze-Foltz (2003) provide the following conclusions for geomembrane liners in landfills:

- At the end of a geomembrane installation with construction quality assurance, the typical density of defects is 1-5 defects/ha. These defects are generally small holes.
- The placement of a cover layer on the geomembrane can result in from a few (say, 1 to 5) to 20 defects/ha depending on the care taken in the placement of the cover layer. The defects due to the placement of the protection layer of the geomembrane generally have larger diameters.

For this reason, Giroud (2016) recommends that for geomembranes covered by a protective layer, the electrical leak detection tests must be performed not only after the geomembrane installation but also after the placement of the cover layer.

2.3.5 Detection of Leakages and Measurement of Leakage Rates

Numerous methods and techniques are available for monitoring, detecting and locating leakage from geomembrane liners due to defects. Ling et al. (2019) give the following list of the most common leak detection methods for ponds and reservoirs:

- Volumetric and mass balancing measurements
- Statistical inventory reconciliation
- Liquid sensing probes
- Distributed fiber optic

The first two methods are based on unexplained loss of storage from the reservoir detected by performing mass balance analysis and statistical methods on data from monitoring reservoir levels. These methods may determine the presence of a leak based on an unexplained loss of volume, but cannot identify the location of the defect. Liquid sensing probes, such as conductivity meters buried at strategic locations, can be used to detect the leaks by measuring water content and pressure. Distributed fiber optic relies on detecting leaks due to temperature variations caused by leaks. The last two methods require the installation of sensors during the construction phase.

The most popular method is the electrical leak location (ELL) method, which not only determines whether there is a leak but can find the location of holes and defects. In this method, as shown in Figure 2-17, the anode (positive electrode) of a voltage source is inserted into the impounded water. For safety reasons, a high-voltage source with a very low amperage is used. The cathode (negative probe) is placed outside of the reservoir under the geomembrane to create an electric potential in the reservoir. If the geomembrane does not have any holes or defects, the electric current between anode and cathode cannot be established. If the geomembrane has a hole or a defect, the connection between the anode and cathode is established and electrical current flows. If the reservoir is shallow, a technician wading through the reservoir with a voltmeter or a dipole probe can map the locations of the holes. The voltmeter detects the location of the leak by a spike in the voltage, and with the dipole probe a sudden change in polarity indicates the location of the leak.

The diameter of the smallest hole size that can be detected by electrical leak location methods is 1 mm for an exposed geomembrane and 5 mm for a geomembrane under a 0.5 m thick protective cover layer (Giroud, 2016).

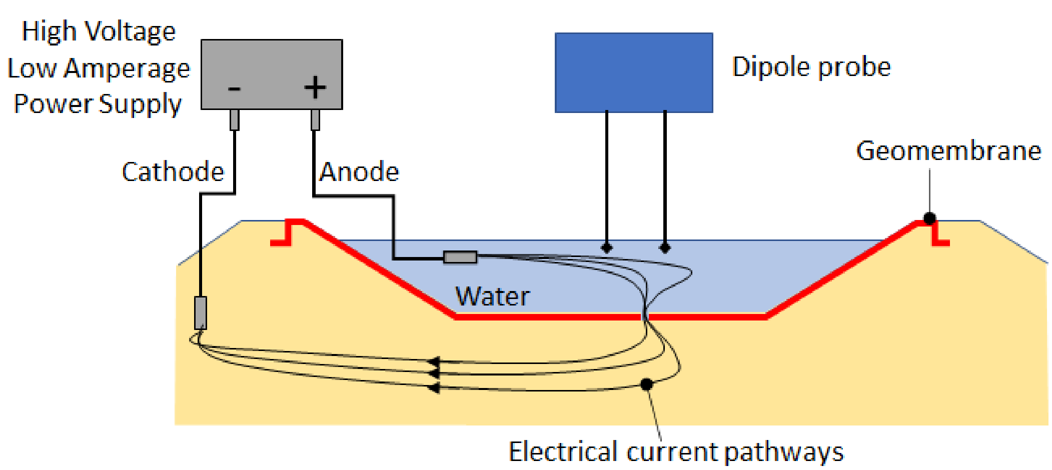

Figure 2-17 Illustration of the ELL Method.

For deep reservoirs, the ELL method can be automated to quickly scan the entire reservoir: A multi-electrode cable spanning a reservoir is moved at regular intervals. At each stop, a measuring instrument automatically scans and records dipoles along the cable. When this process is repeated at right angles to the first scan, the result is a grid of fixed interval dipole measurements for the reservoir. At the location of a leak, the voltage map shows a sudden change in polarity.* Using this method, holes having a diameter as small as one centimeter or less can be detected. The success of the method, however, depends on various factors, some of which are specific to the location of the reservoir. If conductive paths in the reservoir, such as pipes, pumps, etc., are not electrically isolated, this leak detection method would not give reliable results near those features. The ELL methodology is described in ASTM D7007-03.

Darilek and Laine (1989) discuss electrical leak detection location surveys, including both technical aspects and site preparation that helps avoid errors. Interested readers will find additional information on ELL in Koerner et al. (2016). Ling et al. (2019) carried out laboratory experiments as well as field tests on electrical leak detection methods. They used sandbox tests to study the errors in the detection of voltage polarity and compared “mise-à-la-masse” with electrical resistivity tomography methods.

The geomembrane liner system may also be designed to reduce leakage due to defects in the geomembrane liner by associating the geomembrane with other materials (Giroud, 2016).

- Association of the geomembrane with a geotextile layer provides it with additional protection against punctures and dents from protruding elements or stones in the cover layer. In practice, nonwoven geotextiles are generally chosen to be associated with geomembranes. Geotextiles with a mass per unit area ranging from 500 g/m^2 to 1,000 g/m^2 are typically used. In special cases, the use of a geotextile with a density of 2,000 g/m^2 may be warranted.

- A double liner system—a drainage layer sandwiched between a top and bottom geomembrane layer—is more suitable for liquid containment reservoirs. The drainage layer between the top and bottom geomembranes serves as a leakage collection and detection layer. This concept was originally proposed by Giroud (1973). The top or primary geomembrane is subjected to the high head of stored water above while the bottom or the secondary geomembrane is generally subject to a very low head. It is this low head over the secondary geomembrane that ensures a very low leakage into the subgrade. The double liner provides additional advantages. Any leakage through the primary geomembrane can be detected and measured to monitor its integrity. The liquid collected from the drainage layer can be treated if it is contaminated, or, if it is clean, it can be pumped back into the reservoir, minimizing the loss of storage. Various materials with an appropriate porosity can be used as a leakage collection layer, which may also include collector pipes. An example case discussed by Giroud (2016) uses 0.20 m thick rounded gravel stabilized with a small amount of mortar.

* Sean (2020) shows a typical voltage map for a single hole, generated using this automated scanning method.

2.4 Structural Design of Liner Systems with Geomembranes

This section will briefly outline the basic considerations and the main steps in designing a geomembrane liner for a water-impounding surface reservoir. Detailed information with numerical examples can be found in Koerner (2016a and b) and various other papers and publications. USBR (2018) also describes some of the methodologies described in this section. It is also important to note that the design methods discussed in this section are rather simple analytical models for demonstrating the general principles of analysis. They can be used for a preliminary analysis and design. Real-life design of liner systems with geomembranes considers more realistic foundation soil characteristics and more complex geometries, and it is often performed using various commercially available software products.

2.4.1 Dimensions of the Reservoir

The first step in designing a reservoir is the selection of the reservoir geometry based on the volume to be impounded and the area available for the construction of the reservoir. The geotechnical and mechanical properties of the soil on which the reservoir will be constructed must also be known, as this information determines the quality of the subgrade as a foundation. The friction angle of the material will be used to decide the slope ratio (horizontal to vertical) to be used. In addition, the dominant wind direction and speed are also important design parameters in selecting the freeboard to be used.

Consider the rectangular reservoir shown in Figure 2-18. Let the length L and the width W on the ground surface define the limits of the available area. Let a be a suitable freeboard height considering the surcharge of the reservoir and the wave height due to the wind. Based on the characteristics of the soil, a slope ratio of $S = h/v$ is decided on, where v is the vertical rise on the side slope for a horizontal distance of h. For a given height H, the volume of the excavation (V_e) is given by the following (Koerner, 2016b):

$$V_e = HLW - SH^2L - SH^2W + 2S^2H^3 \quad (11)$$

Note that the height of the excavation is equal to the sum of water depth D and the freeboard height:

$$H = D + a \quad (12)$$

Thus, the length and width at the reservoir surface are $L - 2Sa$ and $W - 2Sa$, respectively. The volume of stored water is calculated as follows:

$$V_w = D\,(L - 2Sa)\,W - SD^2\,(L - 2Sa) - SD^2\,(W - 2Sa) + 2S^2D^3 \quad (13)$$

Equation (13) can also be used to find the water depth D and the excavation depth H corresponding to a specified volume. These formulas are for the special case of a rectangular reservoir with the same side slope on all edges, and they can be useful for a preliminary assessment of the dimensions that may be required. Real life reservoirs may have more complex shapes dictated by the available space and topography. In some cases, it may be necessary to include berms or to have different side slopes along different edges. In some cases, the excavation volume can be reduced by constructing embankments around the reservoir using, if possible, the excavated material, at least partially. For these special geometries, suitable geometric relationships must be worked out to calculate the optimal depth and surface area.

It is important to note that the positioning of the reservoir and its optimal dimensions are not decided based solely on geometrical considerations. Designing a large reservoir and setting optimal depth to create the desired storage volume requires also consideration of the position of the water table and its interception, difficulties related to the stabilization of the side slope during construction when the water table is high, cost of excavation, disposal of the excavated soil, etc.

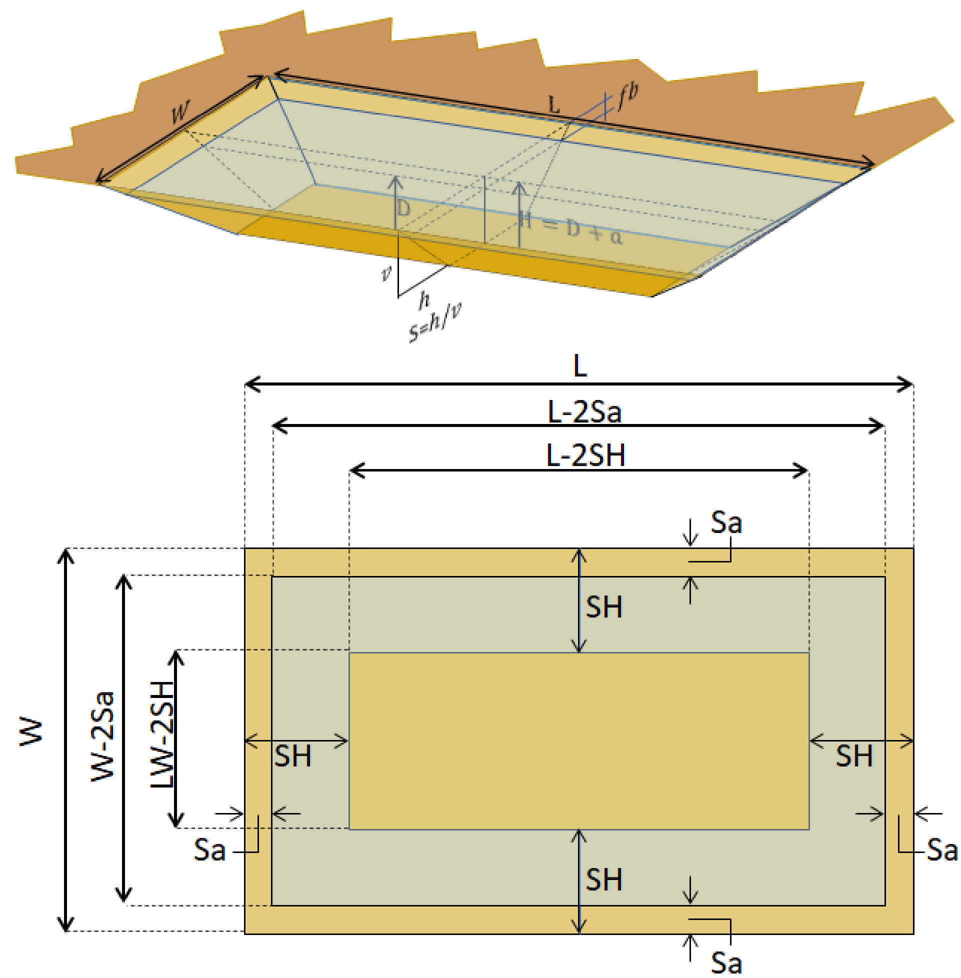

Figure 2-18 Definition Sketch for a Rectangular Reservoir.

2.4.2 Determination of the Thickness of the Geomembrane

When the geometry of the reservoir and its depth are determined, it is necessary to select the thickness of the geomembrane that will serve as the barrier layer. Koerner (2016b) criticizes the use of guidelines for the selection of the thickness of geomembrane based only on water depth.

He argues that the selection of the thickness of the geomembrane should be decided based on the subsurface deformation that will create tension in the geomembrane. Such subsurface deformation may be due to random differential settlement, settlement of backfills under the geomembrane, local settlement of certain areas of soft soils under the geomembrane, or major events such as earthquakes. While it is true that this method also considers the pressure exerted by the depth of water on the geomembrane, it considers various other factors and follows a rigorous procedure. Once the tension in the geomembrane is calculated, the thickness is determined based on the allowable stress and a suitable factor of safety.

Figure 2-19 illustrates a deformation of the subgrade and the tension force that develops in the geomembrane if it remains in contact with the sublayer. The deformation, which has an angle β with the horizontal, creates a tension T in the geomembrane. The vertical component of this tension is considered as a downward normal force of $F_G = T \sin \beta$ acting on the geomembrane. Since this force is dissipated over the mobilization distance x, the force is represented as a triangular distributed load with an area equal to F_G. The mobilization distance is measured using the methodology described in ASTM D6706 (ISO 13430), which is the standard test method for measuring geosynthetic pullout resistance in soil. Note that the same test is used to determine the embedment distance for anchorage as discussed in Section 2.2.3.7.

Figure 2-20 shows typical values of mobilization distance curves for various types of geomembranes compiled from two different sources. It is interesting to see how short the mobilization distance is, which allows us to conclude that geomembranes should never be used as reinforcement. Over the mobilization distance, the geomembrane is also subjected to normal force due to the pressure of the stored water and the cover soil, F_L.

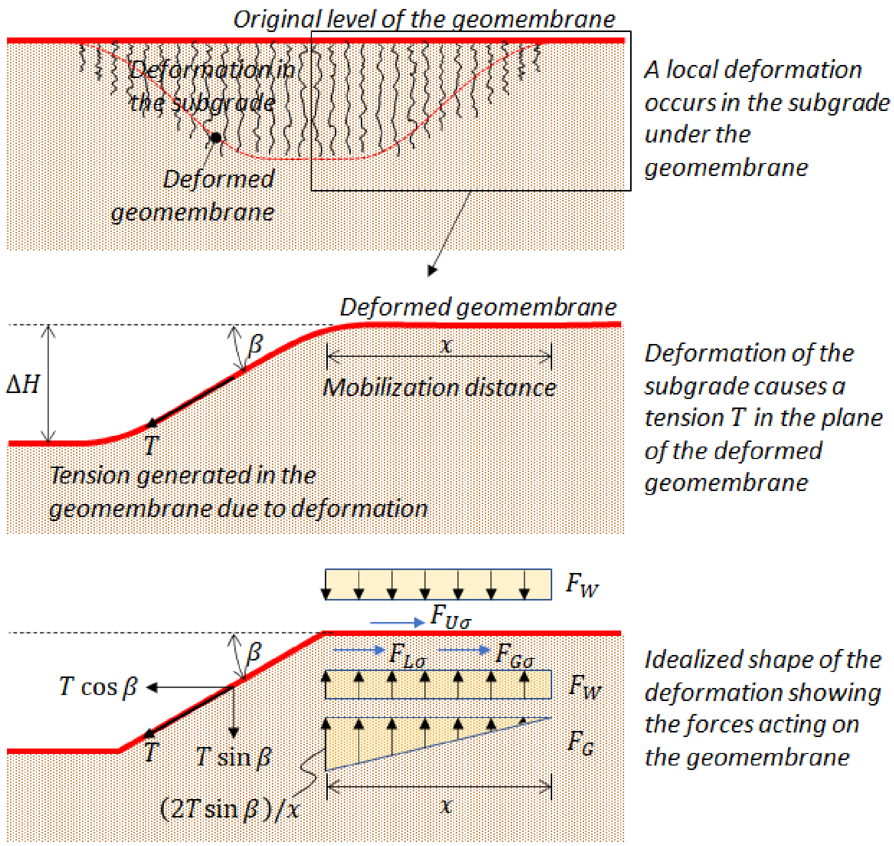

Figure 2-19 Analysis of the Tension in the Geomembrane Due to a Deformation of the Subgrade, Used to Calculate the Required Geomembrane Thickness (Koerner, 2016b).

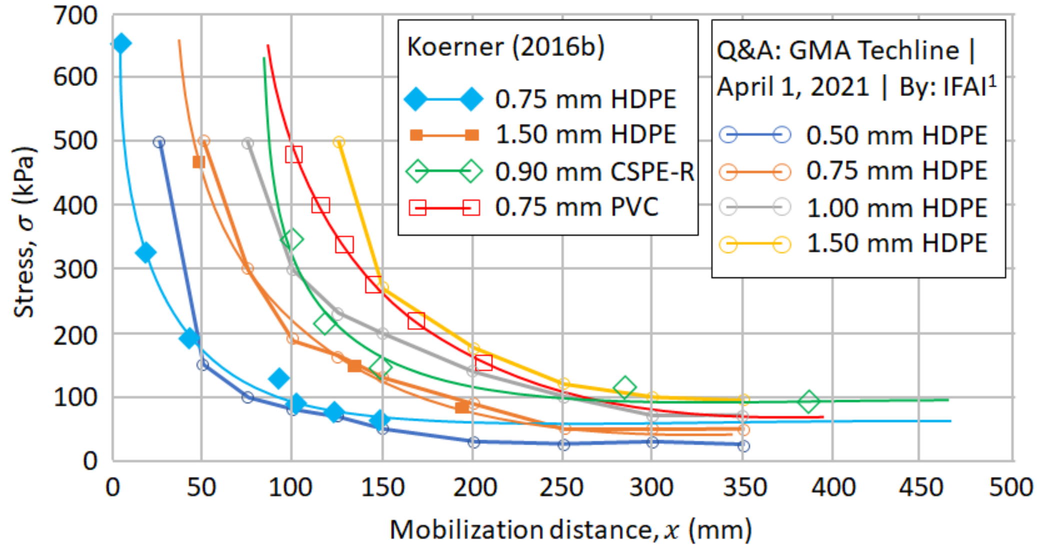

Figure 2-20 Mobilization Distance (Also Embedment Depth for Anchorage) for Different Types of Geomembranes as Determined Based on ASTM D6706 (ATA, 2021).

The tension T in the geomembrane is calculated by considering the equilibrium of forces in the horizontal direction:

$$T\ \cos\beta = F_{U\sigma} + F_{L\sigma} + F_{G\sigma} = x\left[F_W\ \tan\delta_U + F_W\ \tan\delta_L + \left(\frac{T\sin\beta}{x}\right)\tan\delta_L\right] \qquad (14)$$

where $F_{U\sigma} = x\,F_W\ \tan\delta_U$ is the shear force above the geomembrane due to the weight of water and the cover soil, $F_{L\sigma} = x\,F_W\ \tan\delta_L$ is the shear force below the geomembrane due to the weight of water and the cover soil, and $F_{G\sigma} = (T\sin\beta)\ \tan\delta_L$ is the shear force below the geomembrane due to the vertical component of the shear force T in the geomembrane. The terms $\tan\delta_U$ and $\tan\delta_L$ represent the angle of shearing resistance between the geomembrane and the adjacent material above and below it, respectively. It is important to note that if the geomembrane is exposed to water or covered with only a thin layer of soil, we have $\delta_U \cong 0$. Rearranging equation (14), we obtain the expression for the tension in the geomembrane due to the deformation of height ΔH with angle β:

$$T = \frac{F_W\ x\ (\tan\delta_U + \tan\delta_L)}{\cos\beta - \sin\beta\ \tan\delta_L} \qquad (15)$$

Let the allowable tension in the geomembrane σ_{allow} be determined from the wide-wide tensile behavior test of the geomembrane according to ASTM 4883 (see Section 2.2.3.1 and Figure 2-9).

Using a suitable factor of safety, we have the following equation:

$$T = \sigma_{allow}\, t \tag{16}$$

Note that working with the allowable stress (σ_{allow}) instead of the ultimate stress (σ_{ultim}) implicitly builds in a safety factor of SF = $\sigma_{ultim}/\sigma_{allow}$. The thickness of the geomembrane is then given by the following:

$$t = \frac{T}{\sigma_{allow}} = \frac{F_W\, x\, (\tan\delta_U + \tan\delta_L)}{\sigma_{allow}\,(\cos\beta - \sin\beta\, \tan\delta_L)} \tag{17}$$

2.4.3 Design of Side Slopes

Typical designs of geomembrane lining systems include sloping reservoir sides. Designing the side slopes of a reservoir with a geomembrane liner system requires addressing two issues: 1) stability of the side slope from a soil engineering design point of view, and 2) stability of the cover layer placed on the geomembrane as protective layer.

Slope stability is a widely studied soil mechanics and geotechnical engineering subject. A detailed treatment of the topic is outside the scope of the present report; however, detailed information can be found in various textbooks on soil mechanics and geotechnical engineering. Most of the traditionally available methods are based on limit equilibrium analysis using the "slices" method shown in Figure 2-21. The general approach is to estimate a slip boundary, which can be an arc or a circle (as in the Swedish circle method) and check the safety factor of the block above the slip boundary against sliding along that boundary. Different types of slip boundaries, i.e., failure scenarios, should be checked for their factor of safety. If the lower end of the slip boundary terminates away from the toe, it is called a base failure scenario. If the lower end terminates at the toe, it is a toe failure scenario. If the slip circle begins and ends on the face of the slope, it is a slope failure scenario.

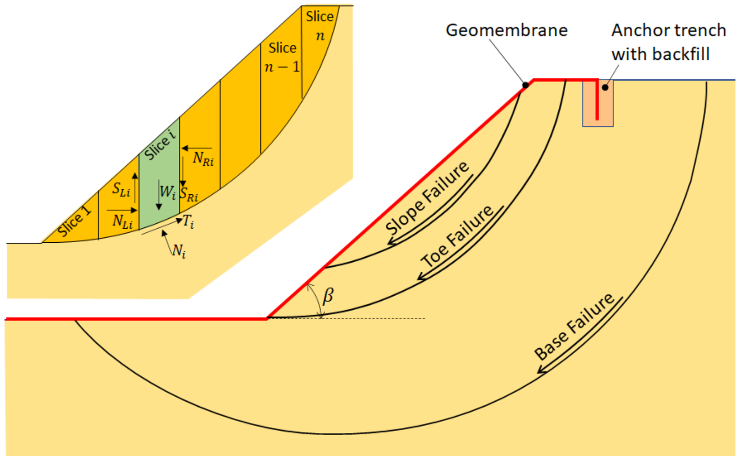

Figure 2-21 Slope Failure Types and the Limit Equilibrium Analysis Using Slices Method (Koerner, 2016b).

The safety factor is computed by dividing the mass above the slip boundary into vertical slices as shown in the inset in Figure 2-21. Considering a total stress analysis (which assumes that the slope is above the groundwater table), the forces acting on the slice i are the weight of the slice, W_i, normal force N_i on the at the bottom of the slice touching the slip circle, the shear stress T_i resisting slip failure, the interslice normal forces on the right and left faces of the slice, N_{Ri} and N_{Li}, and the interslice shear forces T_{Ri} and T_{Li} on the right and left faces of the slice. For each slice i, the driving forces promoting sliding and the resisting forces working against the sliding are calculated from these basic forces.

The safety factor for each slice is calculated as the ratio of the sum of the resisting forces of all slides to the sum of the driving forces promoting the sliding failure. Several methods are traditionally available. They differ depending on which of the above listed forces they consider and which they ignore. It is possible to perform the analysis for multiple layers of soil with differing characteristics and considering the pore water pressure due to groundwater. The calculations for finding the slip boundary with the lowest safety factor are tedious and are no longer performed manually. Numerous software products are available for performing slope stability analysis; these use a realistic representation of the geometry and the soil and groundwater conditions.

2.4.4 Design of Cover Layers on Side Slopes

Geomembrane liner systems may include a cover layer to prevent damage due to UV light, temperature variations, damage by sharp objects, vandalism, etc. The cover layer is also useful for protecting the geomembrane from uplift due to wind or to gases that may form beneath the geomembrane from decomposition of organic matter in the subgrade.

The cover layer can have a constant thickness or a tapered thickness, as shown in Figure 2-22. In both cases, the safety factor is calculated from the equilibrium of forces in vertical and horizontal directions. It is important to recognize that in both cases the cover layer is subdivided in an active wedge and passive wedge. The active wedge rests on the slope and has a tendency to slide down over the geomembrane, which is assumed to not move because it is anchored at ground level at the top of the slope. Note that a tension crack is assumed at the top of the slope which breaks the continuity of the soil at the top edge of the slope. Since the side slope of the reservoir has a finite length, the triangular area at the lower part of the cover layer is resting on the horizontal surface at the bottom of the reservoir. This passive wedge resists the downslope sliding tendency of the active wedge. All the forces acting on the cover layer are shown in Figure 2-22.

Details of the stability analysis for constant thickness and tapered thickness cover layers and design curves can be found in Koerner (2016b). Without going into the details of the derivation, in both cases the safety factor is the solution of a quadratic equation. Table 2-13 summarizes the coefficients of the quadratic equation and the variables. Although not noted in Table 2-13, for both active and passive wedges the computation of weight, effective normal force normal to the failure plane, adhesion to geomembrane, and some lengths is done using the geometry and dimensions shown in Figure 2-22 together with the adhesion coefficients and soil properties.

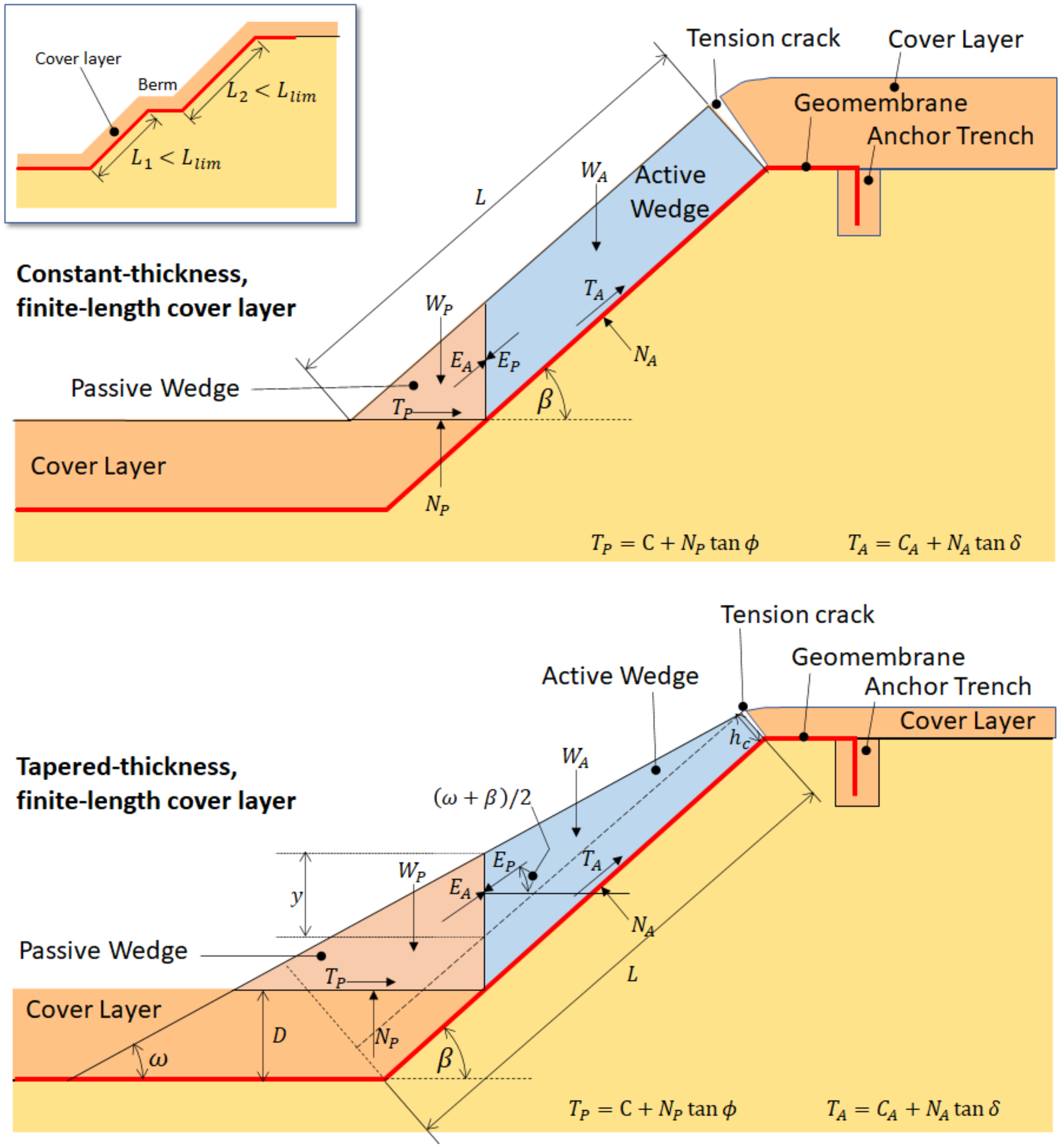

Figure 2-22 Stability Analysis of Constant Thickness and Tapered Thickness Geomembrane Cover Layers of Finite Length (USBR, 1992; Koerner, 2016b).

Table 2-13 Safety Factor for Constant- and Tapered-Thickness Finite Length Soil Covers.

<table>
<tr><td colspan="4">$$\boldsymbol{FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}}$$</td></tr>
<tr><td colspan="2">Constant thickness, finite length slope</td><td colspan="2">Tapered thickness, finite length slope</td></tr>
<tr><td colspan="2">$a = (W_A - N_A\ cos\,\beta)$</td><td colspan="2">$a = (W_A - N_A\ \cos\beta)\cos\left(\frac{\omega + \beta}{2}\right)$</td></tr>
<tr><td colspan="2">$b = -[(W_A - N_A\ \cos\beta)\sin\beta\tan\phi + (N_A\tan\delta + C_a)\sin\beta\cos\beta + (C + W_P\tan\phi)]$</td><td colspan="2">$b = -\left[(W_A - N_A\cos\beta)\sin\left(\frac{\omega+\beta}{2}\right)\tan\phi + (N_A\tan\delta + C_a)\sin\beta\cos\left(\frac{\omega+\beta}{2}\right) + \sin\left(\frac{\omega+\beta}{2}\right)(C + W_P\tan\phi)\right]$</td></tr>
<tr><td colspan="2">$c = -[(N_A\tan\delta + C_a)\sin^2\beta\tan\phi]$</td><td colspan="2">$c = -\left[(N_A\tan\delta + C_a)\ \sin\beta\ \ \sin\left(\frac{\omega+\beta}{2}\right)\tan\phi\right]$</td></tr>
<tr><td>W_A</td><td>Total weight of active wedge</td><td>W_P</td><td>Total weight of passive wedge</td></tr>
<tr><td>N_A</td><td>Effective normal force normal to failure plane of active wedge</td><td>N_P</td><td>Effective normal force normal to failure plane of passive wedge</td></tr>
<tr><td>ϕ</td><td>Friction angle of the cover soil</td><td>δ</td><td>Friction angle between cover soil and geomembrane</td></tr>
<tr><td>β</td><td>Soil slope angle under the geomembrane</td><td>ω</td><td>Tapered-thickness cover layer angle ($\omega < \beta$)</td></tr>
<tr><td>C_a</td><td>Adhesion force between active wedge and geomembrane</td><td>C</td><td>Adhesive force under the passive wedge</td></tr>
<tr><td>D</td><td>Thickness of cover slope at the bottom of the slope</td><td>h_c</td><td>Thickness of cover slope at the top of the slope</td></tr>
</table>

2.4.5 Rapid Drawdown and Cover Layer Design

In some ways the cover layer constitutes a dilemma for the design engineer. The cover layer is necessary to protect the geomembrane against weathering, wind uplift, accidental damaging, vandalism, and other outside factors. On the other hand, a cover layer is also a source of problems when the reservoir is subject to rapid drawdown and filling, as in the case of water storage reservoirs for PSH facilities.

When the water in the reservoir is rapidly lowered during hydropower generation, the cover layer must also allow the release of pore water (i.e., the water held within it) sufficiently quickly to prevent a buildup of pore water pressure. Build-up of excess pore water pressure may endanger the stability of the cover layer.

To prevent excessive pore water pressure in the cover layer, the cover layer material must have a minimum conductivity (USBR, 2018):

$$k_{min} = \frac{1}{\sin^2\beta}\,\frac{dh}{dt} \tag{18}$$

where β is the side slope angle. The term dh/dt represents the time rate of change of water depth in the reservoir due to drawdown.

Let the storage versus elevation and the surface area versus elevation curves for the reservoir be denoted by $V(h)$ and $A(h)$, respectively. Then the discharge Q released from the reservoir at a time t can be written as follows:

$$Q = \frac{dV(h)}{dt} \tag{19}$$

Since the infinitesimal volume at the top of the reservoir can be written as $dV = A(h)dh$, the rate of drawdown and the discharge from the reservoir are related as follows:

$$\frac{dh}{dt} = \frac{Q}{A(h)} \tag{20}$$

Inserting equation (20) in equation (18) provides the following expression for the minimum conductivity to prevent excess pore water pressure during rapid drawdown

$$k_{min} = \frac{1}{\sin^2 \beta} \frac{Q}{A(h)} \tag{21}$$

Let us now consider a reservoir side slope of 1.0V:1.5H (33.7°) and $\frac{dh}{dt} = 0.0005\ \mathrm{m/s}$. Inserting these values into equation (18), we obtain a hydraulic conductivity value of 0.0018 m/s or 0.18 cm/s. As shown in Figure 2-1, this hydraulic conductivity value is achieved only for the upper range of high-conductivity soils beginning at the upper range of clean sand and clean sand and gravel mixes (GW, GP, SW, SP, and SM) and extending into clean gravel (GP). The presence of silt or clay must be avoided, thus the soil must be "clean."

The equations presented above are suitable for a preliminary verification. The design of the stability of cover soil is normally performed using more sophisticated analysis techniques, and several commercially available software packages based on numerical modeling are available for this type of analysis. Using these software packages, the design engineer can analyze the cover layer under different drawdown scenarios, including the theoretical case of instantaneous removal of the reservoir water ($dh/dt = \infty$).

Numerous papers and documents are available for further reading on this topic. Johansson (2014) extensively discusses the impact of water level variations on the stability of slopes. Nuriya et al. (2008) reviews methods available for evaluating the stability of slopes and embankments for rapid drawdown conditions. They criticize the classical methods of the "stress-based" undrained approach (for impervious materials) and the "flow" approach for rigid pervious materials (granular soils) for being too simplified to take into account real conditions in the field. They promote a fully coupled flow-deformation approach.

NRCS (2019) recommends transient seepage analysis for the study of the stability of slopes under rapid drawdown conditions. Fredlund et al. (2011) compares the traditional limit equilibrium methods to combined transient seepage and slope stability analysis. They conclude that the traditional limit equilibrium provides results similar to those provided by more sophisticated methods. Appendix G of the U.S. Army Corps of Engineers (USACE) Engineer Manual (EM) 1110-2-1902 discusses two procedures for rapid drawdown analysis.

2.4.6 Design of Runout Distance and Anchor Trench

The last step in the preliminary design of the geomembrane liner system is anchoring the geomembrane at ground level at the top of the slope by sandwiching it between the subgrade and the cover layer. Figure 2-12 shows different methods used for anchoring geomembranes. The simplest design provides a horizontal runout length that can hold the geomembrane in place by means of the friction between it and the subgrade and cover soil on its lower and upper surfaces. In some cases, a V-shaped depression or a trench is excavated, and the geomembrane is wrapped partially (I and L shapes) or completely (U shape) along the inside perimeter of the trench. The trench is then backfilled with the excavated material laid in layers, and each layer is compacted. In some cases, concrete is poured into the trench. Koerner (2016b) argues that this is unnecessary, as other types of designs can be used to provide adequate anchorage.

Figure 2-23 shows the free-body diagrams used in the analysis of two different configurations. The top illustration shows the configuration with only a runout section of length L_{RO} (top). The bottom illustration shows a configuration where the runout section of length L_{RO} is followed by an anchor trench with the “I” configuration, and the length of the geomembrane embedded in the anchor trench is d_{at}. In both cases the thickness of the cover layer is d_{CL}.

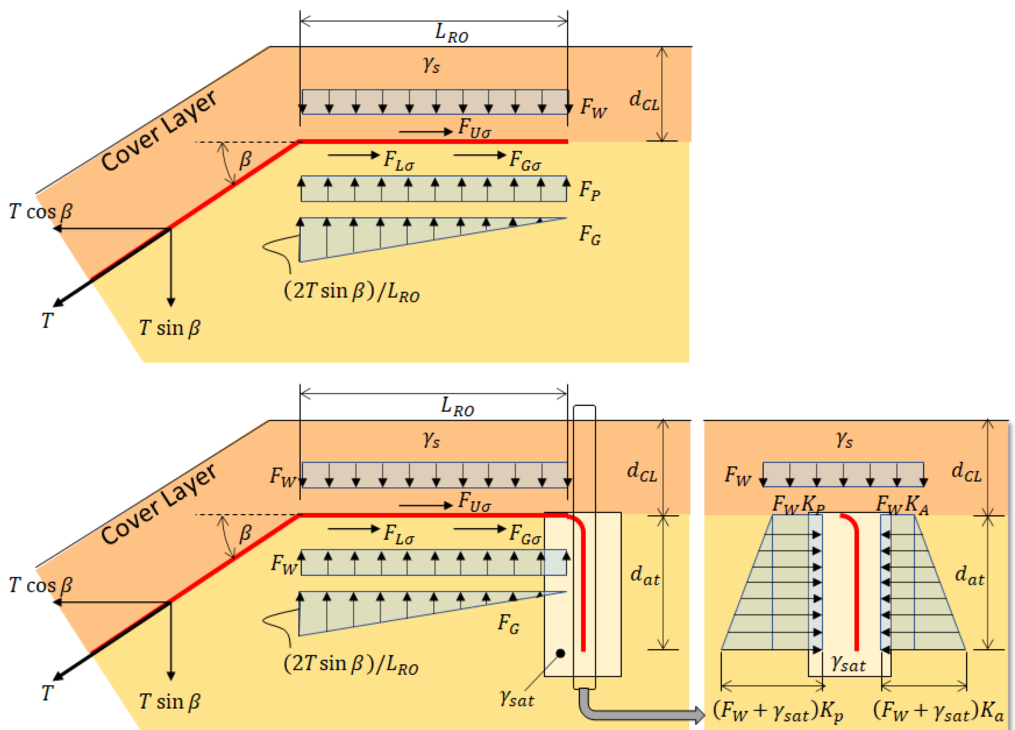

Figure 2-23 Analyzing the Runout Section with Runout Distance (Top) and with Anchor Trench in “I” Configuration (Bottom) (USBR, 1992; Koerner, 2016b).

In the case of using only a runout length of geomembrane (top illustration in Figure 2-23), the method for determining the runout length L_{RO} needed to properly anchor the geomembrane is similar to that used for determining the thickness of the geomembrane (discussed in Section 2.4.2). In the scenario in Figure 2-23, the equilibrium of forces in the horizontal direction is written as follows:

$$\begin{aligned} T_{allow} \cos\beta &= T\, \sigma_{allow} \cos\beta = F_{U\sigma} + F_{L\sigma} + F_{G\sigma} \\ &= L_{RO} \left[F_W \tan\delta_U + F_W \tan\delta_L + \left(\frac{T \sin\beta}{L_{RO}}\right) \tan\delta_L \right] \end{aligned} \tag{22}$$

where $F_W = \gamma_s d_{CL}$ is the weight of the cover soil above the runout length for the unit width, and γ_s is the unit weight of the cover soil, $F_{U\sigma} = L_{RO}\, F_W \tan\delta_U$ is the shear force above the geomembrane due to the pressure of water and the cover soil, $F_{L\sigma} = L_{RO}\, F_W \tan\delta_L$ is the shear force below the geomembrane due to the weight of the cover soil, and $F_{G\sigma} = (T \sin\beta) \tan\delta_L$ is the shear force below the geomembrane due to the vertical component of the shear force T in the geomembrane. The terms $\tan\delta_U$ and $\tan\delta_L$ represent the angle of shearing resistance between the geomembrane and the adjacent material above and below it, respectively. The reader is invited to compare equation (22) with equation (14), also derived using horizontal equilibrium of forces. The manipulation of equation (22) yields the expression for the required runout length:

$$L_{RO} = \frac{T_{allow} (\cos\beta - \sin\beta \tan\delta_L)}{F_W (\tan\delta_U + \tan\delta_L)} \tag{23}$$

As implied by the bottom illustration in Figure 2-23, we also need to consider the presence of the vertical geomembrane sheet in the anchor trench. Some simplifying assumptions are made about the computation of driving and resisting stresses acting on the geomembrane in the trench. In the trench, the geomembrane is pulled to the left by the tension force T, which creates an active lateral soil pressure on the right and passive lateral soil pressure on the left. If the soil in the anchor trench has a unit weight of γ_{sat}, the triangular distribution of active lateral soil pressure on the right has a base value of $\gamma_{sat} K_a$, where K_a is the active earth pressure coefficient. The triangular distribution of passive lateral soil pressure on the left has a base value of $\gamma_{sat} K_p$, where K_p is the active earth pressure coefficient. The active pressure force is considered a destabilizing force, and the passive pressure is a force resisting the pull of the geomembrane. The equilibrium of forces can be written as follows:

$$\begin{aligned} T_{allow} \cos\beta &= T\, \sigma_{allow} \cos\beta = F_{U\sigma} + F_{L\sigma} + F_{G\sigma} - P_A + P_P \\ &= L_{RO} \left[F_W \tan\delta_U + F_W \tan\delta_L + \left(\frac{T \sin\beta}{L_{RO}}\right) \tan\delta_L \right] \\ &\quad - (F_W + 0.5\, \gamma_{at} d_{at}) K_a d_{at} + (F_W + 0.5\, \gamma_{at} d_{at}) K_p d_{at} \end{aligned} \tag{24}$$

This equation has two unknowns, the runout length L_{RO} and the vertical length of the geomembrane embedded in the anchor trench, d_{at}. For design purposes, it is necessary to perform a few iterations by assuming one and finding the other until a suitable combination is found.

2.5 Non-Destructive Field Tests for Geomembrane Liners

The performance of a geomembrane liner system does not depend solely on the material properties of its components. In these liner systems, geomembranes serve as the primary barrier for the containment barrier. Although geomembrane liners have extremely low permeability as a material, leakage can be caused by defects in the geomembrane layer and construction defects in the support and cover layers. Some manufacturing defects in geomembranes are inevitable but normally are minimal. However, the geomembrane comes in rolls that are transported to the site and stored until they need to be installed, and inappropriate handling and transport may cause defects in the geomembrane liner even before it is placed in the field. The placement of the geomembrane and the welding of the seams are also important steps at which damage can happen.

Field tests are performed to assess the integrity of seams and to detect leaks[*] in the geomembrane sheets and seams. The tests can be destructive and non-destructive. This section will only deal with nondestructive field tests, following the summary provided by Darilek and Laine (2012), who review nondestructive field tests in two categories: tests for construction quality control and tests for locating leaks.[†]

2.5.1 Tests for Quality Control of Seams

Non-destructive field tests have been devised for testing seam quality in the construction phase. These tests assess the quality of seam execution, i.e., the continuity of the seam, in providing a waterproof bonding between the geomembrane rolls. They do not, however, provide any information on the seam bond's resistance to shearing or peeling action. The tests for quality include the following:

- Vacuum box testing for extrusion welds is described by the ASTM D5461 standard and uses a rectangular box with a transparent top and open bottom connected to a vacuum pump. The section of the extrusion weld to be tested is first wetted with a soapy solution. The open bottom of the box is placed on this section of the seam and the vacuum pump is operated to create a partial vacuum. If the seam has a leak, the soapy solution forms bubbles. This test is only applicable on flat surfaces, requires a skillful operator to get good results, and is slow. It is only used for selected patches.

- Pressure testing can be used on seams created using double track fusion welders that create two closely spaced seam lines with a hose-like empty space in between. To test the quality of the seams, the ends of the hose-like empty space are closed. The pressure testing equipment has an air pump fitted with an outlet valve. One end of a hose equipped with a pressure gauge is connected to the outlet valve of the pump, and a hollow needle on the other end is used to pierce the top liner between the seam tracks. Air is pumped into the empty area to a specified pressure, and then the valve is closed. The pressure

[*] According to ASTM, "leaks" are "holes, punctures, tears, knife cuts, seam defects, cracks, and similar breaches in an installed geomembrane."

[†] For a video tutorial on these methods, see https://www.youtube.com/watch?v=5yau12JIoTI.

gauge is used to monitor the pressure drop in the empty area between the seams. The standard test procedure is described in ASTM D5820. For PVC geomembranes, the standard test procedure is described in ASTM D7177.

- Air lance testing is used to test the seams of flexible geomembranes. The test and the standard procedure are described in ASTM D4437. For this test, an air compressor is fitted with a hose terminating in a hollow rigid tube with an elbow. The person conducting the test walks along the seam line with the end of the rigid tube inserted between overlapping flaps of the geomembrane, facing the seam line, and listens to the sound the air makes hitting the seam line. When a defect in the seam line is encountered, the unwelded geomembranes vibrate and create a different sound, which is detected by the operator.

- The spark test is used for testing geomembrane seams that cannot be tested using other non-destructive test methods, such as seams in corners, curved surfaces like connections to penetrating pipes, and other difficult to reach places. According to the standard procedure described by ASTM D6365, a conductor is embedded in the upper geomembrane and the other end of the conductor is grounded. A conductive probe or brush attached to a holiday tester of several thousand volts (but limited current) is passed along the well bead. When there is a faulty weld, the air gap creates a spark, which is then registered. The use of direct current gives better results, but in practice alternating current is often used.

2.5.2 Tests for Locating Leaks in Geomembrane Layers and in Seams

Different test methods are available depending on whether the geomembrane is exposed or not, or underwater or not.

- The water puddle method is used to detect leaks in exposed geomembranes and their seams when the geomembrane is dry and exposed to air. One end of a low-voltage electrical power source is grounded in the sublayer under the geomembrane and the other end is connected to a squeegee held by the operator. The operator pushes a puddle of water over the exposed geomembrane surface using the squeegee. When the water puddle encounters a hole in the geomembrane, it leaks below and completes the circuit, causing a current. An electronic circuit records the changes in the current and converts it to an audible signal to be monitored by the person performing the test. Darilek and Laine (2012) reports that a hole with a diameter as small as 1 mm (0.04 in) can be detected using this method. A variant of this method uses a water lance instead of a water puddle pushed by a squeegee.

- A special type of geomembrane has a conductive surface on the underside in contact with the subgrade layer. A variant of the spark method described earlier in non-destructive testing of seams can be used to test for leaks in these geomembranes when they are exposed to air, following the method described ASTM D7240. When a wide metallic brush attached to a holiday tester sweeps the geomembrane, the detection of a spark indicates a defect.

- ASTM 7007 defines the standard practice for detecting leaks in an exposed underwater geomembrane. To apply a voltage across the geomembrane liner, one end of a power supply is connected to the water in the pond and the other end is grounded in the subgrade (if it is a double liner system, the grounding edge should be inserted into the drainage and collection layer between the two membranes). To detect smaller holes, it is necessary to use a higher voltage. Since normally the geomembrane is not a conductor, the applied voltage creates an electrical field within the pond. The location of a leak can be detected based on changes in voltage or polarity of voltage when a dipole probe is moved over the surface of the underwater geomembrane (see Section 2.3.5 for more details on electrical leak detection and localization).

2.6 Failure Modes for Geomembrane Liners

There are three common failure mechanisms for geomembrane liners due to problems with the geomembrane material itself (XR Geomembranes Blog, 2019).

- *Puncturing:* During or after the installation, angular rocks in the subgrade layer or the cover layer, tools dropped on the geomembrane layer, the traffic of workers and construction equipment, animal activity, and other factors can cause punctures. In some cases, a geotextile layer may be used to provide additional protection, but the choice of the geotextile must be made carefully and in consultation with the geomembrane manufacturer.

- *Environmental stress cracking (ESC):* ASTM D883 defines ESC as "an external or internal crack in a plastic caused by tensile stresses less than its short-term mechanical strength." Once it starts, environmental stress cracking can propagate through the liner and compromise its integrity. It is known that molecular bonding, crystalline structure, polymer type, environmental factors, and UV radiation all play a role in ESC. However, how these factors combine to create ESC is not yet well enough known to allow making predictions. Geomembranes with low thermal expansion properties may provide a better resistance to environmental stress cracking.

- *Wicking:* In reinforced geomembranes, wicking is the separation of the coating from the base fabric or the yarns. In non-reinforced geomembranes, wicking is the peeling of the uniform material from itself. Wicking depends on the type of the geomembrane and its manufacturing. Consultation with the manufacturer and supplier is recommended when selecting a geomembrane type for a specific application to avoid failure due to wicking.

Nosko and Touze-Foltz (2000) studied geomembrane failure mechanisms using data from more than 300 sites located in 16 countries and covering an installed geomembrane area of 3,250,000 m^2. These sites were instrumented with an electrical damage detection system, and the data were mainly obtained using SENSOR DDS® technology (Nosko and Ganier, 1999). They carried out statistical analysis of the data based on three criteria: location of the damage, size of the damage, and cause of the damage. Although the study focused on geomembrane liners in landfills, the results of the statistical analysis are of interest to all types of applications. Detailed results of the statistical analysis can be found in the original article by Nosko and Touze-Foltz (2000). Table

2-14 summarizes their analysis of the location and cause of the damage. It can be seen that puncturing due to stones is the most common cause of the damage (71.14%), and it mostly happens on flat areas (77.76%).

Solsky and Bykovskaya (2021) discuss Russian experiences with geomembrane liners in hydraulic structure applications and found that the main causes of structural failures are related to insufficient standards and requirements for geomembrane selection, installation of the geomembranes, and operating conditions. The main types of failures fall into five groups: 1) deformations such as dents and punctures, 2) breaches in geomembrane panels due to tears in transverse and longitudinal directions, 3) perforations, 4) wrinkles causing cracks, and 5) defects in the seams. In addition to the regulatory technical documents, they propose that the project must satisfy: 1) the method of delivery of geomembrane rolls and panels to the job site, 2) the method of placement of geomembrane liner systems, including bedding and protective layers (subgrade and cover layers), welding, and interlocking, 3) the methods and equipment to be used in welding geomembrane rolls in the field, 4) organization and methods to be implemented for quality control of the geomembrane, 5) organization of the construction work and methods of quality control, and 6) specific safety and environmental instructions.

Table 2-14 Statistics of the Causes of Damage at Different Locations of 300 Geomembrane-Lined Installations with Electrical Damage Detection Systems in 16 Countries. Based on the statistical analysis by Nosko and Touze-Foltz in 2000 (Mueller, 2007).

	Stone	Heavy Equipment	Welds	Cuts	Worker Directly	Total
Flat floor	2641	430	130	33	26	3260
	81.01%	13.19%	3.99%	1.01%	0.80%	77.76%
Corner, edge, etc.	234	75	69	4	14	396
	59.09%	18.94%	17.42%	1.01%	3.54%	9.44%
Under drainage pipe	50	24	45	23	24	166
	12.63%	6.06%	11.36%	5.81%	6.06%	3.96%
Pipe penetration			77	1	7	85
	0.00%	0.00%	90.59%	1.18%	8.24%	2.03%
Other	60	125	48		56	289
	20.76%	43.25%	16.61%	0.00%	19.38%	6.89%
Total	2985	654	369	61	127	4196
	71.14%	15.59%	8.79%	1.45%	3.03%	

A discussion of the failure mechanisms for geosynthetic clay liners and textured geomembranes in composite liners can be found Ghazizadeg and Bareither (2021).

Wind action can cause uncovered geomembranes to be lifted. This may create tensile forces at the anchors and/or displace the geomembrane and create wrinkles when the geomembrane falls back onto the subgrade after the wind event. Uplift of the exposed geomembranes due to wind action is discussed briefly in Section 2.7.1.

One of the failure mechanisms generally encountered in exposed geomembrane applications is the formation of "whales" or "hippos" when the uplift pressure of gases accumulated underneath the geomembrane cause it to rise locally and form a dome shape. The dome may be underwater and invisible, or if the pressure under the geomembrane is sufficiently high and the water level in the reservoir low, the top of the dome may even appear above the water surface.

Formation of a gas bubble makes the geomembrane particularly susceptible to mechanical damage and may increase leakage into the subgrade layer. It may even cause the geomembrane to burst. The short summary provided here refers to a series of three articles by Thiel (2017, 2018a, 2018b). The accumulation of gases under the geomembrane may be caused by one or more of the following:

- Air trapped under the wrinkles of a geomembrane that cannot escape into the saturated low-permeability subgrade layer
- The air in the pores of the soil rising and accumulating under the geomembrane layer due to the cyclic rise and fall of groundwater (Giroud and Goldstein, 1982)
- Decomposition of the organic material in the subgrade soils (Giroud, 1983), which may be accelerated by the higher temperatures caused by the presence of the stored water.

Koerner and Koerner (2015) list leakage from a reservoir into the subgrade material and accumulation of hydrocarbon gases from contaminated soils, which may also be triggered by the increased temperatures due to the stored water, as mechanisms that can cause the accumulation of gases under a geomembrane liner.

When a dome becomes tangent to the subgrade material at some radial distance from the top of the dome, Thiel (2017) provides an equation to estimate the shape of the dome and the pressure underneath for a given type of geomembrane and its characteristics that define stress-strain relationship. Thiel (2018a) discusses the general principles of the design of an “underdrain” layer under the geomembrane to collect and relieve gases under the geomembrane layer. Naturally, the underdrain layer must have a sufficient transmissivity for gases, and the gas to be evacuated must be estimated. Koerner and Koerner (2015) discuss the design of underdrains for geomembrane-lined surface reservoirs to prevent the formation of gas domes and to prevent uplift due to leaked liquid. Because of this dome issue, geomembrane systems placed directly on low-permeability subsoil without an underdrain system, as in illustrations a and b in Figure 2-14, are not recommended.

In some cases, a gas dome may form despite the presence of a gas evacuation system. In such cases, it may be necessary to laterally displace the gas dome towards the vent system (Wallace et al., 2006). Thiel (2018b) discusses the force required to manually move a gas bubble towards the venting pipes. The *in-situ* repair of geomembrane whales and hippos is discussed in Koerner and Koerner (2014).

2.7 Special Considerations

2.7.1 Effect of Wind on Uncovered Geomembranes

Geomembranes that are not protected by a cover layer of suitable thickness are subject to suction forces, i.e., pressure forces lower than the ambient pressure, created by the wind, that may cause uplift of the geomembrane. Such an uplift of the membrane is not desirable because it can tear the geomembrane, pull it out from its anchors, displace the position of the geomembrane by creating wrinkles, etc. Giroud et al. (1995) provide an excellent review of the wind suction that causes uplift, the stresses that uplift generates in the geomembrane layer, and the techniques that can be used to prevent uplift.

Suction forces on the geomembrane may be created by local variations of wind speed and the surface pressure it creates: When the wind blows over a reservoir, suction and compression forces are generated due to the specific geometry of the reservoir. Dedrick (1973) performed model tests in a wind tunnel using different reservoir shapes. **Error! Reference source not found.** shows the variation of pressure over an empty reservoir covered with a geomembrane in terms of wind pressure coefficient. The figure was adapted from Giroud et al. (1995), who established it based on the results of the wind tunnel experiments by Dedrick (1973). The wind pressure coefficient c_P, which is widely used in wind engineering, is defined as follows:

$$c_P = \frac{p - p_{ref}}{\Delta p_R} = \frac{\Delta p}{\Delta p_R} \tag{25}$$

where p is the pressure at a specific point in the structure, and p_{ref} is the reference pressure, which can be taken as the ambient air pressure. The reference pressure difference Δp_R in the denominator for a wind velocity of U at an altitude z is given as follows:

$$\Delta p_R = \rho_0 \frac{U^2}{2} e^{-\frac{\rho_0 g z}{p_0}} \tag{26}$$

where ρ_0==1.293 kg/m^3, the density of dry air at sea level, p_0=101,325 Pa, the air pressure at the sea level, g=9.81 m/s^2 is the gravitational acceleration, and U (m/s) the wind speed. Note that at sea level, $z = 0$, and equation (26) reduces to $\Delta p_R = \rho_0 U^2/2$. Inserting the values of ρ_0, p_0, and g in equation (26), we have the following:

$$\Delta p_R = 0.0659\, U^2\, e^{-1.2518\times10^{-4} z} \tag{27}$$

Note that in Figure 2-24, the negative wind pressure coefficients corresponding to negative c_P-values are plotted upwards. A negative wind pressure coefficient corresponds to a negative value of Δp over the geomembrane and tends to lift the geomembrane. As shown in Figure 2-24, negative wind pressure coefficients, indicating suction trying to lift the geomembrane, are observed especially on the leeward side of the geomembrane.

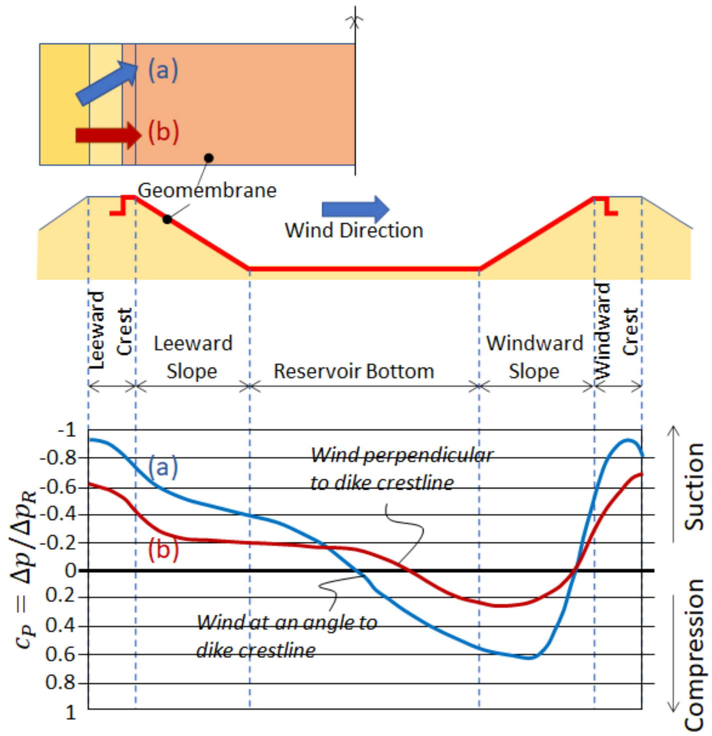

Figure 2-24 Variation of Pressure Due to Wind Blowing Over an Empty Reservoir (Giroud et al. 1995)

It can be seen that the wind blowing at an angle to the dike crestline (case b) creates a greater suction than the wind blowing perpendicular to the dike crestline (case a). The negative pressure on the leeward slope is created by the flow separation at the upwind edge or downwind edge of the crest, depending on the wind velocity and the geometry. Depending on the flow conditions and the size of the reservoir, the detached flow may reattach either on the leeward slope, or the reservoir bottom, or even on the downward slope. At the crest of the leeward dike, the wind pressure coefficient is close to $c_P = -1.00$. Over the leeward slope, the wind pressure coefficient varies in the range $0.40 < -c_P < 0.90$, but can be assumed to have a value of $c_P = -0.70$. On the reservoir bottom, from the toe of the leeward slope to the point of flow reattachment, the wind pressure coefficient can be taken as $c_P = -0.40$.

It can be assumed that a section of geomembrane with a surface area of A will be lifted if the pressure force is greater than the weight per unit area of the geomembrane denoted by μ_{GM} (kg/m^2), which is one of the properties of the geomembranes as discussed in Section 2.2.2.4. (ASTM D1910 and ASTM D1593). Using a suitable wind pressure coefficient, the pressure difference that tends to uplift the geomembrane is given by the following:

$$\Delta p = c_P \, \Delta p_R \tag{28}$$

where c_P relates the horizontal wind speed U at altitude z to the vertical pressure difference with respect to the ambient pressure Δp. Thus, the minimum weight per unit area of the geomembrane to prevent uplift is calculated from the following:

$$\mu_{GM} \geq c_P\ \Delta p_R = c_P\ 0.0659\ U^2\ e^{-1.2518\times10^{-4}z} \tag{29}$$

If the weight per unit area μ_{GM} of the geomembrane and the project altitude z are known, equation (29) can be rearranged to find the maximum wind speed up to which the geomembrane will resist uplift:

$$U < \sqrt{\frac{\mu_{GM}}{c_P\ 0.0659\ e^{-1.2518\times10^{-4}z}}} \tag{30}$$

It is also important to note that turbulence also plays a role. The expressions above were given in terms of the average wind speed at 10 m above the ground. In real-life design the engineer has to consider wind gust factor and other considerations beyond the scope of this report. More detailed information, design tables and other details can be found in Dedrick (1973) and Giroud et al. (1995). The latter provides the derivation of equations and the design procedures, including the design of the measures to be undertaken to prevent uplift. When geomembranes are subjected to uplift, greater tension forces are also generated at the anchor trenches. In the case of uncovered geomembranes, the anchor trenches must be designed with these additional tensile forces in mind.

Various measures can be used to prevent the uplift of the geomembrane due to wind (Giroud et al., 1995):

- Providing a protective cover layer, which provides additional weight on the geomembrane to prevent uplift by the wind.
- Keeping a sufficient depth of impounded water in the reservoir to protect the exposed geomembrane from the direct action of the wind.
- Placing sandbags strategically over the exposed geomembrane surface to provide additional weight to prevent uplift due to low wind speeds.*
- Building suction vents at the top of the slope to provide suction under the geomembrane, pulling it toward the subgrade and thus preventing uplift.†

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* Sandbags are also temporarily used during the construction phase to prevent uplift of the geomembrane by wind before the rolls are seamed and/or before it is covered by a protective cover layer or impounded water depth.

† Some uplift may occur before the suction pressure builds up under the geomembrane, especially if the subgrade layer has a high porosity and is holding air in the pores. There is no standard method of designing suction vents.

- Adding anchor trenches or benches on sloping side walls to provide additional weight against uplift by the wind and to reduce the length of the geomembrane along the wind direction.

- Using mechanical anchorage systems that include mechanisms to provide tension to the geomembrane to keep the lining flush against the subgrade.

Additional information on the design of anchorages to prevent uplift in geomembranes can be found in Giroud et al. 1999. This document also includes several design examples. It is important to note that the temperature plays a role in the wind uplift.

2.7.2 Geomembrane Liner Systems and Earthquakes

Most of the studies of the impacts of earthquakes on geomembrane liners of reservoirs concern solid waste landfill applications and focus on the shear stresses that develop in the geomembranes of liner systems.

Kazanjian (1999) suggested that despite limited experience at the time, liner systems of landfills can resist strong shaking motion without failure. However, he recognized that the potential amplification of the motion of the stored material of the landfill on the liner system requires further study. Fowmes et al. (2005) used the FLAC™ (FLAC, n.d.) model to study the large shear forces that may develop in liner systems of landfills due to settlement of the solids. The geomembrane was represented as a beam element with zero moment of inertia. Arab (2011) extended the FLAC model to model seismic loading and used it to study Chiquita Canyon Landfill during the 1994 Northridge earthquake in California. He developed constitutive models to simulate in-plane cyclic shear behavior of textured geomembrane and geosynthetic clay liner (GMX/GCL) interfaces and GCLs. The first one uses an empirical model, and the latter adopts a kinematic hardening, isotropic softening, multi-yield surface plasticity model.

Arab et al. (2011) also describes a methodology that allows relative displacements (slip) at the interfaces. The simulations, using a modified FLAC model, showed that if the strain concentrations predicted by Giroud (2005) are assumed, the tensile strains in the geomembrane at the crest of the side slopes exceed allowable values. Several researchers attempted to verify the model developed by Arab (2011). Thusyanthan (2007) conducted dynamic centrifuge testing to investigate the effects of simulated earthquake loading on the tension experienced by the geomembrane on a landfill slope. Their results showed that moderate earthquake loading (base acceleration between 0.1 and 0.3 g) can result in a permanent increase in geomembrane tension of 5%–25%. Gutierez (2016) used a large-scale centrifuge facility as well as field data to verify the model of Arab (2011). Wu (2017) also performed centrifuge tests to validate the model by Arab (2011) and performed parametric studies using the validated model. His results showed good agreement between centrifuge test results and simulations from the validated model.

The review of the literature showed only a limited number of studies related to the earthquake performance of geomembrane surface water reservoir liner systems. A limited number of studies found in the literature concern earthquake behavior of earthfill or rockfill dams with geomembrane liners. Erlingsson and Hauksson (2009) investigated the suitability of a geomembrane-faced rockfill dam in a highly seismic area. They used PLAXIS software

(Bentley, n.d.) with a Mohr-Coulomb soil model and applied the earthquake load at the base of the model as a time series. Uğur (2019) used the SLIDE software package (Rocscience Inc., 2002) to model Yiprak Dam in Turkey, which is a 31.5 m high rockfill dam with a storage capacity of 875,000 m^3. SLIDE is a two-dimensional vertical slope stability software package that allows dynamic loading. He concluded that the dam would perform suitably even in the case of a geomembrane failure.

2.8 References for Chapter 2

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3.0 Assessment of Polymeric Geomembrane Liners for PSH Reservoirs

This chapter provides an assessment of the application of polymeric geomembrane liners to PSH reservoirs. Topics covered include selection considerations for using geomembrane liners as the impervious water barrier, a review of current design approaches for application of geomembrane lining systems to PSH reservoirs, a discussion of current regulatory issues for application of geomembrane lining systems to PSH reservoirs in the United States, and a summary of several case histories where geomembrane lining systems have been applied to PSH reservoirs or similar applications.

3.1 Selection Considerations for Geomembranes as Upstream Water Barriers

As discussed in Section 2.0 of this report, dams and reservoirs are constructed to minimize seepage losses and provide a reliable source of water. PSH reservoirs, however, are used for energy storage and generation of electricity. A variety of methods can be used to minimize seepage losses through a dam, such as including an impermeable core within an embankment dam, providing an upstream water barrier for an embankment dam, or constructing the dam out of concrete or other low-permeability materials. This report assumes that site characteristics and engineering evaluations have resulted in the selection of an embankment dam utilizing an upstream water barrier as the most suitable seepage control method. Once an upstream water barrier has been identified by the designer as the most feasible means of providing adequate seepage control from the reservoir, a decision must be made as to what material will form the upstream water barrier. Section 2.0 of this report summarizes a variety of materials that have been identified in the literature as usable upstream water barriers for dams and reservoirs, including those for PSH applications. Of the materials identified, those most commonly used for PSH reservoirs in the past include concrete facings and dense asphalt concrete (DAC) facings (also known as hydraulic asphalt concrete or HAC facings). Therefore, when considering a geomembrane lining system as the upstream water barrier, these alternative materials should be compared to geomembrane lining systems.

There are a number of considerations to be evaluated when selecting the type of upstream water barrier to be adopted for a given PSH reservoir application:

- Material availability
- Project location and siting
- Engineering
- Constructability
- Durability
- Maintenance

- Construction costs
- Regulatory issues

Each of these considerations, except for regulatory issues, is discussed in more detail in the following sections. Regulatory issues are discussed in Section 3.3.

3.1.1 Material Availability

The selection of dam type for any PSH reservoir application is typically driven by the availability of onsite materials for use in construction. For proper construction of any dam and reservoir type, materials will need to be sourced from onsite quarry and borrow areas, from offsite sources, or a combination of the two. The following materials will typically need to be sourced for reservoir construction regardless of the upstream water barrier selected for the PSH reservoir:

- *Fill for construction of the reservoir dam embankments*. The selection of a fill dam type for a given PSH reservoir assumes that adequate volumes of fill materials are available at the project site or from nearby quarries or borrow sources, whether the dam body is composed of rockfill or earthfill.
- *Aggregates for supporting layers, transition zones, filters, and drains*. All upstream lined dams and reservoirs will require a support medium beneath the selected lining system as well as filters and drains for safely collecting and conveying seepage water through the water barrier and dam foundation. Depending on the zoning of the main dam body, one or more transition zones of processed rockfill may also be required. These materials are provided as aggregates conforming to specific material quality and gradation requirements. Materials for these aggregates may be sourced from onsite sources or from nearby quarries and borrow sources but may also need to be sourced from offsite locations if onsite sources are not adequate.

Beyond the materials listed above, the selection of a geomembrane, concrete, or dense asphalt concrete lining system can be driven by the availability of materials specific to each of these three lining systems.

3.1.1.1 Geomembrane Lining Systems

One major benefit of geomembrane lining systems is that the polymeric components of the system (geomembrane, geotextiles, geodrains, etc.) are manufactured offsite in controlled factory settings. Thus, for project sites where the acquisition of materials needed for concrete or dense asphalt concrete facings may be challenging, geomembrane lining systems can be procured and shipped to the project site.

Geosynthetics can be shipped directly to the site and stored onsite for installation or stored offsite until needed. There is also a variety of geomembrane lining types and suppliers of geomembranes and geotextiles in the industry. Since the desired material properties and behavior can be specified, and geotextiles manufactured to meet those specifications, geomembrane lining systems can be used at project locations where materials required for other types of upstream water barriers may be scarce.

3.1.1.2 Concrete and Dense Asphalt Concrete Facings

Concrete facings for reservoirs will require sourcing the following materials, which are not required for geomembrane lining systems:

- *Concrete aggregates and sand*. When onsite sources of aggregates and sand adequate for use in concrete are not available, offsite sources will need to be identified. Given the volume of concrete required for lining a PSH reservoir, offsite sources of sufficient quantity may not be available. Thus, careful consideration of concrete volumes and sources of concrete aggregates is required if concrete facings are to be used for PSH.

- *Cement and flyash.* Concrete used for concrete facings will require procurement of cement from offsite sources and flyash or other pozzolons to obtain the proper concrete mix.

- *Modifiers.* Concrete mixes may use modifiers to help obtain the proper mix design for the placement and long-term performance of the concrete facing. These modifiers are necessarily sourced from offsite suppliers.

- *Reinforcing steel.* Concrete facings will require reinforcing steel for strength and durability. These will necessarily need to be obtained from offsite sources.

- *Waterstops and accessories*. Concrete facings require water stops between each concrete panel and at the perimeters of the dam and reservoir linings, including various other accessories for completing the concrete facing. These will necessarily need to be obtained from offsite sources.

DAC facings for reservoirs will require sourcing the following materials not required for geomembrane lining systems:

- *DAC aggregates and mineral filler*. When onsite sources of aggregates and mineral filler adequate for use in DAC are not available, offsite sources will need to be identified. As with concrete facings, careful consideration of DAC volumes and sources of aggregates and mineral filler is required when DAC facings are used.

- *Bitumen.* Bitumen used for DAC facings will necessarily have to be procured from offsite sources, along with any modifiers incorporated into the mix design for performance considerations.

3.1.2 Project Location and Siting

Project location and siting considerations must be considered when determining whether a geomembrane lining system may be applicable for a given PSH project. All three upstream water barrier types considered in this chapter will require proper siting of the upper and lower reservoirs to avoid locating the reservoirs on major potential fault zones, or over significant karst areas or other adverse features. Some location and siting considerations that may drive the selection of one upstream water barrier type over another include climate, site constraints for batching and material storage, and the risk of major earthquakes that could significantly displace the selected lining system.

3.1.2.1 Geomembrane Lining Systems

An advantage to the use of geomembrane lining systems for PSH reservoir applications is that no onsite batch plant is needed for production of the lining materials. For all PSH projects, however, there will necessarily be a need for significant concrete production for various site facilities, including reservoir intake and outlet works and spillways. Along with the need for batching facilities comes the need for aggregate and cement storage, which somewhat reduces this advantage for geomembrane lining systems.

Geomembrane linings and associated geotextiles do need to be properly stored once they are delivered to the site. Therefore, a proper storage area will need to be designated for the lining materials.

Geomembrane lining systems have been used in a wide variety of climates since their initial development, and current geomembrane materials have been manufactured to minimize degradation of their behavior properties when they are exposed to the sun or temperature extremes, so extreme climate exposure no longer precludes the use of geomembrane lining systems. Selection of a particular geomembrane lining material needs special attention to temperature extremes and exposure to environments, however, as some geomembrane materials perform better than others in various climates and environments.

With proper selection and installation, geomembrane lining systems can be subject to significant deformation before tearing or rupturing. Therefore, they may be an attractive alternative when PSH reservoirs are located in highly seismic regions where the PSH reservoir may be subject to large earthquake-induced displacements.

3.1.2.2 Concrete and Dense Asphalt Concrete Facings

Batching of concrete mixes for concrete and DAC facings is typically done on site due to the need to have large volumes of concrete or DAC readily available for placement. Thus, provisions for location of one or more batch plants with adequate storage space for materials will be required as part of the site layout of the PSH facility. As described above, all PSH projects will require significant concrete production for various site facilities, including reservoir intake and outlet works and spillways. Concrete batch plants will need to be configured to allow for production of concrete for pouring the concrete facing panels and plinth foundation as well. Concrete and DAC facings may be an attractive alternative if suitable concrete aggregates can be sourced onsite or nearby.

Climate exposure for concrete facings is typically addressed during mix design so that the resulting concrete panels have the required durability, and, with proper design, concrete facing panels can accommodate a fair amount of movement due to settlement and earthquakes.

Climate exposure for DAC facings is an important consideration, as exposure to sunlight and heat extremes can affect DAC's long-term performance. Although DAC is designed to be flexible and to accommodate some differential movement of the dam and reservoir foundation, DAC may be more susceptible to damage under large displacements, and the susceptibility may increase as DAC ages and becomes more brittle.

3.1.3 Engineering

3.1.3.1 Geomembrane Lining Systems

Engineering of geomembrane lining systems for PSH reservoirs is a combination of detailed design and development of performance requirements, which geomembrane lining system manufacturers and installers will use to propose specific lining systems. Given that geomembrane lining materials are manufactured and then delivered to the site for installation, details for installation are typically provided by the manufacturers. Although some details, like seams between geomembrane panels, connections to other structures in the reservoirs, and prevention of floatation or billowing from wind, can be provided in engineering design documents prior to bidding for construction, many of these details are provided by the manufacturer only after that manufacturer has been selected. For design-bid-build type projects, selection of a lining manufacturer may be done prior to bidding for construction and installation. However, in design-build or engineer-procure-construct (EPC) projects, the project may be put out to bid without many details provided for the geomembrane lining system, and the design-build or EPC team will then work with their selected lining manufacturer to specify the appropriate materials and installation details.

Therefore, design of geomembrane lining systems requires more interaction with the lining system manufacturer(s) by the owner and engineer during design development and by the contractor during bidding and construction.

3.1.3.2 Concrete and Dense Asphalt Concrete Facings

For both concrete and DAC facings, all design details are developed by an engineering company as part of the project's design development. For concrete and DAC mixes, the design mix and performance requirements for the ultimate job mix are specified. The final mix design for the job mix will be performed by the contractor once their material sources have been identified, so as to confirm that the performance requirements have been met. Therefore, much of the detailed design can be done prior to bidding for design-bid-build projects.

3.1.4 Constructability

3.1.4.1 Geomembrane Lining Systems

Installation of geomembrane lining systems is usually done by construction machinery equipped with special attachments to handle unrolling of geomembrane panels and geotextile materials on grade. Special equipment is required for seaming adjacent panels, and construction quality control and assurance procedures require special attention to confirm that panel seams and connections to other reservoir structures, like intake/outlet works or spillways, have been properly constructed.

Using geomembrane lining materials requires a strong focus on how they are handled during installation, as they can easily be damaged during construction activities, and on quality control and testing during and after installation to confirm that the lining system has been installed properly and has not been damaged. Typically, liner system installation requires manufacturer representation on site. Some lining manufacturers may require that they perform the installation, or that manufacturer training in the quality procedures specific to that manufacturer be used.

3.1.4.2 Concrete and Dense Asphalt Concrete Facings

Concrete slabs for PSH reservoirs are cast in place on the floor or side slopes of the reservoir. This construction requires reinforcement and waterstops as well as special formwork for concrete placement on slopes to be placed by hand. Quality control for concrete placement follows typical procedures for concrete construction.

Once the concrete slabs have hardened, there is little risk of damage to the facing. Thus, protection from damage for concrete facings is typically limited to joints between panels and the supporting plinth.

DAC requires specialized pavement equipment and care during batching and placement. The impermeability offered by DAC is a result of careful mix design plus placement and compaction of the dense asphalt concrete mix at the correct placement temperature at the correct lift thickness to achieve low air void content. Deviations from the approved job mix, placement of the DAC at temperatures outside of the specified range, placement of the DAC at lift thicknesses outside of the specified range, and compaction with too high or too low a compaction effort can lead to an in-place DAC lining that has either "pre-aged" and is brittle or has a too-high air void content and therefore may not be impermeable. Because of the sensitivity of DAC to these variables, an experienced contractor and labor force is required.

Given that even small deviations from specified mix designs, temperatures, lift thicknesses, and compaction could negatively impact the performance of a DAC lining system, quality control during mixing, placing, and compacting is very important. Once the DAC is placed and has cooled to ambient temperature, there is little risk of damage to the lining system.

3.1.5 Durability

3.1.5.1 Geomembrane Lining Systems

Durability of geomembrane lining systems is primarily a function of the type of geomembrane selected and whether the geomembrane lining system is covered or exposed. Modern geomembrane materials used for lining systems for water storage reservoirs and conveyance have been formulated to withstand the effects of climate, and in particular exposure to sunlight and temperature effects. However, the water surface of reservoirs located in northern climates will ice over during winter months, causing possible damage from shearing, adherence of the ice to the lining system as the water surface fluctuates during PSH operation, and/or impacts from floating ice.

3.1.5.2 Concrete and Dense Asphalt Concrete Facings

Properly constructed concrete facings can withstand a variety of environments and last for a long period of time. The joints between concrete panels are the limiting factor to the longevity of a concrete facing system.

The durability of DAC linings is determined largely by its mix design and placement temperature. The bitumen in asphalt oxidizes when exposed to sunlight and heat, and its properties will deteriorate with time. Placement of DAC at higher than specified temperatures will “pre-age” the DAC and make it less durable. Typically, the DAC lining is protected against exposure to sun and weather by a sealing coat. The sealing coat requires maintenance, as it functions as a “sacrificial layer” in a DAC lining system. Most DAC linings will require some form of rehabilitation after about 20 years of service due to exposure to the elements.

3.1.6 Maintenance

3.1.6.1 Geomembrane Lining Systems

Geomembrane lining systems that are left exposed are more susceptible to punctures and tears, and exposed geomembrane lining systems will likely require patching and repair during their lifetimes. Adding a protective cover over the geomembrane lining system will help reduce the likelihood of punctures and tears. However, should maintenance be needed, protective covers make such maintenance more costly due to the need to carefully expose the geomembrane lining needing repair.

3.1.6.2 Concrete and Dense Asphalt Concrete Facings

Maintenance for concrete facings is usually limited to repair of the joints between concrete panels. In some cases, sealing of cracks in the concrete panels may also be required.

DAC will require resealing every 5 to 10 years to maintain the protective sealing coat, depending on the climate. Full rehabilitation of a DAC lining is typically required every 20 years.

3.1.7 Construction Costs

3.1.7.1 Geomembrane Lining Systems

Construction costs for geomembrane lining systems will vary depending on the material specified, whether the lining system is a single lining system or double lining system, and whether the lining system is exposed or covered. Each of these items contributes to the overall costs for installation of a geomembrane lining system. The geomembrane material and installation is usually the most costly element of the lining system and can be a significant cost driver for a PSH project. The cost of a double geomembrane lining system is also approximately double that of a single geomembrane lining system.

Given the potential costs associated with supply and installation of a geomembrane lining system (and in particular double geomembrane lining systems), selection of a geomembrane lining system as the lining system for a PSH project is often based on the consideration of the other factors discussed in this section of the report rather than cost alone.

3.1.7.2 Concrete and Dense Asphalt Concrete Facings

For both concrete lining systems and DAC lining systems, availability of suitable aggregate materials and sands and the need for processing adequate sand and/or gravel on site are factors that affect costs.

For concrete lining systems, the price of cement is typically the controlling material cost. The cost of labor for reinforcing steel placement, formwork, and concrete placement also drives the price of concrete lining systems.

For DAC lining systems, construction costs will vary depending on whether the lining system is a single or double lining system. Bitumen is typically the controlling material cost by unit. Since it is derived from the refining of petroleum products, bitumen's price depends on the price of petroleum, and therefore the material costs for DAC linings will also vary with the price of petroleum.

Another cost factor for construction of DAC linings in the United States is the need to import specialized equipment and labor. Construction of DAC linings differs substantially from placement of asphalt concrete for roads and highways, and there is currently limited DAC construction experience among contractors in the U.S. Therefore, the required expertise and equipment will need to be imported from elsewhere—primarily Europe, where DAC linings have been incorporated into a number of dams and reservoirs, including PSH reservoirs.

3.2 Design Approaches for Geomembrane Lining Systems

As discussed in Section 2.0 of this report, the literature review conducted for this study did not identify design guidelines specific to the design of geomembrane lining systems for PSH, and available guidelines have been developed for design and construction of other water storage facilities not necessarily subjected to the same loads and water fluctuations that PSH reservoirs are subjected to. This section provides a summary of several guidelines that are currently used for the design of geomembrane lining systems and a discussion of their application to PSH reservoirs.

3.2.1 Summary of Current Guidelines

The following are commonly used and cited guidelines for the design and construction of geomembrane lining systems for various types of impoundments (see references for this chapter):

- International Committee on Large Dams (ICOLD) *Bulletin 135, Geomembrane Sealing Systems for Dams* (ICOLD, 2010)
- American Water Works Association (AWWA) Manual M25, *Flexible-Membrane Covers and Linings for Potable-Water Reservoirs* (AWWA, 1999)
- AWWA D130-02, S*tandard for Flexible-Membrane Materials for Potable Water Applications* (AWWA, 2002)
- U.S. Department of the Interior Bureau of Reclamation *Design Standards No. 13, Embankment Dams, Chapter 20, Geomembranes* (USBR, 2018)
- Le Comité Français des Géosynthétiques (CFG) *General Recommendations for the Use of Geomembranes in Barrier Systems* (CFG, 2017)

The general content of these guidelines is summarized in the following sections.

3.2.1.1 ICOLD Bulletin 135

ICOLD *Bulletin 135* provides an overview of the application of geomembrane sealing systems to various types of dams, including embankment dams. The bulletin was prepared for the international dam design and construction community and was an update to an earlier bulletin prepared on the same topic (ICOLD, 1991). *Bulletin 135* covers the following:

- Introduction to geosynthetics (Chapter 1)
- Geomembrane materials, properties, testing, and ageing (Chapter 2)
- General loads applied to geomembrane sealing systems (Chapter 3.
- Geomembrane applications for fill dams (Chapter 4)
- Geomembrane applications for concrete and masonry dams (Chapter 5)
- Geomembrane applications for roller compacted concrete (RCC) dams (Chapter 6)
- Special applications (Chapter 7)
- Quality control for geomembrane sealing systems (Chapter 8)
- Guidance on technical contracts for geomembrane sealing systems (Chapter 9)

Of primary interest to PSH applications are Chapters 2, 3, 4, 8, and 9.

Geomembrane Materials, Properties, Testing, and Ageing

Chapter 2 of *Bulletin 135* provides a good overview of the types of polymeric and bituminous geomembrane materials available for dam and reservoir applications, the composition of the various geomembrane types available, a brief overview of how the geomembranes are supplied for use, applications of geomembranes to potable water reservoirs, seaming considerations, a comparison of the overall behavior of different geomembrane types, a summary of various performance tests performed on geomembranes, and a summary of applications of various geomembrane types in practice. The chapter also includes a discussion of the longevity of geomembranes used in dams. The summary of geomembrane types and their frequency of application to dams is useful, as it provides an idea of the precedents for use of various geomembrane materials in water storage dams and reservoirs.

General Loads Applied to Geomembrane Sealing Systems

Chapter 3 of *Bulletin 135* provides an overview of the types of loads to be considered for the application of geomembrane materials to dams. The chapter does not differentiate between types of dam applications (RCC, fill dams, etc.) and refers the reader to specific chapters that focus on each dam type. The general loads introduced and discussed include the following:

- Mechanical loads
  - Gravity (self-weight and weight of covering layers)
  - Subgrade differential settlement
  - Puncture load
  - Wind
  - Wave action
  - Ice
  - Air and/or water uplift
- Physical, chemical, and biological stress
  - Heat
  - Ultraviolet (UV) radiation
  - Water quality
  - Microorganisms
  - Vegetation
  - Animals
  - Vandalism

Geomembrane Applications for Fill Dams

Chapter 4 of *Bulletin 135* provides an overview of the application of geomembrane materials to fill dams. The chapter was written using experience and information from a database of 183 fill dams that use geomembranes as their primary water barrier. The chapter describes both upstream sealing systems and internal sealing systems for fill dams. The upstream sealing system discussion is of interest for the scope of this study.

The database used to develop Chapter 4 of *Bulletin 135* includes 22 dams designed as new dams with exposed geomembranes as their primary water barrier and 66 dams designed with covered geomembranes as their primary water barrier. Advantages and disadvantages of upstream geomembrane sealing applications are summarized, and the chapter introduces the following key principles for design of dams with upstream geomembrane facings:

- Watertightness of the geomembrane

- Watertightness and performance of connections with foundations and various reservoir structures, such as intakes
- Ability to accommodate deformation and settlement of the dam and foundation
- Anchoring systems to keep the geomembrane sealing system in place
- Provision of adequate drainage behind the geomembrane to capture or route seepage water safely away and prevent uplift of the sealing system

Chapter 4 also contains a discussion of advantages and disadvantages presented by exposed and covered geomembrane lining system applications and includes a comparison table that can be of benefit when determining whether to utilize an exposed or covered geomembrane sealing system.

A discussion of stresses on each geomembrane layer in an upstream sealing system and the stresses on the interfaces between the various layers of an upstream sealing system (e.g., the interfaces between cover and protective geotextile, geotextile and geomembrane, geomembrane and drainage layer) is also provided.

Chapter 4 of *Bulletin 135* provides a summary of the geomembrane materials commonly used for upstream geomembrane sealing systems, with PVC, LLDPE, bituminous geomembranes, and HDPE being the most common. Considerations of geomembrane material behavior in an upstream sealing system are provided for general geomembrane material selection purposes.

Quality Control for Geomembrane Sealing Systems

Chapter 8 of *Bulletin 135* provides an overview of manufacturing and construction quality control and assurance considerations for geomembrane sealing system applications. The chapter makes the important distinction between quality control and assurance performed during the manufacture of the geomembrane and geosynthetic materials and quality control and assurance performed upon receipt and installation of the materials during construction. The chapter includes a discussion of steps to be taken to prevent damage to the manufactured materials during transportation from the manufacturing facility to the construction site. Chapter 8 also includes a useful table and discussion on the types and frequency of quality control and quality assurance checks and tests to be performed, with allocation of responsibilities between the contractor and owner.

Guidance on Technical Contracts for Geomembrane Sealing Systems

Chapter 9 of *Bulletin 135* provides a summary of the considerations for preparing technical contract documents for geomembrane lining systems:

- General considerations
- Materials
- Sealing system

- Installation plan and schedule
- Quality control plan
- Acceptance of work
- Warranty
- Bill of quantities and conditions

The various subsections of the chapter provide a good discussion of the considerations listed above but do not offer specifics relative to preparing technical specifications for geomembrane lining systems. The subsection on general considerations does provide some important insights into approaches engineers and owners may take when preparing technical contract documents, such as whether to provide detailed geomembrane system designs for tender or put the detailed design responsibility on the contractor, selection of geomembrane materials during design development and the impact on the dam design, and additional discussion of the advantages and disadvantages of covered and exposed geomembrane lining systems. The chapter suggests that exposed systems may be a better option, given the challenges and additional costs that come with covered systems.

3.2.1.2 AWWA Manual M25

AWWA *Manual M25* is a technical guide for the preparation of geomembrane sealing system designs for potable water applications and serves as a companion reference to AWWA's *D130-02, Standard for Flexible-Membrane Materials for Potable Water Applications* (AWWA, 2002; see next section). *Manual M25* also includes provisions for floating cover applications for water storage reservoirs. (However, coverage of geomembranes for use of floating covers is outside the scope of this report.) *Manual M25* covers the following:

- An introduction to geomembranes for reservoirs and floating covers (Chapter 1)
- Design and installation of flexible-membrane floating covers (Chapter 2)
- Operation and maintenance guidelines for floating covers (Chapter 3)
- Design and installation of flexible-membrane linings (Chapter 4)
- Operation and maintenance guidelines for linings (Chapter 5)

Of primary interest to PSH applications are Chapters 4 and 5.

Design and Installation of Flexible-Membrane Linings

Chapter 4 of *Manual M25* provides information on the design and construction of geomembrane lining systems for reservoirs. The chapter briefly introduces the use of geomembrane linings for water storage reservoirs and discusses the following general areas:

- Economics (costs for geomembrane lining systems). *Manual M25* suggests reviewing costs for geomembrane lining systems in terms of overall life-cycle costs, given the potential for more frequent maintenance of the lining system.

- Climatic conditions. The manual's focus is on ice loading and the potential damage ice loads may cause to exposed geomembrane linings systems. *Manual M25* suggests a "chafing strip" in the zone of water level fluctuations, where the lining system may be exposed to ice loads, to minimize such damage.

- Life expectancy. Past performance of similar lining systems and consideration of manufacturers' warranties are discussed.

- Design qualifications for engineers designing geomembrane lining systems.

- Degree of reliability desired.

- Maintenance methods and equipment. Consider how the owner/operator will maintain the lining system once it is installed.

- Regulatory issues. Will a proposed geomembrane lining system be accepted by the regulator?

- Foundations.

- Lining support and drainage blanket.

- Underdrains.

- Venting.

- Penetrations.

- Piping.

- Valves.

- Reservoir walls.

Other design and construction considerations in the chapter include the following:

- Geomembrane seams, including types of seams, seaming procedures, adhesives, and testing.

- Construction sequencing, equipment, floor traffic, and leakage.

- Quality control.

- Warranty.

- Lining installation and testing, with guidance provided on reservoir filling and leakage testing.

Operation and Maintenance Guidelines for Linings

Chapter 5 of *Manual M25* provides guidelines for maintenance of exposed geomembrane lining systems, including cleaning and provisions for emptying and filling the reservoir for maintenance and inspection.

3.2.1.3 AWWA D130-02, Standard for Flexible-Membrane Materials for Potable Water Applications

AWWA *Standard D130-02* provides AWWA's minimum requirements for procurement, installation, and quality testing of geomembrane materials. Minimum requirements included in *Standard D130-02* pertinent to PSH reservoirs include the following:

- General requirements for materials and material composition for use in water storage reservoirs.

- Minimum thickness requirements for geomembrane materials.

- General requirements for fabrication, including factory seaming quality and testing.

- General requirements for field installation, including seaming and quality testing. A brief discussion on determining permissible leakage rates is included, but no specific recommendations are provided.

Section 5 of *Standard D130-02* provides general guidance on quality assurance, including development of a quality assurance program. The section includes tables delineating the geomembrane tests, test frequency, and testing responsibilities for manufacturers, fabricators, and installers. Section 6 of *Standard D130-02* provides minimum requirements for delivery of geomembrane materials to the project site.

AWWA *Standard D130-02* and AWWA *Manual M25* should be considered companion documents when used to develop geomembrane lining system designs. AWWA *Standard D130-02* provides a discussion of the various considerations for design and construction of geomembrane lining systems, and AWWA *Manual M25* provides minimum requirements to guide the design and construction of geomembrane lining systems.

3.2.1.4 USBR Design Standards No. 13, Embankment Dams, Chapter 20, Geomembranes

USBR *Design Standards No. 13, Embankment Dams, Chapter 20, Geomembranes* provides significant information and guidance on the use of geomembranes in embankment dams. The design standard is provided in two volumes covering the following:

- Introduction to the use of geomembranes in dams (Section 20.1).

- Description of geomembranes, including types, composition, reinforcement of geomembranes, production, and a summary of the physical characteristics of geomembranes (Section 20.2)
- Application and design of geomembranes for use in embankment dams. This section covers a variety of uses for geomembranes, including the use of geomembranes as embankment dam facings and for partial or total lining of reservoirs (Section 20.3).
- Design considerations including:
    - Laboratory testing
    - Interface strength
    - Slope geometry
    - Seam design
    - Anchorage trenches and connections
    - Leakage
    - Uplift
    - Settlement
    - Exposed versus covered geomembranes
    - Protective cover design
- Construction monitoring, including manufacturing and fabrication, transportation, onsite storage, handling, placement, seaming, placement of cover materials, applications for repair of rockfill dams, construction quality assurance, and monitoring and corrective measures (Section 20.5).

Chapter 20 is a comprehensive document and provides the most detail for design and construction of geomembrane lining systems. Engineers designing geomembrane lining systems for dams and reservoirs, including PSH reservoirs, should be familiar with its contents. Some highlights are provided below.

Introduction to Geomembranes

Section 20.2 provides an introduction to geomembrane materials and covers the following topics:

- Composition of geomembrane materials
- Reinforcement of geomembranes
- Production of geomembranes

- Physical characteristics of geomembranes

The section also provides comparisons of various geomembrane material types.

Applications and Design

Section 20.3 provides a comprehensive guide to the application of geomembranes in embankment dams in general along with detailed design guidance. Of interest for this report is the coverage of applications of geomembranes as upstream facings for embankment dams.

Section 20.4 introduces the design elements common to various geomembrane applications in embankment dams. Of note in Section 20.4.10 is the discussion of protective covers for geomembrane lining systems. According to Section 20.4.10, the design of protective covers should include two aspects: the resistance of the protective cover to wave action, and the stability of the lining system (i.e., protective cover, geomembrane liner, and associated drainage layers) against the effect of gravity forces, seismic actions, and pore water pressures. Chapter 20 covers the second of these two aspects and refers to Chapter 7 of *Design Standards No. 13 – Embankment Dams* (USBR, 2014) for resistance of protective covers to wave action.

Section 20.4.6 covers the following topics for leakage design for geomembrane lining systems:

- Leakage evaluation
- Leakage collection and detection

Section 20.4.6.1 presents methods for evaluating rates of leakage of water through geomembranes. Evaluation guidance is provided for the following:

- Permeation through geomembranes
- Leakage due to pinholes in geomembranes
- Size and frequency of holes in geomembranes
- Leakage through holes in geomembranes overlain and underlain by highly permeable materials
- Leakage through holes in geomembranes underlain by low-permeability soil.
- Leakage through holes in geomembranes in contact with medium-permeability drainage materials

Of note are the details and guidance for determining leakage through holes in geomembranes. Per Section 20.4.4.3.4., guidance cited from the U.S. Environmental Protection Agency (EPA) for waste containment indicates that under strict construction supervision, the number of geomembrane defects is on the order of one per acre, and the size of defects is on the order of 0.016 square inches or less. USBR (1992) cites EPA (1987), which recommends using two hole sizes for design: 0.16 square inches for worst case conditions and 0.016 square inches for average case conditions. Quality control and quality assurance during manufacturing and installation of geomembrane lining systems should require more stringent controls on leakage from dams and reservoirs. For PSH reservoirs specifically, more stringent requirements may be required to minimize leakage through the lining system, or provisions for collection and return of leakage water through defects in the lining system should be considered when make-up water may not be readily available.

Section 20.4.6.3 covers the topic of leakage collection and drainage. The section covers selection of drainage layer materials, evaluation of the flow capacity of the drainage layer, and flow capacity of geonet drainage layers used in lieu of other drainage materials.

Section 20.4.7 covers considerations for wind and uplift caused by wind, including the following factors:

- Suction caused by wind
- Thickness of cover to prevent wind uplift
- Geomembrane tension caused by wind
- Geomembrane anchorage against wind

Section 20.4.8 covers the effects of differential settlements, connections of geomembrane lining systems to concrete structures, the resistance of geomembrane lining systems to lack of underlying support, and resistance of geomembranes when supported by an underlying geotextile reinforcement layer to bridge foundation defects.

Section 20.4.10 covers topics for the design of protective covers, including tension in geosynthetics on slopes, soil cover stability, and concrete cover design, and provides typical slopes used for protective covers. The discussion in this section of design of protective covers to withstand rapid drawdown of the reservoir is of particular interest to PSH applications.

Section 20.4.5 covers the design topic of geomembrane anchorage and connections, including anchor trench design and connections to rigid structures.

Although specific equations and figures are provided for design guidance in Section 20.4, the section makes numerous references to other publications on the topics covered.

Construction and Monitoring

Section 20.5 covers construction and monitoring and provides construction considerations for geomembrane panels, seaming, placement of cover materials, repair of dams, and construction quality assurance.

Sections 20.5.1 and 20.5.8 cover seaming of geomembrane panels including the types of seams, seaming methods, and seam layout.

Section 20.5.2 covers construction quality assurance (CQA) and the following CQA elements:

- Qualifications of the manufacturer, fabricator, installer, and other parties involved with the manufacture, fabrication, transportation, and installation of geomembrane lining systems

- Seam quality, testing, patching and repairs

- Visual observations

- Documentation

Section 20.5.3 through 20.5.7 focus on the manufacturing, fabrication, and construction of the geomembrane panels themselves:

- Manufacturing and fabrication, including packaging and delivery to the project site

- Onsite storage

- Handling

- Placement, including site preparation, installation planning, placement, and the impacts of placement temperature and weather (wind and rain)

Sections 20.5.9 and 20.5.10 cover field testing, patching, and repair of placed geomembranes.

Section 20.5.11 covers corrective measures in the event that flaws in the installed geomembrane are identified.

Section 20.5.12 covers final acceptance of installed geomembrane linings.

Section 20.5.13 covers protective cover materials placed over the installation geomembrane lining, which may be concrete, granular materials, or other geosynthetics.

3.2.1.5 CFG's General Recommendations for the Use of Geomembranes in Barrier Systems

CFG's *General Recommendations for the Use of Geomembranes in Barrier Systems* was prepared by the French Committee on Geosynthetics as a guide for the manufacture, design, and construction of geomembranes as barriers for different applications, including dams and reservoirs. The document provides an overview of the following topics:

- Introduction to geomembrane lining systems (Part 2)

- Design considerations (Part 3)

- Construction considerations (Part 4)

- Control of construction (Part 5)

- Construction quality assurance (Part 6)

- Guaranties, insurance, and disputes (Part 7)

Part 2 of *General Recommendations* covers the various elements of a geomembrane lining system, including support for the geomembrane lining, drainage of water and gas from beneath the geomembrane lining, geomembrane lining materials and their general characteristics, and protective covers. The coverage is general in nature but provides another point of reference for the design of geomembrane lining systems.

Part 3 covers design of geomembrane lining systems including the following considerations:

- Hazard classification (consequence classes). Note that the definitions used in *General Recommendations* differ from those used in the United States.

- Site characteristics including location, surroundings, exposure and climatic conditions, hydrogeological and geological conditions, topography, accessibility, and environment.

- Function of the reservoir, such as type of storage, confinement, and filtering.

- Geometry of the reservoir, including volume, depth, and dam cross section.

- Nature of products stored (water) and its composition, temperature, and duration of exposure.

- Conditions of installation, including site geometry, accessibility, delivery timeline, climatic conditions, and security.

- Conditions of use and maintenance, such as variability of storage levels, surroundings, cleaning and dredging requirements, hazards, possible developments (geometry and use), and inspection.

Given that *General Recommendations* is primarily for use in France, there are references to French and European standards and requirements that may not be pertinent to the design of geomembrane lining systems in the United States.

Part 4 covers construction, including compaction of supporting materials, vegetation removal, treatment of the slope crest and toe, foundation preparation, drainage of water and gas, transportation, storage, and placement of geomembrane panels, seaming and welding, anchoring, connections to concrete structures, and installation of the protective cover layer. Of note in Part 4 is the provision of specific guidelines for welding of geomembrane panels by geomembrane material type, including bituminous geomembranes, polyvinylchloride (PVC), high-density polyethylene (HDPE), F-PP (polypropylene), and ethylene propylene diene monomer (EPDM).

Parts 5 and 6 cover quality control and assurance topics for geomembrane lining system installation, including quality control for the materials at the job site, performance tests, and installation. Part 5 includes a summary of non-destructive and destructive tests for geomembrane installation quality control. Part 6 provides guidelines for the development of a quality assurance plan.

Part 7 covers topics related to geomembrane lining system manufacturing and installation guarantees, insurance, disputes, and the roles of the various parties in such matters (owner, contractor, manufacturer, engineer, or third party construction quality assurance provider).

3.2.2 General Design Considerations

ICOLD *Bulletin 135*, AWWA *Standard D130-02* and *Manual M25*, USBR's *Design Standards No. 13, Embankment Dams, Chapter 20, Geomembranes*, and CFG's *General Recommendations* all provide design guidance pertinent to the design of geomembrane lining systems for dams and reservoirs. Although none of the above guidelines address design considerations specific to PSH reservoirs, the topics covered by these guidelines can be used to assist in developing geomembrane lining system designs for PSH reservoirs with the following additional considerations:

- *Hazard classification of the PSH reservoir.* CFG's *General Recommendations* briefly mentions hazard classification as a design consideration for geomembrane lining systems, but, as noted previously, it focuses on hazard classifications used in France. For PSH reservoirs in the United States, hazard classification of dams and impoundments will be defined by the requirements of the Federal Energy Regulatory Commission (FERC). FERC's *Engineering Guidelines for the Evaluation of Hydropower Projects* (FERC, 2022) provides the requirements for such classifications. Designation of a dam as low, significant, or high hazard will control how critical the design of the geomembrane lining system is relative to dam safety and prevention of dam failure. This may ultimately control the determination of lining type, whether a single or double lining may be required, and whether a protective cover system may be needed. As PSH developments necessarily will require review by FERC as the main regulatory agency for hydropower development in the United States, FERC's *Engineering Guidelines* should be utilized along with guidelines for geomembrane lining system design and construction. Section 3.3 of this document provides additional discussion of regulatory considerations when designing geomembrane lining systems for PSH.

- *Allowable leakage rate.* USBR (2018) discusses the topic of allowable leakage rates relative to selection and design of a geomembrane lining system. Allowable leakage rates through the lining system of a PSH reservoir will be a function of the availability of make-up water and may control whether a single or double geomembrane lining system is required. In closed-loop PSH reservoirs, make-up water may come at a premium cost to the owner, and either the allowable leakage rate through the geomembrane lining system will need to be minimized or a mechanism designed to collect water that seeps through the lining system and redirect it back into the reservoir through a pump-back system. These situations may require the use of a double lining system, so that leakage through the primary geomembrane liner can be collected in a drainage layer sandwiched between the primary and secondary lining system and routed towards a pump-back system back into the reservoir. A leak detection system may also be necessary in order to detect defects in the geomembrane lining system so that repairs can be implemented.

- *Use of protective covers.* The decision to utilize an exposed geomembrane lining system or a covered system is a function of a number of considerations. ICOLD *Bulletin 135* Table 25 provides a good discussion of the advantages and disadvantages of covered versus exposed geomembrane lining systems and summarizes the following factors to consider:
  - Risk of damage to the geomembrane lining system during construction
  - Risk of damage to the geomembrane lining system by floating debris or ice
  - Risk of damage to the cover material
  - Risk of vandalism damaging the geomembrane lining system
  - Ability to visually inspect the geomembrane lining
  - Cost of inspection of geomembrane lining system
  - Ability and cost to repair the geomembrane lining system while dry or wet
  - Durability of the geomembrane lining system
  - Cost and schedule impacts

No specific guidance has been provided in the documents reviewed for this study on whether a geomembrane lining system should be covered or exposed. Therefore, the owner and engineer should carefully consider the above factors for this design consideration.

Each of the design guidelines provides a summary of the various loading conditions to consider for design of a geomembrane lining system. The lists of loading conditions taken together is comprehensive and covers most of the loads a geomembrane lining system will be exposed to in a PSH application. However, some additional attention should be paid to the following:

- *The rapid and regular water level rises and falls in a PSH reservoir.* Stability and uplift loads from rapid drawdown of the reservoir need to be considered as a normal loading condition, which differs from non-PSH reservoirs where rapid drawdown conditions are considered unusual or extreme.

- *Water velocities near intake/outlet structures in a PSH reservoir*. These may be higher than those typically encountered in non-PSH reservoirs. Water flow is also in both directions depending on whether the reservoir is being filled or emptied during the generation/pumping cycle. Forces on the geomembrane lining system and at the connections made between the geomembrane lining system and structures like the intakes/outlets will require special consideration.

- *Water quality and temperature considerations*. For PSH applications, some additional consideration of water quality and water temperature is required, particularly for closed-loop systems where new water is not regularly added to the reservoirs. Over time, suspended solids, microorganisms, and other contaminants may accumulate in the reservoirs. Geomembrane selection should consider this potential. In addition, over time the water temperature in closed loop PSH reservoirs may increase over what might normally be experienced in non-closed loop reservoirs. The behavior of the geomembrane lining system selected in the temperature range it may be exposed to throughout the lifetime of the PSH facility should be explored.

- *Dam safety and impacts on dam performance in the event of a rupture in the geomembrane lining system.* Although not specific to PSH reservoirs, an evaluation of potential failure modes and mitigations for such failure modes is an important part of the design of a geomembrane lining system. Design of the dam and reservoir should take into consideration a failure mode in which the geomembrane lining system ruptures and becomes ineffective. In such an event, water in the reservoir would be lost through at least the primary geomembrane lining system. A potential rupture should be reflected in drainage provisions beneath the geomembrane liner as well as the composition of the dam fill and foundation of the reservoir floor. A phreatic surface and through-seepage may develop through the dam and foundation, and dam stability and the potential for internal erosion of fill or foundation materials needs to be evaluated. Thus, use of a geomembrane lining system may impact the overall zoning of a dam and the foundation treatment required for the dam and reservoir floor.

3.2.3 Application of Current Approaches to PSH Reservoirs

Based on the summary of the design guidelines provided in Section 3.2.1 of this report and general design considerations discussed in Section 3.2.2, the current approaches for design and construction of geomembrane lining systems for PSH reservoirs are practical but have some shortcomings. Current design guidelines do not address PSH applications, and the design considerations listed in Section 3.2.2 require additional thought by owners and engineers when applying current design guidelines to the design of geomembrane lining systems:

- The most comprehensive coverage for design of geomembrane lining systems for dams is USBR's *Design Standards No. 13, Embankment Dams, Chapter 20, Geomembranes*, and the topics covered are pertinent to PSH reservoirs. However, since it does not address PSH issues directly, the additional considerations in Section 3.2.2 should be accounted for.

- AWWA's *Standard D130-02* and *Manual M25* provide an overall summary of minimum requirements and considerations for design and construction of geomembrane lining systems. However, as discussed in Section 3.2.2, requirements for PSH reservoirs may be more stringent than those for the applications considered in the AWWA guidelines. The AWWA guidelines also have not been updated since 2002 and may suffer from the same shortcomings in this area as USBR (1992). Therefore, the AWWA guidelines provide a useful starting point for the development of designs and construction documents for geomembrane lining systems for PSH reservoirs, but the minimum requirements provided should be reviewed and adjusted to fit the performance requirements for PSH reservoirs.

- ICOLD *Bulletin 135* also provides a good overall summary of design and construction considerations for geomembrane systems applied to dams. It is a useful companion reference to USBR (1992) although it lacks the specific guidance that USBR (1992) provides.

- CFG's *General Recommendations* is more current than the other guidelines reviewed for this report, but again only provides general discussion on considerations for design and construction of geomembranes. It also references French and European standards for geomembrane testing and quality control, and those standards could contradict those adopted and used in the United States. Therefore, CFG's *General Recommendations* provides useful information as a complement to the other guidelines, but likely is best left as a secondary source of information on the design and construction of geomembrane lining systems for PSH in the United States.

In summary, no one reference provides updated and comprehensive design guidance for the design of geomembrane lining systems for PSH applications. Owners and engineers will need to draw from multiple sources when developing such designs and carefully consider the additional design considerations for PSH reservoir geomembrane lining systems. Given the current state of the geomembrane industry, we recommend the following:

- PSH facility owners and engineers should become familiar with the current guidelines summarized in Section 3.2.2, along with other guidelines they may identify in the development of a PSH project. Decisions concerning whether a single or double geomembrane may be required and whether the geomembrane lining system should be covered are a determined by a combination of factors that require forethought by owner and engineer together.

- Selection of a geomembrane material can be informed by the guidelines in Section 3.2.2; however, the geomembrane industry has evolved since the issuance of these guidelines and will likely continue to evolve in the future. Therefore, it is recommended that a short list of applicable lining types be identified and that then the engineer and owner reach out to specific geomembrane suppliers to work with to develop the geomembrane lining system design. Geomembrane suppliers often have the engineering resources and expertise to assist with the design of their lining systems and often incorporate proprietary design details and elements of the system. In design-bid-build projects, it may be best to select the geomembrane supplier and prepare the geomembrane lining system design to the desired level of development with them before contracting for the procurement and installation of the geomembrane lining system.

- Per ICOLD (2010), the owner and engineer will need to consider the level of detail they wish to include in the contract documents for the geomembrane lining system. Supplying more detail will require selection of the geomembrane lining material and likely the geomembrane supplier early on, as indicated above. This provides the owner and engineer with more control over the details of the system to be installed but places the responsibility for preparation of a complete and detailed design on the owner and engineer. Such an approach may be taken in design-bid-build contracts. The owner and engineer may decide to issue contract documents with less design detail and place the detailed design responsibility on the contractor. In this situation it will be important to clearly delineate the performance criteria for the geomembrane lining system.

3.3 Regulatory Considerations

The primary regulator for PSH facilities in the United States is the FERC. FERC's regulatory mission includes the engineering review of hydropower projects, including PSH reservoirs. Depending on the location and owner/operator of a given PSH facility, other agencies that may be involved in the review and approval of engineering designs for PSH include the U.S. Army Corps of Engineers (USACE), USBR, and state dam safety agencies. This section summarizes the current state of regulatory requirements as they apply to the use of geomembrane lining systems at PSH facilities.

3.3.1 Application of FERC Regulations

FERC's *Engineering Guidelines for the Evaluation of Hydropower Projects* (FERC, 2022) is the primary document that outlines dam safety and performance criteria for dams and other appurtenant works for FERC-licensed hydropower projects. FERC's general regulatory requirement for the design of any new project is provided in Chapter 4, Section 4-1.3.2 Review of New or Proposed Dams:

For proposed dams, an analysis of the stability and adequacy is required unless specifically exempted by the Commission. The methods and procedures used in the evaluation of any embankment should be consistent with the latest, accepted state-of-the art methods and criteria, and with guidance contained in this chapter of the guidelines. For proposed or new dams, the licensee will be required to submit a design report in accordance with the Commission's Regulations. This report will be thoroughly examined to determine if all appropriate design criteria have been met (FERC, 2022).

Additional details on requirements for the evaluation and design of dams, intakes, penstocks, and other features pertinent to PSH facilities can be found in FERC's *Engineering Guidelines*. However, the guidelines currently do not address any specifics related to the use of geomembrane lining systems for the design of dams or PSH reservoirs.

FERC also provides technical guidance on the design and operation of PSH systems in its *Pumped Storage Hydro-electric Project Technical Guidance* document (FERC, 2007). The document includes general guidance on technical considerations for PSH reservoirs, but there are no specifics provided pertaining to the use of geomembrane lining systems.

On the other hand, the guidelines summarized in Section 3.2 of this report specifically address the design and construction of geomembrane lining systems, including a description of geomembranes, properties and testing of geomembranes, applications and design, and construction and monitoring. The guidelines presented in FERC's *Engineering Guidelines* should be supplemented with those in Section 3.2 of this report and of other regulatory agencies as described below.

3.3.1.1 FERC Design Review Process

FERC's design review process is common to all projects under FERC regulation, and early engagement for new construction is strongly recommended. Specific requirements for design development are often found in the various license articles included in a FERC license for the development of a PSH facility, and they may include design review by an independent Board of Consultants and submittal of design documents for FERC review at least 60 days before the start of construction. However, at a minimum, new projects will require a potential failure modes analysis (PFMA) per FERC (2022). The PFMA and associated workshops will need to be performed prior to final review and approval by FERC and will require early interaction with FERC.

3.3.1.2 Risk Informed Approach to Design

FERC's *Engineering Guidelines* contains requirements for risk assessments in addition to PFMAs for the evaluation of existing hydropower projects, as part of FERC's risk-informed decision making (RIDM) initiative. For new hydropower developments, including PSH, following FERC's RIDM processes should be anticipated and considered. Details on FERC's RIDM processes are provided in FERC (2016). FERC's *Engineering Guidelines* provides specifics on applying RIDM in the context of FERC's dam safety program. Among the potential failure modes most relevant to PSH reservoir liners and geomembranes is the potential for leakage through the liner, leading to internal erosion or instability of the embankment and potential piping failure.

3.3.2 Application of Other Guidelines

As indicated previously, depending on the location of a given PSH facility, other agencies that may be involved in the review and approval of engineering designs for PSH include the USACE and USBR along with state dam safety agencies. Engineering guidelines and requirements for projects requiring review and approval by the USACE are contained in a number of publications available from the USACE's website including Engineering Regulations (ERs), Engineering Circulars (ECs), and Engineering Manuals (EMs). For PSH projects, the USACE will apply the guidance provided in these various documents to their evaluation of the adequacy of the reservoir design, including a proposed geomembrane lining system. Currently, USACE publications do not contain guidance on the application and design of geomembrane lining systems for reservoirs or PSH reservoirs.

For PSH developments that will require review by USBR, the evaluation will reference USBR (1992).

Although FERC's federal jurisdiction over PSH facilities often supersedes state or local jurisdiction, some state dam safety offices may be part of the design review of a PSH project. In such cases it will be important to review any state dam safety guidelines pertinent to PSH reservoirs and the use of geomembrane lining systems for dams and reservoirs.

3.4 Case Histories

This section presents several case histories identified in the literature for geomembrane linings as applied to PSH reservoirs or to reservoirs that are subject to loadings and water level fluctuations similar to those of PSH reservoirs. The case histories selected are limited to those where geomembrane lining systems were utilized as part of the design of the original project. Applications of geomembrane lining systems for repair of or rehabilitation of other types of lining systems, like concrete or DAC, are not included. The case histories identified include the following:

- Mt. Elbert Forebay (USA)
- Okinawa Yanbaru (Japan)
- Afourer (Morocco)
- Calheta/Pico da Urze (Portugal)
- Mount Gilboa (Israel)
- Kokhav Harden (Israel)
- Panama Canal third set of locks water saving basins (Panama)
- Abdelmoumen (Morocco)

All of the above projects, with the exception of the Panama Canal third set of locks water saving basins, are PSH facilities. The Panama Canal third set of locks water saving basins were included as they represent a recent application of an exposed double geomembrane lining system with loadings and water level fluctuations similar to those of PSH reservoirs.

3.4.1 Mt. Elbert Forebay (USA)

The Mt. Elbert pumped-storage project is a 200 MW pumped storage project completed by the USBR in 1981. The power generated at Mt. Elbert derives from water originally pumped from Twin Lakes, which acts as the Mt. Elbert afterbay, and also from supplemental water delivered from Turquoise Lake to the forebay.

The original forebay reservoir was built between 1975 and 1977 and was formed by constructing a small dike in the open southwest corner and a 27 m high zoned earth embankment across the open north side of a topographic depression. A ridge, composed of glacial deposits overlying weakly indurated formation materials, forms the south side of the reservoir and separates it from the lower Twins Lakes Reservoir.

Initial construction of the forebay reservoir included a 1.5 m thick compacted earth lining under the entire reservoir. However, when water was introduced to the forebay to a depth of 7.5 m between November 1977 and March 1978, it was determined through monitoring wells that the earth lining failed to provide adequate seepage control. The seepage had the potential to reactivate an ancient landslide adjacent to the project, endangering the power plant below.

A flexible membrane lining over the entire bottom and side slopes was constructed in 1980 as a remedial measure. The top 0.6 m of the existing earth lining was removed, and approximately 290 acres of 45-mil reinforced chlorinated polyethylene (CPER) geomembrane was installed. The geomembrane was then covered with 18 inches of earth cover.

After 22 years of service, the geomembrane was noted to be performing well, with no visual signs of deterioration. Observation wells showed that the groundwater levels were stable, and that the geomembrane was essentially watertight. Some properties, such as membrane tensile strength, were noted to have declined 25% to 50% by 2000, with most of the decrease noted in the first few years of service.

Although the Mt. Elbert Forebay was not originally constructed with a geomembrane lining system, the project represents an early use of geomembrane linings in the United States and the earliest application of a geomembrane lining system at a PSH facility. The case history is notable for the long-term performance of the geomembrane lining system.

More details on the Mt. Elbert Forebay and lining installed in the forebay can be found in Morrison et al. (1991), USBR (2002), and USBR (1981)

3.4.2 Okinawa Yanbaru (Japan)

The Okinawa Yanbaru seawater pumped storage power station was located in Okinawa, Japan, and had a maximum output of 30 MW. The construction of the project was completed in 1999 and was the first PSH facility to use seawater and one of the first PSH facilities to utilize an exposed geomembrane lining system. Conceived and constructed as a demonstration project, the project was decommissioned in 2016 after 15 years of operation.

The project used the Pacific Ocean as the lower reservoir. The upper reservoir was excavated into a bluff approximately 150 meters above the sea level. The upper reservoir had a maximum depth of 25 m, with an octagonal shape and an effective storage capacity of 564,000 m^3.

The upper reservoir was lined with a 2 mm thick (EPDM) rubber sheet. Below the lining, a geotextile cushioning layer (nonwoven spun bonded polyester fabric) was installed to prevent damage to the rubber lining. The underlying drainage layer consisted of gravel with a maximum particle size of 20 mm. Seawater sensors and pressure gauges were installed beneath the lining system in pipes connected to the drainage layer. The sensors would emit an alarm to indicate seawater leakage. A seepage collection and pump-back system was also constructed.

More information on the Okinawa Yanbaru PSH project can be found in Hiratsuka, et al. (1993) and JCOLD (2001).

3.4.3 Afourer (Morocco)

The 484 MW Afourer PSH station in Morocco was completed in 2004 and includes two upper reservoirs and two hydrostations containing reversible pump-turbine units. The combined reservoirs have a useable capacity of 1,260,000 m^3 each, and were constructed on permeable foundation materials in karst conditions. In order to provide seepage control for the reservoirs, a 1.5 mm-thick PVC geomembrane lining sandwiched between two non-woven geotextiles was used. The composite lining was supported by a granular drainage layer and protected by a 0.3 m thick granular covering layer on the reservoir floors and a 0.2 m thick granular covering layer on the reservoir slopes. The reservoir slopes were relatively flat (5H:1V to 3H:1V) to allow for vehicle access into the reservoirs and to allow for laying the granular protective layer on the geocomposite lining system.

More information on the Afourer PSH project can be found in Fayoux and Dewalque (2006).

3.4.4 Calheta/Pico da Urze (Portugal)

The 30 MW Calheta PSH facility is located on the island of Madeira. A new upper reservoir was constructed and completed in 2020, formed by Pico da Urze dam and reservoir with a total storage volume of 1,000,000 m^3. Pico da Urze dam is a 31 m high rockfill embankment with 1.4H:1V slopes. The depth of the reservoir is approximately 25 m. The foundations of the reservoir and dam are permeable, and a geomembrane lining system was chosen as the upstream water barrier. The tender design for the project included an exposed 2.5 mm thick HDPE geomembrane installed over a thick anti-puncture geotextile.

The final design was changed to a single geocomposite Carpi SIBELON® CNT 3950 geomembrane lining, consisting of a 2.5 mm thick PVC geomembrane heat-bonded during fabrication to a polypropylene geotextile. The primary anchorage system consists of longitudinal trenches filled with a concrete ballast and placed along the crest of the dam and reservoir crest of the reservoir, the toe of the slope and along traffic paths. The anchorage system along the slopes of the reservoir consists of vertical concrete trenches at 10 m spacing where the geocomposite is anchored with a stainless-steel batten strip. The dam slopes were treated with shotcrete, and the geocomposite is anchored to the shotcrete with continuous stainless-steel batten strips at 8 m spacing. At various locations the geocomposite is ballasted by reinforced concrete slabs. A protective geotextile was placed on the geocomposite under the slabs to prevent damage.

The reservoir was impounded in January 2020 and is currently performing according to expectations. Additional details can be found in Carpi (n.d.) and Vaschetti et al. (2021).

3.4.5 Mount Gilboa (Israel)

The 300 MW Mount Gilboa PSH project in Israel generates 3,000 MWh of electricity annually and was completed in 2020. The project is composed of two geomembrane-lined reservoirs, each with a capacity of 2,500,000 m^3, connected by a 500 m deep shaft and a 6 km conveyance tunnel having a diameter of 4.5 m. The project consists of a 50 m tall underground powerhouse containing two 150 MW turbines.

The geomembrane liners are connected to the cast-in-place reinforced concrete intake structures and the mechanically stabilized earth retaining walls in the upper reservoir adjacent to the intake. The primary geomembrane lining system consists of an exposed 1.5 mm thick HDPE geomembrane placed atop a protective geotextile and supporting granular drainage layer.

More information on the Mount Gilboa PSH project can be found in Ingram (2020) and Carpi (n.d.).

3.4.6 Kokhav Hayarden (Israel)

Kokhav Hayarden, owned by Star Pumped Storage Ltd., is under construction in the northern region of Israel and will be the largest pumped storage facility in Israel, with a capacity of 344 MW and comprising an upper and lower reservoir with a total capacity of approximately 3,000,000 m^3 each. Both reservoirs will be waterproofed with a PVC geomembrane lining system manufactured by Carpi. The upper reservoir will consist of a 25 m high and 1.8 km long earthfill dam, and the lower reservoir will consist of a rockfill concrete dam running 1.7 km long and 18 m high. The slope of the upper reservoir is 3.5H:1V. The slope of the lower reservoir is 3H:1V. The total surface to be lined is approximately 432,000 m^2.

The design consists of an exposed 2.0 mm PVC geomembrane bonded to a 500 g/m^2 nonwoven geotextile for the reservoir side slopes and a 1.5 mm PVC geomembrane bonded to a 500 g/m^2 nonwoven geotextile for the reservoir floor. A geodrain beneath the geomembrane/geotextile composite provides underdrainage. The geomembrane lining system is anchored at the crest of the reservoirs by embedment in a trench, and at the bottom by maintaining a minimum height of water in the reservoir. Both reservoirs have a diffuse face anchorage system, consisting of anchor strips of PVC embedded in vertical trenches ballasted with compacted granular material, to which the waterproofing liner is anchored by heat-seaming, and a top anchorage consisting of a mechanical anchoring system along the parapet wall.

Installation of the geocomposite system started in 2020. To keep the system from being uplifted during installation, the geodrain and waterproofing liner were temporarily ballasted on the slopes by sandbags, the weight and spacing of which were estimated based on wind loads. The spacing and dimensions of the anchor trenches at both the reservoirs were calculated based on a design wind velocity of 40.5 m/s and 36.0 m/s for the upper and lower reservoirs respectively.

More information on the Kokhav Hayarden PSH can be found in Carpi (n.d.), NS Energy (2020), and Vaschetti et al. (2022).

3.4.7 Panama Canal Third Set of Locks Water Saving Basins (Panama)

The Panama Canal expansion project, completed in 2016, included construction of three new navigation lock chambers on the Atlantic Ocean approach and three new navigation lock chambers on the Pacific Ocean approach to accommodate larger ships traversing the canal. Water from Lake Gatun, a significant freshwater lake between the Atlantic and Pacific locks, is used during operation of the locks. In order to conserve the water supplied from Lake Gatun, water saving basins (WSBs) on each side of the canal, adjacent to the lock chambers, were constructed. Each navigation lock chamber includes three water saving basins (a total of 18 new basins for the project), with an average of 5.5 filling cycles each day. The 18 new basins allow for the reuse of 60% of the water required for each transit. Due to the cyclical filling characteristic, the operation of the basins can be compared to that of a PSH scheme.

The design of the WSBs included a geocomposite waterproofing system with a functional service life of at least 100 years. Due to the long service life, the project requirements for the liner system included considerations of low permeability, settlement, wind and seismic forces, water velocities, traffic loads, and durability. The selected waterproofing liner was a SIBELON CNT 4400 geocomposite (3 mm thick SIBELON geomembrane heat-bonded at fabrication to a 500 g/m^2 nonwoven geotextile) for the slopes, and a SIBELON CNT 3750 geocomposite (2.5 mm thick SIBELON geomembrane heat-bonded at fabrication to a 500 g/m^2 nonwoven geotextile) for the bottom. The slopes of the WSBs are 2H:1V.

The WSB design also incorporates several types of geomembrane anchorage systems, including flexible anchorages in trenches. Anchorage with tensioning trenches was used on the invert of all the basins. On slopes with good geological conditions, the liner was anchored at points, and on slopes with unfavorable geological conditions, the liner was anchored by seaming the geocomposite to PVC anchorage strips embedded in trenches. The total areal coverage of the geomembrane lining system for the WSBs is 596,572m^2.

More information on the water saving basins for the Panama Canal third set of locks can be found in Carpi (n.d.) and Vaschetti et al. (2022).

3.4.8 Abdelmoumen (Morocco)

The 350 MW Abdelmoumen pumped storage project is currently under construction in Morocco. The upper and lower reservoirs are separated by approximately 500 meters of head. The reservoirs will be able to store 1,300,000 m^3 of water.

The reservoirs are formed by compacted earth fill embankment at 2H:1V slopes. The water level fluctuation in both reservoirs is 20 meters. Due to the high exposure to UV radiation, the geomembrane material manufactured for the project was tailored for the climate exposure. The geocomposite liner selected is a Carpi SIBELON CNT 4400 L, consisting of a 3.0 mm thick PVC SIBELON C 3900 L geomembrane heat-bonded during fabrication to a non-woven polypropylene geotextile placed atop a granular drainage layer. The geomembrane includes a surface lacquered treatment designed to increase the liner durability under intense UV radiation. Overall, the project will utilize approximately 195,500m^2 of lining.

The anchorage for the geomembrane lining system is based on embedding geomembrane anchor “wings” at 8 m spacing inside the embankment during construction. The anchor wings have been designed to resist pull-out due to wind and uplift loads. The geocomposite is anchored at the crest and at the toe of the slope in a longitudinal continuous trench backfilled with concrete. Over the bottom of the reservoirs, the geocomposite is anchored within trenches backfilled with concrete as well. An additional anti-puncturing non-woven geotextile is placed above the geocomposite liner inside all the anchorage trenches to protect the liner during casting of the concrete ballast.

Installation of the geomembrane lining system is ongoing as of the writing of this report.

For more information see Vaschetti et al. (2022a) and Vaschetti et al. (2022b).

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4.0 Key Findings and Recommendations for Further Study

4.1 Key Findings

This scoping study consisted of a literature review for pertinent and publicly available information about geomembrane lining materials and their use in PSH reservoirs and an assessment of the applicability of geomembrane lining systems to PSH reservoirs. The literature review and assessment has yielded the following key findings:

- Geomembrane lining systems are one of several lining systems that can be considered for the impervious lining of PSH reservoirs. Other lining systems that have been used for PSH reservoirs include dense asphalt concrete (DAC) and concrete linings. Selection of one lining system over another must consider a number of factors, as summarized in Section 3.1 of this report.

- As discussed in Section 2.0 of this report, the use of polymeric geomembrane materials for the design of dams and reservoirs should be considered only one part of a comprehensive lining system carefully designed to control seepage from a reservoir, whether the reservoir is for PSH or another purpose. For geomembrane lining systems, the geomembrane itself is considered the impervious membrane. However, important considerations in the design and construction of a geomembrane lining system also include the cover and protection of the geomembrane lining, drainage and collection of water that leaks through the geomembrane lining, and support of the geomembrane lining material.

- There are a number of geomembrane materials available in the marketplace that may be suitable for PSH application. Section 2.0 provides a summary of the more commonly used materials; however, selection of a given geomembrane material is subject to a variety of factors. There is not one material that can be considered superior in all factors.

- Detailed design of a geomembrane lining system cannot be completed until a geomembrane lining manufacturer has been selected. For design-bid-build type projects, the owner and engineer will need to decide whether to select the manufacturer during preparation of the overall project design, or leave final design details and selection of the manufacturer to the contractor. For design-build and engineer-procure-construct (EPC) projects, selection of the lining system and geomembrane lining system may be left to the design-build or EPC teams.

- No PSH-specific design guidelines for geomembrane lining systems could be identified in the literature search conducted for this study. The literature is lacking in such guidance.

- No PSH-specific regulations on the use of geomembrane lining systems could be identified in the literature search conducted for this study. For FERC-regulated projects, early interaction with FERC and the use of FERC's risk informed decision making (RIDM) are recommended when a geomembrane lining system is selected for a PSH.

4.2 Recommendations for Further Study

This current report summarizes the result of a scoping study that relied on a literature review of available materials for geomembrane linings, lining systems, and their application to PSH reservoirs and dams. The scope, schedule, and budget for this study was limited. During this study and the development of this summary report a number of topics were identified for further study and investigation, including the following:

- Expand the assessment of PSH reservoir liners to all lining systems. As described in Section 4.1, geomembrane lining systems are one of several lining systems that may be used for the design and construction of lining systems for PSH reservoirs. Additional study would include a comparative assessment of geomembrane lining systems to other lining systems, including dense asphalt concrete (DAC), concrete facings, and clay. The assessment would build on the discussion provided in Section 3.1 of this report.

- Perform a market assessment for potential liner applications by leveraging the results of the recent National Renewable Energy Laboratory study on mapping the PSH resource potential in the U.S. (Rosenlieb, 2022), and the upcoming study led by Oak Ridge National Laboratory on repurposing existing mines for PSH projects.

- Further engage FERC and other relevant agencies to better understand regulatory issues related to geomembrane lining systems:

    - Promote stakeholder engagement with FERC, USBR, USACE, and others to discuss issues pertinent to regulators and other stakeholders relative to the use of geomembrane lining systems for PSH reservoirs.

    - Review and update USBR design standards to include considerations for PSH reservoirs.

    - Perform a generalized potential failure modes analysis (PFMA) and semi-quantitative risk analysis (SQRA) per FERC guidelines to develop a list of general potential failure modes for consideration in the design development of geomembrane lining systems.

- Develop a cost model for pricing geomembrane lining systems for PSH applications. The cost model would be based on general reservoir characteristics, site parameters, and other factors. Based on user inputs, the model will approximate design parameters and calculate the estimated cost of geomembrane lining system for the specific reservoir. Pending discussion with the Department of Energy (DOE), the cost model would be made publicly available to the hydropower industry and stakeholders.

- Develop a preliminary reference design and cost assessment. Develop a case study illustrating the application and preliminary design of a geomembrane lining system for a typical PSH reservoir. This illustrative example would provide information to PSH developers on the design and components of the geomembrane lining system, the installation method and process, regulatory requirements, maintenance needs, etc. The cost model would be applied to estimate the costs of a geomembrane lining system for this hypothetical PSH reservoir. The estimated costs of applying alternative lining systems (e.g., DAC, concrete, etc.) for the same reservoir could also be provided for cost comparison purposes.

One or more of the above follow-on studies would help advance the current state of practice for use of geomembrane lining systems and other lining systems for PSH development.

4.3 References for Section 4

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5.0 Appendix: Literature Review on Polymeric Geomembrane Liners

5.1 General

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