Keynote Lectures

Geosynthetic barriers systems for dams

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ABSTRACT: In more than 270 dams worldwide, geomembranes are the main waterproofing component. The geomembrane is generally associated with other geosynthetics performing various functions, thereby forming a geosynthetic barrier. In this paper, uses of geosynthetic barriers in the various types of dams are reviewed. The types of dams reviewed include: embankment dams (earthfill and rockfill dams), concrete and masonry dams, and roller compacted concrete (RCC) dams. Design and construction aspects are considered, as well as selection of geosynthetic materials and performance (including seepage control and durability). The paper is illustrated using a number of examples of new dams and rehabilitation of existing dams, including examples of the early dams constructed or rehabilitated using geosynthetic barriers in the 1950s, 1960s and 1970s.

1 INTRODUCTION

The use of geosynthetics as water barriers is one of the numerous uses of geosynthetics in hydraulic and geotechnical applications of geosynthetics. The use of geosynthetics in dams is a major application of geosynthetics since "dams have a particular status due to their impact on the environment and on safety", as pointed out by Heibaum et al. (2006). This paper discusses various aspects of design and construction of geosynthetic barriers in dams, and is illustrated by innovative case histories as well as by case histories of historical interest.

The discussions address the geosynthetic barrier as a system, which includes not only the geomembrane barrier but also the other geosynthetic components of the system, such as the geosynthetics providing drainage, support, protection, etc. All types of dams are discussed, listed under three main groups, according to the specific issues pertaining to each group: embankment dams, concrete and masonry dams, and roller compacted concrete (RCC) dams. New construction as well as rehabilitation of dams are addressed, following the same approach adopted by Cancelli & Cazzuffi (1994). Cofferdams and heightening of embankment dams are not discussed specifically, but are mentioned when they represent important milestones in the development of the geosynthetics technology. The same approach is used for tailings dams, which are not discussed specifically, in accordance with a request from the conference organizers.

2 HISTORY AND DEVELOPMENT

2.1 *Early uses of geomembranes*

The use of polymeric liners since the 1950s has been made possible by the availability of synthetic polymers after the war. This availability resulted from the significant increase in production of synthetic rubber and plastics in the 1940s in the United States because supply of natural rubber was quasi impossible due to the war situation in Southeast Asia.

Geomembranes were first used as barriers in hydraulic structures shortly after the Second World War (Scuero & Vaschetti 2009). For example, in the 1940s and 1950s, the US Bureau of Reclamation (USBR) carried out research and field experimentation on various types of canal linings. The first publications of the USBR on the subject were in 1957: on "asphalt membrane canal linings" (Elsperman 1957) and on "Plastic films as canal lining mate-

rials" (Hickey 1957). They were the first publications of the USBR on the materials called today "geomembranes", a terminology proposed twenty years later (Giroud & Perfetti 1977) and adopted worldwide.

The polymeric materials used in these early applications were low density polyethylene (LDPE), polyvinyl chloride (PVC) and butyl rubber. The thermoplastic liners (LDPE and PVC) were very thin: 0.25 mm and even less some times. The thickest thermoplastic liners were typically 0.5 mm (LDPE) and 0.75 mm (PVC). In contrast, as soon as the mid-1950s, butyl rubber liners became available in a variety of thicknesses ranging from 0.5 mm to 2.5 mm. Thus, in 1957, a 3.7 m deep brine reservoir, Daisetta Reservoir, in Texas, USA, was lined with 32,000 m^2 of 2.25 mm thick butyl rubber liner. In 1963, a 16 m deep existing concrete-lined water reservoir, Olinda Reservoir, in Maui (Hawaii, USA), with 1V:1H side slopes, was lined with a 1.5 mm thick butyl rubber liner. A very large reservoir was built in 1969 in Molokai Island (Hawaii, USA): the Kualapuu Reservoir for municipal and irrigation water. This 17 m deep reservoir was lined with 420,000 m^2 of exposed (i.e. unprotected) nylon-scrim reinforced butyl rubber liner, 0.8 mm thick. The exposed butyl rubber liner was preferred to another design which consisted of thinner LDPE or PVC liner covered with a layer of protective soil. The Kualapuu Reservoir was for a long time the largest reservoir in the world with a polymeric liner. However, the exposed portion of the liner, above the water level, was extensively damaged by wind. The slopes above the water level are now covered with a concrete liner on one part of the perimeter and a geomattress (i.e. a geotextile-formed concrete layer) in the other part of the perimeter. The butyl rubber liner, which is still believed to be more or less intact under the water level, is not connected to the concrete liners located above the water level. Extensive leakage is taking place. This project is of the same magnitude as a large dam and its unsatisfactory performance illustrates the need for careful design. Learning from failure, it should be remembered that it is important in design to identify and adequately evaluate the external actions to which the geomembrane can be subjected during its service life. This is particularly important in dam design.

While improperly designed liners fail, properly designed liners may have a long service life. This is the case of the 10 m deep Pont-de-Claix Reservoir, constructed in 1974 with the first double geomembrane liner ever. The concept of a double liner, i.e. two liners with a drainage layer in between to control leakage into the ground by ensuring extremely low head on the secondary liner, had been presented the preceding year (Giroud 1973). A double liner was selected to effectively control leakage from the Pont-de-Claix Reservoir, because this reservoir is located on top of a steep slope whose stability could be impaired by excessive leakage. At the Pont-de-Claix Reservoir, the secondary liner is a bituminous geomembrane made in-situ by spraying hot bitumen on a needle-punched nonwoven geotextile, the drainage layer is made of rounded aggregate (stabilized with cement on the 1V:2H slopes), and the primary liner is a 1.5 mm thick butyl rubber geomembrane. A polyester needle-punched nonwoven geotextile was used between the geomembrane and the aggregate. The butyl rubber geomembrane is exposed and the reservoir is still in service. Learning from success, it should be remembered that the consequences of liner failure should always be considered in design. This is particularly important in dam design.

In 1972, the first high density polyethylene (HDPE) geomembrane was used in a reservoir in Germany. Thanks to their excellent durability and chemical resistance, HDPE geomembranes have rapidly become the most widely used geomembranes in waste disposal landfills worldwide. HDPE geomembranes are also extensively used in reservoirs for industrial liquids. As a result, many engineers are familiar with HDPE geomembranes and are tempted to use them in hydraulic structures, including dams. However, the use of HDPE geomembranes in dams has been limited due to their rigidity, which restricts their ability to adapt to large deformations of the supporting material caused by high hydrostatic pressures involved in dams.

In the 1960s and 1970s, PVC liners were extensively used to line reservoirs in many countries. In many cases, thin and inexpensive PVC liners were used to line reservoirs for non-technical applications. Such reservoirs had a limited service life, which led some to conclude that PVC liners should be reserved to low-tech applications with limited service life. Also, some engineers, unaware of the wide range of durability of available PVC geomembranes, have selected inadequate PVC geomembranes for technical applications where a high-performance PVC liner should have been used. As a result, in the early 1970s, it would have been difficult to predict that the highest level of performance would be achieved in dams by PVC geomembranes.

The use of geomembranes in reservoirs has paved the way for the use of geomembranes in dams. Successful reservoir projects have help build confidence in geomembranes. Lessons learned from problems and failures associated with reservoirs make it possible to avoid the same problems or failures, which could be catastrophic in the case of dams.

2.2 *Concept of using geomembranes in dams*

While the concept of using geomembranes in dams instead of conventional impervious materials such as clay, cement concrete (hereafter simply called concrete) or bituminous concrete, obviously derived,

among other considerations, from the successful use of geomembranes in canals and reservoirs, the credibility of synthetic materials in dams had been established by the good performance of embedded PVC waterstops in a very large number of concrete dams worldwide. In those dams, waterstops play an essential role by preventing water seepage through joints that are indispensable to accommodate concrete expansion and contraction. A geomembrane placed on the upstream face of a dam or inside a dam can be considered, from a conceptual viewpoint, as one wide waterstop sealed at the abutments and the bottom.

The first applications of geomembranes in dams took place in new embankment dams because many of these dams, being too permeable, required a separate element to provide imperviousness. In many cases, it appeared that geosynthetic barrier systems were more economical and easier to install than traditional impervious materials such as clay, concrete or bituminous concrete.

Geomembranes were used in embankment dams before they were used in concrete dams probably for two reasons: (i) because geomembrane installation on the slope of an embankment dam is similar to installation on the slopes of a pond, an application where geomembranes has been used since the late 1950s; and (2) because installation on a gentle slope is less demanding than installation on a vertical face.

2.3 *Early uses of geomembranes in dams*

It is important to study the early uses of geomembranes in dams, because this provides an opportunity to learn from successes and failures with the benefit of hindsight, concerning in particular: geomembrane selection, geomembrane thickness, geomembrane durability, geomembrane protection, design concepts, design details (which are so important in dams), failure modes of geomembranes used in dams, and geotechnical failure modes associated with geomembranes.

Contrada Sabetta Dam, constructed in 1959 in Italy, is the first example of use of a geomembrane as the only water barrier in a dam (Cazzuffi 1987). Contrada Sabetta Dam is a 32.5 m high rockfill dam with a crest length of 155 m. It is a dam of a special type: rocks were arranged as dry masonry, which made it possible to achieve very steep slopes, 1V:1H upstream and 1V:1.4H downstream. The geomembrane used was 2.0 mm thick and made of polyisobutylene, an elastomeric compound that is no longer used as a geomembrane today, not because of any performance problem, at least when it is covered, but essentially because modern geomembranes are easier to seam. A general cross section of Contrada Sabetta Dam is shown in Figure 1 and a detailed cross section is shown in Figure 2. Special features of Contrada Sabetta Dam included the following:

- The geomembrane barrier consisted of two layers of identical geomembranes placed on top of each other over the entire upstream face of the dam. (It is important to note that two geomembranes (or two liners of any type) placed on top of each other, without a drainage layer in between, do not form a double liner.) A total of 3900 m^2 of geomembrane were used on the 1900 m^2 upstream face. These two geomembrane layers were glued to each other along the edges.
- The lower geomembrane layer was glued using a bitumen adhesive on the 0.1 m thick supporting material made of porous concrete. The porous concrete rests on 0.25 m thick reinforced concrete slabs, resting on the dry masonry.
- The geomembrane barrier was covered by unreinforced 2 m × 2 m concrete slabs, 0.20 m thick, cast on site. The joints between adjacent slabs were left open, 1 mm wide, and were not filled by any porous material, to allow for free circulation of water and to provide some flexibility in case of settlement. There was a sheet of bituminous paper-felt between the concrete slabs and the upper geomembrane layer to protect the geomembrane during the casting of the concrete slabs.

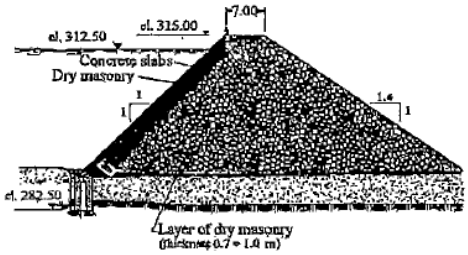


Figure 1. General cross section of Contrada Sabetta Dam (modified after Cazzuffi 1987 and ICOLD 1991).

In 1960, a geomembrane was used at Dobsina Dam, a 10 m high earthfill dam in Slovakia with a crest length of 204 m. At Dobsina Dam, according to the scant information available, a 0.9 mm thick PVC geomembrane was placed on the 1V:2.5H upstream face. The geomembrane was then covered with paper and prefabricated concrete slabs. A total of 850 m^2 of geomembrane were used.

According to the information available from ICOLD, seven years elapsed before geomembranes were again used in dams, and again it was on embankment dams. In 1967, Miel Dam was constructed in France. It is a 15 m high earthfill dam with a crest length of 130 m. A 1 mm thick butyl rubber geomembrane was placed on the 1V:2.5H upstream slope. The geomembrane was covered with 0.20 m

of sand and gravel overlain by 0.70 m of rip rap. A total of 2700 m^2 of geomembrane was installed in that dam.

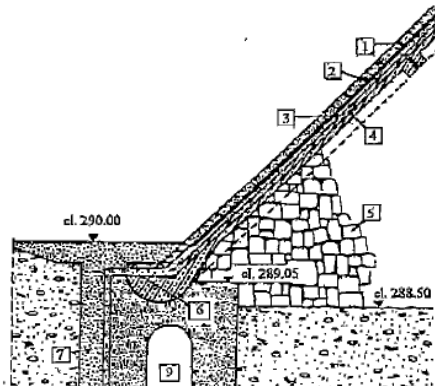


Figure 2. Detailed cross section of upstream toe of Contrada Sabetta Dam (modified after Cazzuffi 1987 and ICOLD 1991): [1] 0.20 m thick concrete slabs, [2] one sheet of bituminous paper-felt + two sheets of polyisobutylene geomembrane (2.0 mm thick) + bituminous adhesive, [3] porous concrete (0.10 m thick), [4] reinforced concrete slabs (0.25 m thick), [5] dry masonry (thickness ranging from 2.00 to 3.00 m), [6] joint between plinth and upstream facing, [7] plastic concrete diaphragm wall, [9] inspection and drainage gallery.

In most of the early projects, the geomembrane was installed during construction of the dam, on the upstream slope and covered. The first exceptions were: (1) the first geomembrane used inside a dam was a CPE geomembrane used in 1970 on a 1V:0.67H slope (56º) inside a rockfill dam (Odiel Dam in Spain) with upstream and downstream slopes of 1V:1.3H; (2) the first geomembrane used for repairing a dam was a 0.9 mm thick PVC geomembrane installed in 1971 at Obecnice Dam, an earthfill dam in the Czech Republic; and (3) the first geomembrane used exposed on the face of an embankment dam was a 4 mm thick bituminous geomembrane installed in 1973 at Banegon Dam, a 17 m high earthfill dam in France. It was also the first use of a bituminous geomembrane in a dam.

2.4 *A famous but special case history*

Mission Dam (now called Terzaghi Dam), a 55 m high rockfill dam constructed in 1960 in Canada, is a special case for both historical and technical reasons. It is a special case for historical reasons because Karl Terzaghi was the designer and because it was one of the first dams including a geomembrane. It was a special case for technical reasons because the geomembrane application was not typical. A detailed description of the construction of the dam was provided by Terzaghi & Lacroix (1964) and a description of the geomembrane installation was provided by Lacroix (1984).

Special features of Terzaghi Dam relevant to the geomembrane application are the following:

- The water barrier in the dam was a 1.5 m thick clay layer covered by 2 m of rubble fill.
- Large differential settlement was expected and the clay layer was designed to be convex in order to remain in compression when settlement occurs.
- However, at the transition between the 15º slope and the quasi-horizontal area near the toe of the upstream face, the clay layer was necessarily concave.

In this 10,000 m^2 transition zone, a geomembrane was used to prevent the clay from cracking. The function of the geomembrane can be described as follows. The geomembrane, placed on top of the clay, ensures that a uniform pressure equal to the hydrostatic pressure is applied on the clay. The hydrostatic pressure being higher than the compressive strength of the clay, clay cracking is prevented. Furthermore, if some cracks develop in spite of the normal pressure applied by the geomembrane, water does not penetrate into the cracks and, therefore, no hydrostatic pressure is applied within the cracks.

In contrast, without geomembrane, as cracks tend to develop as a result of differential settlement, water would penetrate in the cracks. The hydrostatic pressure, acting on both sides of cracks, would open the cracks and cause the cracks to propagate through the entire thickness of the clay layer, thereby impairing the clay barrier function of the clay layer.

A PVC geomembrane was selected because of its large elongation before rupture. The PVC geomembrane selected was 0.75 mm thick, which was considered, particularly in North America, to be a thick PVC geomembrane at that time. It was smooth on one side and embossed on the other side.

Clearly, Terzaghi Dam is an interesting use of a geomembrane in a dam, which can provide an example for some applications in the future. However, it cannot be considered as a precursor of modern applications of geomembranes in dams.

2.5 *Types of geomembranes used in early dams*

In the pioneer applications of geomembranes in dams, different types of geomembranes available in the market were used. The following geomembranes were used in early embankment dams:

- Contrada Sabetta (1959), Italy, 32.5 m high, 155 m long, polyisobutylene, 2.0 mm, covered
- Dobsina (1960), Slovakia, 10 m high, 204 m long, PVC, 0.9 mm, covered.
- Miel (1968), France, 15 m high, 130 m long, butyl rubber, 1.0 mm, covered.

- Odiel (1970), Spain, 41 m high, length unknown, CPE, thickness unknown, inside the dam on a 56º slope.
- Obecnice (1971), Czech Republic, 16 m high, 370 m long, constructed in 1966, and repaired in 1971 with a PVC geomembrane, 0.9 mm, covered
- Wenholthausen (1971), Germany, 17 m high, 100 m long, PVC, unknown thickness, covered.
- Neris (1972), France, 18 m high, 100 m long, butyl rubber, 1.5 mm, covered.
- Bitburg (1972), Germany, 13 m high, 95 m long, PVC, unknown thickness, covered (the geomembrane became brittle and was punctured by the edges of its protecting slabs; it was replaced by an HDPE geomembrane in 1978).
- Landstein (1973), Czech Republic, 26.5 m high, 376 m long, PVC, 1.1 mm, covered.
- Banegon (1973), France, 17 m high, unknown length, bituminous, 4 mm, exposed (in the lower part of the dam face, where hydrostatic pressure is higher, the geomembrane was punctured by stones from the supporting soil and defective seams opened; in this area, the geomembrane was replaced by the same geomembrane on a needle-punched nonwoven geotextile).
- Herbes Blanches (1975), Ile de la Reunion, France, 13 m high, 85 m long, butyl rubber, 1.0 mm, exposed (the geomembrane was torn and displaced by cyclone winds, and burst on cavities at the toe of the dam shortly after construction; the geomembrane was replaced in 2003 by a PVC composite geomembrane consisting of a 2.0 mm PVC geomembrane laminated with a 800 g/m^2 needle-punched nonwoven geotextile).
- Twrdosin, (1977), Slovakia, 16 m high, 307 m long, PVC, 0.9 mm, covered.
- L'Ospedale (1978), Corsica, France, 26 m high, 135 m long, bituminous, 4.8 mm, covered.
- Avoriaz (1979), France, 11 m high, 135 m long, bituminous geomembrane, 4 mm, exposed (the geomembrane failed in the reservoir due to localized collapse of the supporting soil and improper seaming, and was replaced by 2.0 or 2.5 mm HDPE geomembrane in 1981).
- Gorghiglio (1979), Italy, 12 m high, 125 m long, PVC, 2 mm, exposed on the slopes and covered on the bottom of the reservoir.
- Mas d'Armand (1981), France, 21 m high, 403 m long, bituminous, 4.8 mm, covered.
- Kyiche (1983), Czech Republic, 17.5 m high, 1660 m long, polymeric made in situ, unknown thickness, covered.
- Trnavka (1983), Czech Republic, 21 m high, 165 m long, polymeric made in situ, unknown thickness, covered.
- Codole (1983), Corsica, France, 28 m high, 460 m long, PVC composite geomembrane, consisting of a 1.9 mm PVC geomembrane bonded to a 400 g/m^2 needle-punched nonwoven geotextile, covered.
- La Lande (1983), France, 17 m high, 80 m long, bituminous, 4.8 mm, covered.
- Rouffiac (1983), France, 12.5 m high, 157 m long, bituminous, 4.0 mm, covered in upper part and exposed to water in the lower part.

A number of lessons can be learned from these early dams:

- There has been a strong regional influence in the choice of geomembrane, which confirms the well-known fact that the engineers designing dams are inspired by precedents. For example: approximately 1 mm thick PVC geomembranes used in Central Europe (Czech Republic, Slovakia, Germany), with the replacement of a failed PVC geomembrane by an HDPE geomembrane typical of the emergence of HDPE in the mid-1970s in Germany; butyl rubber, then bituminous geomembranes in France. Another example of regional influence will be seen in Section 2.6 with the use of PVC geomembranes on concrete dams in the Italian Alps.
- Failures have occurred for a variety of reasons: inadequate seams, puncture by stones, damage by wind in the case of an exposed geomembrane, localized collapse of the supporting soil, and aging of the geomembrane. It is important to note the rapid aging of a PVC geomembrane in five years (1972-1977) in one of the dams mentioned above: PVC is a rigid material that must be made flexible to be used as a geomembrane; depending on the method and products used to "plasticize" PVC, the durability of the geomembrane can vary from a few years to more than 50 years (see Section 8.1). Therefore, it is of utmost importance to properly select a PVC geomembrane.
- Some types of geomembrane tend to become less used such as the elastomeric geomembranes (polyisobutylene, butyl rubber) in spite of their good mechanical properties and durability, but because they are difficult to seam.
- In most cases the geomembrane was covered for protection against a variety of external actions such as: wind, waves, floating debris, ice, vandalism, temperature variations, UV radiation, etc. Today, still, the majority of geomembranes used in embankment dams are covered.

2.6 *From embankment dams to all types of dams*

2.6.1 *Rehabilitation of old concrete dams*

In the 1970s, the use of geomembranes was extended to the rehabilitation of concrete dams. The first projects were made on dams situated at high elevation in the Italian Alps, where traditional facings (shotcrete and concrete) were susceptible of quick ageing caused by frequent freeze-thaw cycles, low temperatures and ice action. As previous experience on embankment dams was satisfactory, geomembranes technology had improved and confidence in the materials had increased, it was estimated that a robust geomembrane could sustain such environment.

The first application of a geomembrane on a concrete dam was made in 1971 at Lago Baitone Dam, Italy, a 37 m high, 227 m long, concrete gravity dam constructed in 1930 at elevation 2280. The geomembrane was a 2.0 mm polyisobutylene and was left exposed on the quasi-vertical face of the dam. There was no system to prevent the ice from adhering to the geomembrane. The geomembrane was damaged by ice and floating wood and was replaced in 1994 by a 2.0 mm PVC geomembrane laminated in factory to a needle-punched nonwoven geotextile providing protection and drainage.

The first entirely successful applications of concrete dam rehabilitation were made in 1976 at Lago Miller Dam, Italy, an 11 m high gravity dam constructed in 1926 at elevation 2170, where a 2.0 mm thick PVC geomembrane was adopted, and in 1980 at Lago Nero Dam, Italy, a 45.5 m high gravity dam constructed in 1929 at elevation 2027, where a composite geomembrane consisting of a 2.0 mm thick PVC geomembrane laminated to a 200 g/m^2 needle-punched nonwoven geotextile was adopted. The Lago Nero Dam represents the first use of this type of composite geomembrane on a dam. This type of geomembrane will then be used successfully on many dams.

All pioneering applications of PVC geomembranes to rehabilitate concrete dams were located in the Italian Alps, at more than 2000 m elevation. In the 1970s and 1980s, a total of eight large dams were thus rehabilitated, all in the Italian Alps. It is interesting to note that that the age of the concrete dam, when rehabilitation was deemed necessary ranged from 15 to 60 years, with the most typical values around 50 years. It will be seen in Section 8 that the durability of well selected geomembranes can be superior to 50 years under the same exposure conditions. In other words, well selected geomembranes are no less durable than traditional construction materials such as concrete.

At all of these dams, the PVC geomembrane was left exposed to the environment, which at such elevation is quite demanding in terms of resistance to UV rays, to freeze-thaw cycles, to extremely low temperatures, and to high daily and seasonal temperature excursions. For these reasons, relatively thick PVC geomembranes were used, i.e. thicknesses of 2.0 and 2.5 mm, which contrasts with the thicknesses of 0.75 or 0.5 mm or less used in many ponds as indicated in Section 2.1, particularly in North America, or even the 0.75 thickness used at Terzaghi Dam (see Section 2.4). Indeed, it was suspected that, since one of the mechanisms of aging is the migration of some constituents of the polymeric compound outside the geomembrane, durability might be proportional to the square of thickness by analogy with phenomena such as soil consolidation or heat transfer.

The exposed position on quasi-vertical dam faces required anchoring of the geomembrane to the dam against displacement of the geomembrane (by wind, waves etc.) and sagging of the geomembrane due to gravity. At Lago Nero Dam, for the first time an anchorage system was adopted that later became the most used one in the rehabilitation of concrete dams, as will be discussed in Section 4.4.

The considerable experience gained in the rehabilitation of concrete dams in Italy had (and still has) a major influence on the use of geomembranes in dams, and not only concrete dams. For example, Codole Dam, a major rockfill dam built in France in 1983, uses a composite geomembrane consisting of a 1.9 mm PVC geomembrane bonded to a needle-punched nonwoven geotextile, i.e. a geomembrane similar to those used in the rehabilitation of concrete dams in Italian Alps.

Another example is the use of the same technique for waterproofing RCC dams (see Section 5). More generally, the quality of the geomembranes and the quality of installation of these geomembranes for concrete dam rehabilitation have significantly contributed to establishing the credibility of geomembrane use in dams of all types.

2.6.2 *Waterproofing of RCC dams*

At the beginning of the 1980s the construction of roller compacted concrete (RCC) dams started. These dams use construction methods similar to those of embankment dams, and materials typical of concrete dams.

As will be discussed in Section 5, using a geomembrane can provide substantial practical and financial benefits in RCC dam construction. Therefore, as early as 1984, a geomembrane was adopted as water barrier for the construction of Carrol Ecton Dam (formerly Winchester Dam), a new RCC dam in the USA; and, in 2000, geomembranes started being used for the local repair of existing RCC dams (failing joints, cracks). The association of RCC and geomembranes has been the most significant innovation in the art of constructed dams.

2.6.3 *Underwater installation*

Another milestone is the first underwater installation of a geomembrane used for a dam carried out in 1997 at Lost Creek Dam, an arch dam in the USA. Underwater installation will be discussed in Section 6.

2.6.4 *Current situation*

At present, geomembranes are adopted all over the world to waterproof all types of dams, as well as all types of hydraulic structures (reservoirs, ponds, canals, hydraulic tunnels, surge shafts, pumped storage reservoirs, forebay reservoirs, underground tanks, etc.), with a total of several hundred millions of square meters installed. Geomembranes have been perhaps the most innovative novelty in the field of hydraulic structures in the past 50 years.

According to the ICOLD database, geomembranes are the only waterproofing element in almost all of the 280 dams compiled. There is no limit to the water head they can withstand. Thus, the world records are at present:

- 198 m for new embankment dams (Karahnjukar Dam, Iceland, 2006, toe wall and horizontal joint between Phase 1 and Phase 2 face slabs),
- 188 m for new RCC dams (Miel I, Colombia, 2002),
- 200 m for new concrete dams (Kolnbreinsperre, Austria, 1985, partial application at heel), and
- 174 m for rehabilitation of concrete dams (Alpe Gera, Italy, 1994).

2.7 *The ICOLD bulletins*

ICOLD, the International Commission on Large Dams, has dedicated two theme bulletins to the use of geomembranes in dams, one in 1981 (Bulletin 38, ICOLD 1981) and one in 1991 (Bulletin 78, ICOLD 1991).

Some concepts and conclusions of these two bulletins have been presented by Scuero & Vaschetti (2006). Bulletin 135, the new theme bulletin on geomembranes, is under editing and will be published shortly. The scope and contents of Bulletin 135 have been discussed by Heibaum et al. (2006) and by Scuero & Vaschetti (2006).

Some information contained in this paper, including tables, is based on the database prepared for Bulletin 135 (ICOLD 2010).

Since 2006, the database has undergone periodical updating and the number of dams has increased. It is foreseen that ICOLD will make the updated database available to the community of engineers and scientists involved.

The authors hope that updating and availability of the database will continue over the years, to provide valuable information to the worldwide engineering community.

3 EMBANKMENT DAMS

3.1 *General considerations*

3.1.1 *Geomembranes and other barriers*

Geomembranes have been used in 167 large embankment dams, according to the ICOLD database. This is a remarkable success achieved over 50 years since the first geomembrane was used in an embankment dam (see Section 2.3), and in fact mostly in the past three decades. In embankment dams, geomembranes are in competition with traditional barriers such as: clay cores, upstream faces made of concrete slabs or bituminous concrete layers, and vertical barriers made of made of concrete, bituminous materials, or low-permeability slurry or cement-bentonite.

3.1.2 *Advantages of geomembranes*

Geomembranes are increasingly used because they have numerous advantages over traditional barrier materials: cost, imperviousness, construction, and practical considerations. These advantages are discussed below.

Geomembranes are significantly more impervious than all other barrier materials. This geomembrane property is essential for the containment of liquids that could contaminate the ground or the ground water, but typically is not considered essential in dams. However, with the growing emphasis on water conservation, it is likely that the superior imperviousness of geomembranes will be considered to be a significant asset of geomembranes in the future.

Some geomembranes can undergo large strains (e.g. 100% or more) without rupture. A geomembrane with a high elongation capability will maintain watertightness in presence of differential settlements and movements that could cause: (1) cracking of concrete slabs in concrete face rockfill dams and, in extreme cases, could cause failure of the waterstops; and (2) cause disruption in the connection of the bituminous facing to the concrete structures in the case of bituminous concrete face rockfill dams. Concerning dams with clay core, the imperviousness of the core heavily relies on construction quality (too often influenced by weather conditions) and on the skill of the contractor. It can therefore be said that geomembranes can improve safety of embankment dams because they are engineered to maintain imperviousness in presence of events that could impair the performance of other waterproofing systems.

Geomembrane waterproofing barriers can provide substantial advantages in the construction of embankment dams as compared to traditional waterproofing barriers, because they avoid problems such as lack of suitable materials and deterioration of waterstops. Also, they simplify construction by elimi-

nating the need for installing multiple lines of waterstops and by being easier to connect to ancillary concrete structures than clay cores or bituminous concrete.

Construction times and constraints are reduced when geomembrane barriers are used. With traditional barrier materials, the impact that the installation/construction of the face slabs, or the placement/compaction of the impervious core, can have on the overall construction schedule, and the complexity of the techniques needed to construct the waterproofing system, must be taken into consideration when evaluating times of completion. In dams with a central core (made of clay or bituminous material), a crucial point is that, the construction of the dam body and the construction of the central core being related, the constraints imposed by the weather conditions, or any disruption in the placement of the filter material, will affect the rate of construction of the entire dam body. On the contrary, installation of a geomembrane barrier system can be scheduled in function of the general schedule of construction, and is not significantly affected by weather.

3.1.3 *Uses of geomembranes in embankment dams*

A variety of geomembranes are used in embankment dams. As indicated in Section 3.1.1, based on the ICOLD database, geomembranes have been used in a total of 167 embankment dams. The breakdown per type of geomembrane is as follows:

- PVC, 76 (45%),
- LLDPE, 25 (15%)
- Bituminous, 18 (11%)
- HDPE, 13 (8%)
- Butyl rubber and other elastomers, 11 (7%)
- CSPE, 7 (4%)
- PP, 6 (4%)
- Others, 2 (1%)
- In situ, 9 (5%)

It should be noted that this breakdown is only approximate, because the ICOLD database includes a variety of applications of geomembranes in embankment dams that could be considered of lesser importance, such as: geomembranes used in some small dams, geomembranes used over a height of only a few meters to heighten existing dams, very thin geomembranes (i.e. 0.7 mm or less), which would not normally be used in permanent dams, etc. If only geomembranes with a thickness greater than 0.7 mm are considered, the total number of embankment dams with such geomembranes is 126, and the breakdown per geomembrane type is as follows:

- PVC, 54 (43%),
- Bituminous, 18 (14%)
- HDPE, 13 (10%)
- Butyl rubber and other elastomers, 11 (9%)
- CSPE, 7 (6%)
- LLDPE, 6 (5%)
- PP, 6 (5%)
- Others, 2 (2%)
- In situ, 9 (7%)

Comparing the two above breakdowns, it appears that thin LLDPE geomembranes are used in a number of dams that probably do not meet the standards for high quality dams.

3.2 *Design concepts for new dams*

Two important design concepts are discussed in Section 3.2: location of the geomembrane, and type of liner system. These concepts apply only to new dams. The case of dam rehabilitation is addressed in Section 3.6.

3.2.1 *Location of the waterproof barrier*

In an embankment dam, two positions can be considered for the geomembrane barrier: (1) the geomembrane can be at the upstream slope, covered or not; or (2) the geomembrane can be internal, i.e. located inside the dam body, either inclined inside the upstream zone of the dam, or vertical or quasi vertical, in a central position.

Based on the ICOLD database, in approximately 90% of the dams where a geomembrane is used, it is at the upstream slope and in approximately 10% it is internal. Among the geomembranes used at the upstream slope: 70% are covered, and 30% are exposed.

The advantages and drawbacks of upstream and internal geomembranes are discussed below.

A vertical geomembrane is significantly smaller than a geomembrane installed on the upstream slope. For example, the size of a vertical geomembrane is half the size of a geomembrane installed on a 1V:1.7H upstream slope. However, this benefit is lost in great part if the geomembrane is quasi-vertical with an “accordion” (or “zigzag”) shape. Also, whether the geomembrane is vertical or quasi-vertical, the increased installation cost may offset the reduced purchase cost. Furthermore, both vertical and quasi-vertical geomembranes have the following drawback: the hydrostatic pressure from the impounded water is horizontal, which is much less favorable to the stability of the dam than the inclined pressure applied in the case of a geomembrane located on the upstream slope. More precisely, calculations show that: (1) the horizontal component of the force applied by water is the same regardless of the position of the geomembrane; and (2) the vertical component deceases when the inclination increases. Since the vertical component contributes to the stability of the dam, the risk of horizontal sliding of the dam is greatest if the geomembrane is vertical or quasi-vertical.

Two additional advantages, with respect to the stability of the dam, of a geomembrane along the upstream face are the following: (1) the entire weight of the dam contributes to stability; and (2) there is no pore water pressure in the embankment.

The construction of a dam with a vertical geomembrane or an accordion-shaped quasi-vertical geomembrane is generally more difficult than the construction of a dam with an upstream geomembrane. In particular, connections of an accordion-shaped geomembrane with ancillary structures are difficult. In cofferdams with no ancillary structures, an accordion-shaped quasi-vertical geomembrane can be a good solution.

It should be noted that, if the reservoir is lined, it is logical to place the geomembrane on the upstream slope.

Since it is generally not advantageous to select a vertical (or quasi vertical geomembrane), this configuration is very rarely used. This configuration will be discussed and examples will be presented in Section 3.5.

When a geomembrane is used on the upstream slope, it can be exposed or covered (i.e. "protected") by a layer of heavy material such as soil, concrete, etc. The case of exposed geomembranes will be discussed in Section 3.3 and the case of covered geomembranes will be discussed in Section 3.4.

3.2.2 *Liner system concept for leakage control*

The design of a dam with a liner (any type of liner) should be such that the seepage resulting from a major breach in the liner should not cause the rupture or a major distress of the dam. Therefore, the various zones that constitute a dam should comprise adequate filters to prevent internal erosion of the dam. This is particularly important when the liner is an exposed geomembrane because geomembranes can be breached accidentally. Geomembranes are significantly more waterproof than concrete or clay, but they can be damaged by some mechanical actions.

If the dam does not meet the conditions indicated above and is sensitive to internal erosion in case of seepage, a possible solution consists in minimizing the rate of leakage through the liner system, even in case of a breach in the geomembrane. Two possibilities for minimizing leakage are the use of a double liner; and or the use of a composite liner.

However, if an embankment dam is properly designed, it should not be sensitive to internal erosion. Therefore, the need for strict leakage control (i.e. for a double liner or a composite liner) is rare in the case of a dam. In contrast, in the case of a reservoir, the need for strict leakage control is more frequent because the natural ground beneath a reservoir is not controlled by the design engineer and can be sensitive to internal erosion.

Composite liners and double liners are discussed in the next sections.

3.2.3 *Composite liner for leakage control*

The term "composite liner" could have several meanings. It is generally used to designate a liner composed of a synthetic component and a mineral component. The most frequent type of composite liner consists of a geomembrane and a layer of compacted low-permeability soil such as clay. Composite liners significantly reduce the rate of leakage through geomembrane defects compared to a geomembrane alone (Giroud 1997; Giroud & Bonaparte 1989; Touze-Foltz & Giroud 2003).

It is important to note that a composite liner should not be used on the upstream slope of an embankment dam. This is because during normal operation, in case of a leak, even small, through the geomembrane, water may accumulate in the space between the geomembrane and the soil component of the composite liner. In case of rapid drawdown of the reservoir, the pressure of the water entrapped between the two components of the composite liner is no longer balanced by the pressure of the water in the reservoir. Depending on the amount of water entrapped between the two components of the composite liner, and the weight of material above the composite liner, instability of the upstream slope may occur at the interface between the two components of the composite liner. Even if instability does not occur, the geomembrane and the materials above the geomembrane may be uplifted, which may have detrimental consequences such as permanent deformations or cracking.

Therefore, if a composite liner is used in a dam, the weight of materials on top of the geomembrane should be sufficient to exceed the pressure of the water likely to be entrapped between the two components of a composite liner. Furthermore, the normal stress applied by the materials located on top of the geomembrane, reduces the amount of water likely to be entrapped under the geomembrane.

The conclusion of this discussion is that a composite liner (or any two superposed liners) can be used in a dam only if sufficient load is placed on the geomembrane. This conclusion applies to all cases where a liner is placed on top of another liner (which should not be done, if possible). This conclusion even applies to cases where two low-permeability layers are placed on each other. There is one possible exception to this conclusion: the use of discharge valves is currently envisioned for a project to release water that could accumulate beneath a geomembrane liner. It will be interesting to observe the performance of this system.

Geomembrane uplift by entrapped water occurred at Worster Dam. This took place at the first drawdown of the reservoir after geomembrane installation, in spite of the presence of a 0.3 m thick soil cover (see Section 3.4.6 (1)) (Johnson 2010).

It should be noted that that this problem had been identified as soon as composite liners were used. As

pointed by Giroud & Bonaparte (1989): "Composite liners must be used with caution in liquid containment facilities. If the geomembrane component of the composite liner is directly in contact with the contained liquid (in other words, if the geomembrane is not covered with a heavy material such as a layer of earth or concrete slabs), and if there is leakage through the geomembrane, liquids will tend to accumulate between the low-permeability soil (which is the lower component of the composite liner) and the geomembrane, since the submerged portion of the geomembrane (whose specific gravity is close to 1) is easily uplifted. Then, if the impoundment is rapidly emptied, the geomembrane will be subjected to severe tensile stresses because the pressure of the entrapped liquids is no longer balanced by the pressure of the impounded liquid. Therefore, a composite liner should always be loaded, which is automatically the case in a landfill or in a waste pile, and which must be taken into account in the design of a liquid containment facility."

Essentially, in new embankment dams, the layers underlying the geomembrane liner should be sufficiently permeable to avoid accumulation of water. In contrast, in the rehabilitation of dams (of any type), it should be recognized that the layer on which the geomembrane is to be installed often has a low permeability and, therefore, a drainage layer is needed between the geomembrane and the supporting layer.

As a final note, it is important to remember that the stability against gravity (and, in some areas, against seismic forces) of any layer added to a dam should be checked. This implies an accurate measurement of interface shear strengths at all interfaces.

3.2.4 *Double liner for leakage control*

As mentioned in Section 2.1, the concept of double liner has been used successfully for reservoirs, and it is used routinely for waste disposal landfills. One may wonder if this concept could, or should, be used for dams. As explained in Section 2.1, the concept of a double liner is as follows: two liners with a drainage layer in between to control leakage into the ground by ensuring extremely low head on the secondary liner. The extreme situations where a double liner is needed are extremely rare for embankment dams, but not for reservoirs, as indicated in Section 3.2.2. An example is presented in Section 3.2.5 and is related to Les Arcs Reservoir.

3.2.5 *Example of reservoir dam with double liner*

(1) The project

A double geomembrane liner system has been used in 2008 at Les Arcs Reservoir, France, a facility that comprises two embankment dams and a 400,000 m^3 reservoir entirely lined. Used for the production of artificial snow at an elevation of 2200 m, the reservoir is formed by two embankment dams having a maximum height of 21.7 m and 10 m, respectively. Due to the high permeability and the sensitivity to erosion of the terrains, the designer selected a double geomembrane liner system with a double drainage system, to ensure that seepage through the primary geomembrane would be intercepted by the primary drainage system, thereby reducing the risk of leakage into the ground, which could cause internal erosion. Indeed, it was important to take all precautions to ensure the safety of this structure built immediately above populated residential areas.

(2) Alternative design

To decrease construction time (i.e. to ensure that the liner system could be constructed in the two months time allotted by extreme climatic conditions at the site even in summer) and to reduce costs, the waterproofing contractor proposed to modify the original design including two gravel drainage layers each 0.15 m thick, so as to adopt exclusively geosynthetic components. In total nine different types of geosynthetics are used in the liner system, either independently or in combination to form geocomposites. The cross section of the liner system used on the slopes is shown in Figure 3.

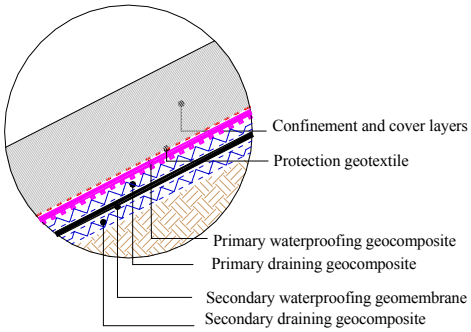


Figure 3. Cross section of the double geomembrane liner system on the slopes of Les Arcs Dam.

The geosynthetics that constitute the double liner system on the slopes are described below, from the upper to the lower component:

- The primary liner is a composite geomembrane that consists of a 2 mm thick PVC geomembrane, laminated during fabrication to a 500 g/m^2 geotextile that ensures adequate friction with the spikes of the underlying geonet. This geomembrane is approved for potable use in accordance with the French regulation.
- The primary draining geocomposite, formed by a spiked geonet having a geotextile on the bottom side in contact with the spiked secondary geomembrane, collects and monitors water that would leak through the primary liner.

- The secondary liner is a 1.2 mm thick very low density polyethylene (VLDPE) geomembrane with spikes on both sides that provide high friction with the geotextiles below and above. It separates the primary drainage system (which drains leakage through the primary liner) from the secondary drainage system (which drains seepage from the ground).
- The secondary draining geocomposite, formed by a high transmissivity geonet having on both sides a geotextile that enhances friction, collects water drained from the ground.

At the bottom of the reservoir, the liner system is also a double liner, but the materials are slightly different, because high interface shear strength is not a requirement:

- The primary geomembrane is the same but without geotextile because high friction is not necessary.
- The secondary geomembrane has the same thickness and composition as the one on the slopes but it has no spikes, again in consideration of friction.
- The draining geosynthetics are slightly different because the only requirement is draining capacity and not friction.

(3) Approval of the alternative design

To obtain approval by the French Permanent Technical Committee of Dams and Hydraulic Structures under which jurisdiction the reservoir stands, due to the potential impact of the reservoir on public safety, the drainage capability and the interface shear strengths had to be validated by two laboratories in France and one in Italy (Delorme et al. 2009). The whole system was then evaluated for feasibility and reliability in a full scale trial on four test embankments for the slopes, each about 60 m long and 4 to 6 m wide, and on two 10 m long and 6 m wide test areas for the bottom. The proposed alternative design was approved.

(4) Design details

The liner system was covered on the slopes and the bottom with a 700 g/m^2 needle-punched nonwoven geotextile overlain by a 0.3 m layer of 0-30 mm gravel, overlain by a 0.5 m layer of 0-400 mm rocks.

The geosynthetics composing the double liner system and the protection geotextile are anchored at the crest in a trench. At all concrete appurtenances (free flow spillway, outlet) the geomembrane system is anchored with the mechanical tie-down type anchorage typical of perimeter seals made on concrete (See description of mechanical tie-down in Section 4.4.4).

Water that would leak through defects in the primary geomembrane would flow in the primary draining geocomposite down to a longitudinal trench, and from there to transverse drainage trenches connected to the main drainage collector. The drainage system is divided into six separate compartments to allow for an approximate location of leaks: right part of the main dam, left part of the main dam, left slope between the two dams, upstream dam, right slope between the two dams, and bottom of reservoir.

(5) Construction

Construction proceeded as follows:

- The excavations and the embankments were prepared with the procedures established after the trial embankments. In particular, surface preparation was finalized with power shovels equipped with long booms and aided with GPS.
- Anchor trenches were prepared at crest.
- The secondary draining geocomposite was unrolled from the crest after having been temporarily ballasted in the top anchor trench. Adjacent rolls were placed so that the geotextile components overlap and there is no discontinuity in the geonet components.
- The secondary VLDPE spiked geomembrane, supplied in 6 m wide rolled sheets, was then unrolled from the crest and positioned on top of the secondary drainage layer (Figure 4).
- The installation of the primary draining geocomposite and of the primary waterproofing geomembrane was carried out with methods similar to those of the secondary layers. The placement of the primary geomembrane on the underlying spiked geonet was difficult due to the high friction at the interface of the two materials.
- A protection geotextile (Figure 5) was placed on the primary geomembrane liner, to avoid damaging it during placement of the cover. In order to prevent the development of tension along the protection geotextile in case of sliding of the cover layer due to dynamic stress, an expansion loop was left free at its top.
- To minimize outdoor exposure of the liner system materials, the cover layer (Figure 5) was placed immediately after placement of the various components of the liner system.

Figure 4. At left, soil subgrade, white secondary draining geocomposite, black VLDPE geomembrane, black primary draining geocomposite, and white PVC geomembrane. At right, the geomembrane being unrolled down the slope.

Figure 5. The cover layer placed on top of the geotextile at the right slope, and a general view of the site.

3.3 *Upstream exposed geomembranes*

3.3.1 *General discussion*

Exposed geomembranes account for approximately 30% of the geomembranes used at the upstream slope of embankment dams; and the geomembranes used at the upstream slope of embankment dams are approximately 90% of the geomembranes used in embankment dams.

Geomembranes exposed on the upstream face of embankment dams are subjected to a variety of potentially detrimental actions:

- Mechanical damage by ice, floating debris, rocks falling from the sides, animals, vandals, and traffic.
- Degradation by exposure to environmental agents (oxygen, UV, heat).
- Displacement by wind, wave action, fluctuations of water level, and gravity (causing creep).

Geomembranes can be used exposed if they have appropriate strength and composition to resist mechanical damage and degradation. Precautions must be taken to prevent or reduce geomembrane displacement by wind, waves and gravity. Generally, the main risk is displacement by wind. Therefore, geomembranes must be anchored against wind uplift.

As shown by Giroud et al. (1995), the tension generated by wind in the geomembrane is proportional to the square of wind velocity and to the distance between anchors. Therefore, if the wind velocity and the height of the dam are limited, it is possible to rely only on anchorage at the periphery of the upstream slope (including the crest). This case will be discussed in Section 3.3.2.

In other cases, exposed geomembranes are anchored with a face anchorage system in addition to the watertight peripheral anchorage that is present in all dams. A variety of face anchorage systems are discussed in the following sections:

- Anchorage by multiple trenches or beams (Section 3.3.3).
- Anchorage by strips (Section 3.3.4).
- Anchorage by strips and curbs (Section 3.3.5).

Anchorage by gluing, nailing and profiles will be discussed in Section 3.6 related to the rehabilitation of embankment dams with a rigid surface (such as concrete or bituminous concrete). However, they could conceivably be used in the case of new dams where the geomembrane is supported by a rigid drainage system made of porous concrete or bituminous concrete.

It is important to note that, with all the systems discussed in subsequent sections, the wind can uplift the geomembrane between the anchors. Assuming that the geomembrane and the anchors have been adequately selected and designed to withstand the wind-generated tension, it is important that the geomembrane return to its original position after the wind has ceased blowing. From this viewpoint, it is important to remember an incident that happened during construction at Figari Dam, a 35 m rockfill dam constructed in Corsica, France, in 1991, with a 2 mm PVC geomembrane on its 1V:1.7H upstream slope. Beneath the geomembrane, there was an independent geotextile. Two major problems occurred:

- The geomembrane crept down the slope during construction (by 4 m at the toe of the 70 m long slope).
- The geomembrane was uplifted by the wind with no damage, but the geotextile was displaced.

As a result, after the wind had ceased to blow, the geomembrane rested on the displaced geotextile. The geomembrane had to be removed to reposition the geotextile, and in the process 50% of the geomembrane had to be discarded. Two lessons can be learned:

- Non-reinforced PVC geomembranes should not be used on steep slopes.
- If geomembrane uplift by wind is likely to occur, the geotextile (if any) underlying the geomembrane should be: (1) bonded to the geomembrane; or (2) bonded to the underlying material (assuming this material is rigid); or (3) anchored at multiple locations to prevent its displacement by the wind.

At Codole Dam, a 28 m rockfill dam with a 1V:1.7H slope, built in 1983 in Corsica, France, with a composite geomembrane consisting of a 1.9 mm PVC geomembrane bonded to a needle-punched nonwoven geotextile, portions of the geomembrane were uplifted by wind during construction. The composite geomembrane was repositioned, albeit with some effort due to the high interface friction angle between the needle-punched nonwoven geotextile and the underlying porous bituminous concrete. No material was lost.

3.3.2 *Geomembrane anchored only at periphery*

Geomembranes have been used exposed, with anchorage only at the periphery, in several relatively small dams. This is possible, because, as mentioned In Section 3.3.1, the tension generated by wind in

the geomembrane is proportional to the distance between anchors.

The first such dam was Banegon Dam, a 17 m high embankment dam in France, mentioned in Section 2.3, where the geomembrane was a 4 mm thick bituminous geomembrane installed in 1973. At Banegon Dam, to the best of our knowledge, the geomembrane was not damaged by wind. It had other minor problems mentioned in Section 2.5.

If the water level in the reservoir is not expected to be drawn down, the geomembrane can be left uncovered below the water level.

Another example of a dam with an exposed geomembrane is Golden Camp Dam. It will be discussed in Section 3.6.

The upstream geomembrane can be left exposed at cofferdams considering their short service life. An example is Locone Cofferdam in Italy. It was constructed in 1982 and was incorporated in the dam in 1986. The geomembrane was a 1.5 mm butyl rubber geomembrane. The exposed geomembrane suffered some damage, but the cofferdam performed its function.

3.3.3 *Anchorage by multiple trenches or beams*

Anchorage of the geomembrane on an upstream slope can be done using horizontal or quasi-horizontal anchor trenches or anchor beams. With the crest anchor trench, these anchor trenches or anchor beams form a set of parallel or quasi-parallel anchors. The spacing between these anchors is calculated using the methodology published by Giroud and coworkers (Giroud et al. 1995; Zornberg & Giroud, 1997, Giroud et al. 1999; Giroud et al. 2006; Giroud 2009). Based on this methodology, the spacing between parallel anchors should be less in the upper part of a slope than in the lower part, because wind-generated suction is greater near the crest of a slope than near the toe.

An interesting application was made at Barlovento Reservoir in the Canary Islands, Spain. The Barlovento Reservoir is located in a volcano crater and it has a circular shape. The depth of the reservoir is 27 m: 20 m on the 1V:2.7H slopes and 7 m on the gently sloping bottom. The geomembrane installed in 1992 is a 1.5 mm PVC geomembrane reinforced with a polyester scrim on the slopes and unreinforced at the bottom. There are several levels of anchorage on the slopes. The top level is the horizontal crest anchor trench. The other anchorage levels are elegantly achieved by a unique beam spiraling down, on the slopes of the circular reservoir, from below the crest to the bottom. The resulting anchor spacing along the 58 m slope length is: 2 to 14 m for the upper spacing; 15 m for the next spacing downslope; 19 m for the next spacing downslope; and 10 to 22 m for the last spacing at the toe of the slope. The spiraling anchor beam contributes to drainage. The geomembrane has been subjected to wind velocities up to 160 km/h without damage.

3.3.4 *Anchorage using strips along the slope*

(1) Benefits of geomembrane deformability

The face anchorage discussed in this chapter is a recently developed system, which benefits from the high elongation capability of the geomembranes used. Both the impervious liner and its attachment system are made of geomembranes with high elongation capability and are thus capable of following differential movements that can take place in the dam during service.

(2) Description of a system of anchor strips

In a patented system, the waterproofing liner and its face anchorage system are made with the same material, a composite geomembrane incorporating the impervious element (a PVC geomembrane) and a multi-function layer (a needle-punched nonwoven geotextile, bonded to the geomembrane in the factory). The cross section is as follows, from the upper to the lower layer:

- Waterproofing liner: the composite geomembrane described above.
- Base/anchorage layer: This single layer forms the transition between the dam body and the geomembrane, incorporates the anchorage system for the waterproofing liner, and can also work as drainage layer.
- Drainage layer, as in other embankment dams but of reduced thickness.
- Dam body: zoning is not strictly required, and a single fill material can be used.

The face anchorage system is integrated in the dam in the form of strips of composite geomembrane, about 0.5 m wide, placed on the base/anchorage layer and anchored into it (Figure 6). Part of the strips is left protruding from the upstream face of the dam, so as to form continuous anchor strips running along the upstream slope. Spacing between anchor strips depends on service loads, the most severe one being generally that of wind.

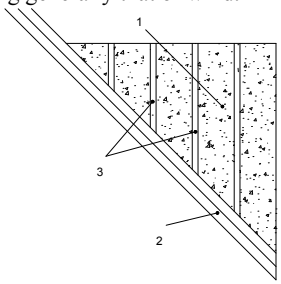


Figure 6. Face anchorage by geomembrane strips.
1. Base/anchorage layer 2. Plinth 3. Anchor strips

The base layer for the waterproofing geocomposite can be gravel stabilized with lean concrete and compacted with a vibratory plate mounted on the arm of an excavator, or it can be made by extruded concrete curbs. The extruded curbs will be discussed in Section 3.3.5.

The sheets of composite geomembrane are unrolled downslope over the base layer, and anchored to the protruding portion of the anchor strips by heat welding (Figure 7). Adjacent composite geomembrane sheets are joined by watertight vertical welds, creating a continuous watertight liner over the entire upstream face of the dam.

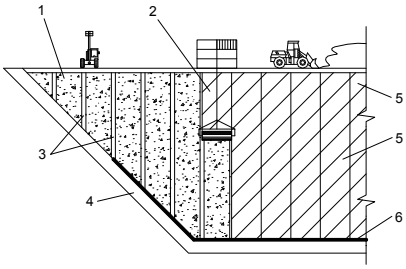


Figure 7. Placement and anchorage of geomembrane.

1. Base/anchorage layer 2. Composite geomembrane sheet being unrolled 3. Anchor strips 4. Concrete plinth 5. Composite geomembrane sheets welded to anchor strips
6. Watertight perimeter seal

At the periphery, the composite geomembrane is anchored by a watertight seal that on concrete is of the tie-down type described in Section 4.4.4, and on other types of subgrade is generally made by embedment in a trench. Perimeter seals are designed to be able to accommodate the differential movements and settlements typical at these locations.

Being prefabricated in the controlled environment of a factory under ISO procedures, all components of the waterproofing system (composite geomembrane, anchor strips, components of the perimeter seal) have pre-established and constant properties that are checked upon delivery of materials at the site and are not altered by weather conditions during installation; joints and perimeter seals are verified for watertightness with standard methods. Therefore, the characteristics and the quality of the waterproofing system are pre-established and constant throughout installation.

(3) Example of use of anchor strips

The described system had its first full field application at Kohrang Reservoir, Iran, in 2004 (Figure 8). The anchor strips were embedded in trenches running along the slopes, and in trenches running along the inclination of the reservoir bottom. The spacing of the anchor strips was calculated based on the uplift that would be caused by the wind load, using the methodology developed by Giroud and mentioned in Sections 3.3.1 and 3.3.2. The calculated spacing of the trenches was 8 m on the slopes and 16 m on the bottom.

The exposed geomembrane liner was placed directly over the compacted fill that forms the 14 m high dams and the bottom, and the anchorage against uplift was provided by welding the geomembrane sheets to the anchor strips.

Figure 8. Kohrang Reservoir. The anchor strips embedded in the trenches of the dams, and the geomembrane anchored by welding to the strips on dams and bottom.

The entirely flexible system (geomembrane + anchorage system) proved to be capable of resisting, and recovering from, deformations and differential movements between the deformable dam body and the concrete appurtenances without failing: settlements that had been foreseen during the design phase have occurred after impounding, with temperatures down to –37ºC, and heavy snow and ice formation.

As a consequence of settlements, some cracking has occurred in the concrete appurtenances, but with no damage to the exposed geomembrane. Additional advantages were related to times and costs: as reported by the owner (Yosefi et al. 2005), using this system "A saving of 60 percent was achieved in the volume of fill material, construction was faster and easier, the volume of impounded water compared with the original design was increased by 25 percent, which creates more energy generating capability, construction costs for the head pond could be decreased by 50 percent".

3.3.5 *Anchorage using extruded curbs*

(1) Principle of the facing using extruded curbs

The method described in this section consists of installing a geomembrane in conjunction with extruded concrete curbs. The method which consists in constructing a slope with extruded curbs is sometimes referred to as the "Itá method" because it was used at Itá Dam in Brazil (but with no geomembrane in that dam). The extruded curbs have a trapezoid

cross section (Figure 9). They are extruded in continuum; therefore, they can be used in any length depending on the project. The curbs interlock with each other to avoid that, while the embankment is being raised, they may be laterally displaced by the pressure of the fill and by the horizontal stresses locked-in as result of compaction. The extruded curbs allow constructing in a short time a slope that has the following characteristics: (1) it is fairly regular; (2) it has constant characteristics; (3) it can be steep; (4) it has surficial stability; (5) it is not erodible by rainwater; (6) if the curbs are made with porous concrete, it has drainage capability; and (7) it can be constructed rapidly.

The installation of the geomembrane in conjunction with the curbs is explained with the following case history. This case history is that of a tailings dam. However, the method could be used for dams retaining water. In the following case history, a PVC geomembrane is used, but the curb facing method has also been used with HDPE geomembrane at Antamina Tailings Dam in Peru.

(2) Example of geomembrane installation with curb facing

An example of use of curb facing was in 2008 at Sar Cheshmeh, Iran, for the heightening of an existing 75 m high tailings dam. The existing dam, designed and constructed in the late 1970s, has an impervious clay core. The upgrading project includes strengthening and raising the main embankment dam and a saddle dam in four stages, for a total of 40 m of heightening.

For the heightening, a rockfill dam with an upstream exposed geomembrane was adopted instead of a clay core, on the basis of it being a more stable, efficient and buildable arrangement, and preferable from a construction, performance and cost standpoint.

The face of the new rockfill dam is formed by extruded porous concrete curbs, a technique used for concrete faced rockfill dams and described in item (1) above. In the case a geomembrane is used in conjunction with extruded curbs, the geomembrane is anchored to all or some of the curbs. The system described herein consists of using "anchor wings" which are connected to form continuous "anchor strips" that run along the upstream slope. The anchor wings shown in Figure 9 are 1.6 m × 0.5 m rectangular pieces of geomembrane. A given anchor wing covers the sloping side, the top side and the vertical side of the curb with extra horizontal length beyond the vertical side for anchorage inside the dam body and extra length below the sloping side for welding to the lower anchor wing (Figure 9). Placement of the anchor wings is carried out concurrent with construction of the curbs. The welding of all anchor wings along the same slope line forms a continuous anchor strip that runs along the dam upstream slope. An anchor strip can be seen on the left of Figure 10.

The spacing of the anchor strips was calculated based on the uplift that would be caused by the wind load, using the methodology developed by Giroud and mentioned in Sections 3.3.1 and 3.3.2. The calculated spacing of the strips was 6 m.

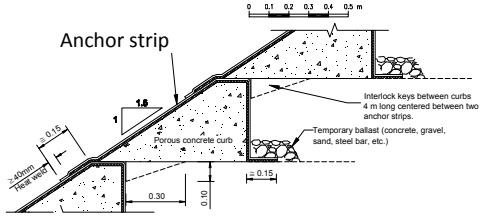


Figure 9. Extruded concrete curbs with one geomembrane anchor wing per curb.

Since the waterproofing liner will be later covered by tailings, a robust geomembrane was used for the liner and the anchor strips: a composite geomembrane consisting of a 3 mm thick PVC geomembrane laminated during fabrication to a 500 g/m^2 polyester geotextile.

Installation of the waterproofing system was staged to follow the construction of the dam, so that the waterproofing system could be completed practically concurrent with completion of the civil works. The geomembrane sheets, temporarily secured at the crest, were deployed over the curbs and then fastened to the anchor strips by heat-welding (Figure 10). The high friction at the interface geotextile/curbs facilitated placement of the composite geomembrane and increased stability with respect to sliding.

Figure 10. Sar Cheshmeh, Iran 2008. Placement of composite geomembrane sheets and welding to the anchor strips.

At the abutments, the composite geomembrane was attached to the concrete plinth by a watertight seal of the tie-down type (see description in Section 4.4.4).

The watertight connection of the geomembrane of Stage II B to the clay core of the existing dam was made by a 10 m wide compacted clay zone in the upper part of the existing core. This required the excavation of 60,000 m^3 of material from the down-

stream half of the existing embankment crest, the majority of which was reconditioned and reused in the core transition construction. The geomembrane was placed over the clay, and covered with compacted clay.

Installation of the waterproofing system of Stage II B took seven weeks for a height of 10 m. The top anchorage of Stage II B, at elevation 2185 m, was made with a flat stainless steel batten strip. Construction of Stage II C (Figure 11) that followed Sage II B took six weeks for a height of 9 m between elevations 2185 m and 2194 m. The waterproofing system of Stage II C is identical to the one of Stage II B. The lower end of the geomembrane of Stage II C overlaps the top anchorage of the geomembrane of Stage II B and is watertight connected to it by welding; the junction was further waterproofed with a PVC cover strip.

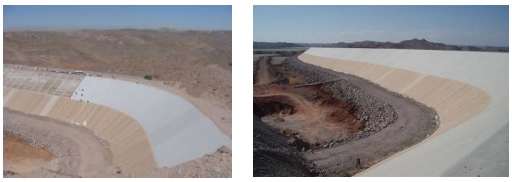


Figure 11. Sar Cheshmeh, Iran 2008. Placement of the geomembrane of Stage II C, and Stage II C completed.

The total length of Stage II C was 1000 m and the total installed geomembrane was 20,500 m^2 for Stage II B and 18,000 m^2 for Stage II C. The 2008 raisings will be followed in 2010 by more raisings for a total height of raisings of 40 m.

As reported by the designer of the dam (Noske 2009) "From the perspective of both the designer and the owner, the selected GSS *[liner system]* resulted in an efficient and economic waterproofing solution for the tailings storage raise. The extruded curbs have proven to be an effective construction method, whilst the anchor strip installation was a simple, routine process. Combined, these features greatly facilitated the installation of the PVC geocomposite. The installation was fast, with each stage installed within a four week mobilization, commencing immediately upon completion of the earthworks. The overall construction period was also significantly reduced, given that the earthworks contractor was able to produce and place rockfill and concrete curbs at a greater rate than would have been achieved if the design had specified a moisture conditioned and compacted clay core *[. . .]* the geocomposite faced rockfill approach is a viable, effective means of tailings dam construction where water retention in the vicinity of the embankment is necessary, and where natural materials are either not available, or are unable to be used from a technical perspective."

3.4 *Upstream covered geomembranes*

3.4.1 *General discussion*

If a geomembrane is left exposed on the face of an embankment dam, it is subjected to a number of actions that could damage it. The reasons for geomembrane protection by a cover layer are:

- Protection of the geomembrane against mechanical damage (by ice, floating debris, rock falling from the sides, animals, vandals, traffic).
- Elimination of exposure to environmental agents (oxygen, UV, heat) that could cause degradation of the geomembrane.
- Prevention of geomembrane displacement by wind, wave action, gravity (causing creep).

Protection of geomembranes on slopes is typically ensured by covering the geomembrane with a layer of heavy material such as concrete or soil. Considering all of the potentially detrimental actions, a majority (70%) of the geomembranes used at the upstream slopes of embankment dams are covered based on the ICOLD database.

Several systems have been used, or could be used, to cover geomembranes:

- Interlocking concrete blocks (see Section 3.4.2).
- Articulated concrete blocks (see Section 3.4.2).
- Concrete slabs (see Section 3.4.3).
- Shotcrete on geotextile (see Section 3.4.4).
- Geocells or geomattresses filled with concrete (see Section 3.4.5).
- Soil and rock protection (see Section 3.4.6).

It is important to note that improperly designed or constructed cover layers can damage a geomembrane during construction or operation. Therefore a thick needle-punched nonwoven geotextile is generally used between the geomembrane and the cover material.

The placement of the cover layer is possibly the most critical part of construction of a covered geomembrane system. Construction quality assurance activities should not stop after placement of the geomembrane. It should continue during the placement of a geotextile protection layer on the geomembrane and, then, during placement of the cover layer.

The important role of the geotextile during operation is illustrated by the fact that the geomembrane was not damaged in spite of extensive displacement of concrete blocks at L'Ospedale Dam (see Section 3.4.2). It is also illustrated by the fact that the geomembrane was punctured by the edges of its "protecting" concrete slabs at Bitburg Dam where no geotextile was used (see Section 2.5).

3.4.2 *Example of cover made by concrete blocks*

(1) Interlocking concrete blocks

At L'Ospedale Dam, a 26 m high, 135 m long, rockfill dam built in 1978 in Corsica, France, the 4.8 mm bituminous geomembrane is covered by interlocking blocks placed by hand. Such blocks are typically used for driveway pavement (hence the name "paver blocks"). The blocks are 0.08 m thick and approximately 0.20 m × 0.12 m (they are not rectangular). These blocks were placed on the needle-punched nonwoven geotextile overlying the bituminous geomembrane on the 1V:1.7H slope. The geotextile is independent of both the geomembrane and the blocks. Its functions are: (1) to protect the geomembrane during placement of the blocks and during operation in case of movement of the blocks; and (2) to allow free movement of the blocks with respect to the geomembrane. The stability of the blocks against gravity was ensured by the fact that the blocks were buttressed at the toe against the concrete plinth.

The blocks were too light to withstand wave action; also, as understood later (and discussed below) some of the blocks were not properly interlocked due to misalignment at installation and thermal expansion-contraction. Four years after installation, when the reservoir was almost full, a storm removed a few thousand blocks at or near the water surface. The geotextile was displaced in a part of the area where the blocks had been removed, and was in place in the rest of the area. The geomembrane was not damaged, which indicates that the geotextile performed its function even in areas where it had been displaced.

Repair was done by replacing the missing blocks by identical blocks. A small amount of concrete was used to secure the blocks that appeared to be loose or not properly interlocked with adjacent blocks. Protection using concrete blocks may not be a bad solution if the blocks are sufficiently heavy and if they are properly interlocked.

From the experience gained at L'Ospedale Dam it appears that it is difficult to align and tightly interlock paver blocks over a large area on a steep slope. Also dilatation-contraction phenomena during service disturb the interlocking between blocks all the more so that the dam face is large. Therefore, the paver block solution may be best for small dams.

(2) Articulated concrete blocks

Articulated concrete blocks anchored from the crest anchor trench are an interesting solution in dams where the geomembrane needs to be covered only in the upper part of the slope. It should be noted that this is also the case of geomembrane cover by geocells (see Section 3.4.5).

Two examples of articulated concrete blocks are:

- Mas d'Armand Dam, France (1981), a rockfill dam, 21 m, 1V:1.6H, where the 4.8 mm bituminous geomembrane is covered with 0.08 m thick concrete blocks are glued on, and supported by, a geotextile anchored at the crest; and
- Mauriac Dam, France (1989), a rockfill dam, 14.5 m, 1V:1.7H, where the 3.9 mm bituminous geomembrane is covered with 0.12 m thick concrete blocks connected with steel cables anchored at the crest.

3.4.3 *Examples of cover made by concrete slabs*

(1) General information

Cast-in-place concrete slabs are the most typical cover for geomembranes used in embankment dams. Joints are needed between slabs to release water pressure in case of reservoir drawdown. Joints also provide flexibility to the concrete slab system, which makes it possible to withstand settlement without cracking.

Steel reinforcement of the concrete slabs is not recommended as it may damage the geomembrane during construction. It is recommended to use concrete slabs with no reinforcement or with polypropylene fiber reinforcement.

A list of typical embankment dams with geomembrane covered by cast-in-place concrete slabs is below:

- Contrada Sabetta (1959), unreinforced 2 m × 2 m concrete slabs, 0.20 m thick. The joints between adjacent slabs were left open, 1 mm wide, and were not filled by any porous material; there was a sheet of bituminous paper-felt between the concrete slabs and the geomembrane.
- Codole (1983), Corsica, France, 28 m, 1V:1.7H, concrete slabs, 4 m wide and continuous along the slope, 0.14 m thick, reinforced with steel bars; joints along slope, 20 mm wide filled with polystyrene foam; independent nonwoven geotextile between the geomembrane and the concrete.
- Jibiya (1989), Nigeria, 23.5 m, 1V:3.0H, concrete slabs, 2m × 4m, 0.08 m thick unreinforced; 300 g/m^2 needle-punched nonwoven geotextile between the geomembrane and the concrete.
- Figari (1993), Corsica, France, 35 m, 1V:1.7H, concrete slabs 0.14 m thick, reinforced PP fibers; independent nonwoven geotextile between the geomembrane and the concrete.
- Bovilla (1996), Albania, 91 m, 1V:1.6H, concrete slabs, 6 m along slope and 3 m horizontal, 0.20 m thick (in upper part) 0.30 m thick (in lower part); joints filled with PP needle-punched nonwoven geotextile; 800 g/m^2 PP needle-

punched nonwoven geotextile between the geomembrane and concrete.

- Ortolo (2000), Corsica, France, 37 m, 1V:1.7H, concrete slabs 0.14 m thick, reinforced with PP fibers.
- La Galaube (2000), France, 43 m, 1V:2.0H, concrete slabs 0.10 m thick, reinforced with PP fibers; 400 g/m^2 PP needle-punched nonwoven geotextile between the geomembrane and concrete.

It appears that a typical cover would be with a 0.15 m thick slab made of concrete either unreinforced or reinforced with PP fibers. A greater thickness would be justified in special cases, as in the upper part of the slope at Bovilla dam, because of the risk of rocks falling from the sides, or, as in the lower part of the slope at Bovilla dam, because this lower part acts as a buttress to ensure the stability of the upper part.

(2) Example of concrete slab at Codole Dam

At Codole Dam, a cost analysis at the design stage showed that the solution adopted was cost-effective compared to other solutions even if the geomembrane and the overlying concrete slabs had to be replaced after 25 years of service. Codole Dam was built 27 years ago and is still in service.

At Codole Dam, the concrete slabs were reinforced with traditional steel bars. This proved to be a potential problem. Great precautions had to be taken during construction to avoid damaging the geomembrane with the reinforcing bars, in spite of the presence of a geotextile above the geomembrane.

(3) Example of concrete slab at Jibiya Dam

Jibiya Dam, Nigeria, 1989, is a 23.5 m high dam, lined with a 2.1 mm PVC geomembrane reinforced with a 50 g/m^2 glass fiber nonwoven fleece and bonded to a 400 g/m^2 polyester staple fiber needle-punched nonwoven geotextile. The geomembrane is covered by 2 m × 4m, 0.08 m thick, concrete slabs with a 300 g/m^2 needle-punched nonwoven geotextile between the geomembrane and the concrete.

An interesting feature of Jibiya dam is the horizontal placement of the geomembrane (rather than placement by unrolling along the slope). This made it possible to place the geomembrane and construct the concrete protection almost simultaneously. Horizontal placement of the geomembrane was possible thanks to the gentle slope: 1V:3.0H.

(4) Example of concrete slab at Bovilla Dam

Bovilla Dam, Albania, is a 91 m high rockfill dam for water supply, flood mitigation and hydropower (Figure 12).

The original design of a concrete face rockfill dam was changed to a geomembrane face rockfill dam for the following reasons: (1) concerns about the final quality of the reinforced concrete face and its potential for future cracking; and (2) need to reduce construction time and costs as the project was behind schedule (Sembenelli et al. 1998).

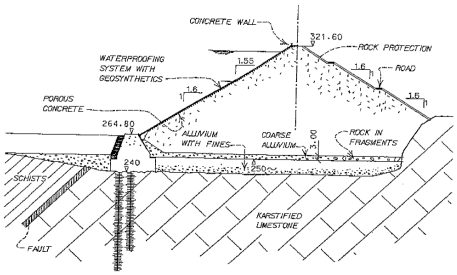
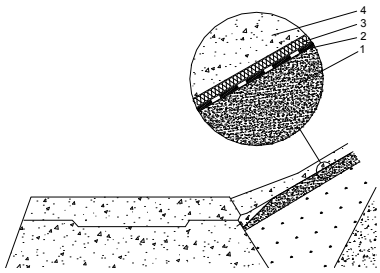


Figure 12. Main cross section at Bovilla Dam.

The upstream composite geomembrane installed in 1996 is the only element providing watertightness to the dam. It consists of a 3 mm thick PVC geomembrane laminated to a 700 g/m^2 polyester geotextile. It was placed directly over the transition layer, gravel stabilized with cement slurry. The geomembrane covers the entire upstream slope, from the crest to the massive toe block, i.e. over a difference in elevation of 54 m. The upstream slope is 1V:1.55H in the upper 40% and 1V:1.6H in the lower 60%.

The geomembrane was covered with unreinforced concrete slabs that were placed on an 800 g/m^2 geotextile (Figure 13). The slabs are 6 m long in the slope direction and 3 m horizontally. The slabs are 0.20 m thick except near the bottom where they are 0.3 m thick as they serve of buttress to the slabs. The geotextile had a double function: providing anti-puncture protection to the geomembrane against casting of the slabs, and act as a light reinforcement for the slabs themselves.



1. Gravel stabilized with cement slurry 2. Geomembrane 3. Anti-puncture PP geotextile, 800 g/m^2 4. Unreinforced concrete slab

Figure 13. Bovilla Dam facing system.

The decision to adopt cast in place concrete slabs rather than prefabricated concrete blocks was taken because casting slabs was considered less aggressive on the PVC geomembrane than the placement of prefabricated concrete elements, and also considering the problems with concrete blocks at L'Ospedale Dam (see Section 3.4.2 (1)).

At Bovilla Dam, the bottom seal of the geomembrane on the toe block was designed to be able to accept differential movements and settlements one order of magnitude larger than the estimated ones (Figure 14). Extra material and protection/decoupling layers were placed for this purpose over the geomembrane at the seal.

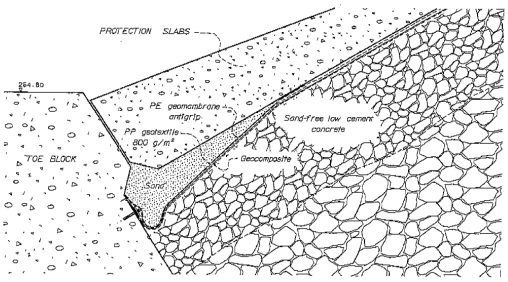


Figure 14. Bottom seal at Bovilla Dam.

The bottom seal, of the tie-down type (see the definition in Section 4.4.4), was made by stainless steel batten strips that compress the geomembrane on the concrete of the toe block. To achieve even compression, the concrete surface was leveled out with watertight resin; and rubber gaskets and splice plates were used for stress distribution. The watertightness of the seal is verified by checking that all anchor bolts that compress the batten strips are tightened to a specified torque.

(5) Example of concrete slab at La Galaube Dam

La Galaube Dam, 43 m high, a rockfill dam in France, is the highest dam waterproofed with a bituminous geomembrane (Gautier 2002). The bituminous geomembrane, 5 mm thick, has been laid on a 0.10 m thick cold asphalt mix placed over a layer of non-bounded gravel impregnated with bitumen on the 1V:2.0H slope. The cover layer is a 0.10 m thick concrete slab reinforced with polypropylene fibers (Figure 15).

The bottom anchorage (Figure 16) is made on the concrete plinth and is of the tie-down type (see Section 4.4.4). Installation of 22,000 m^2 of bituminous geomembrane was completed in 2000.

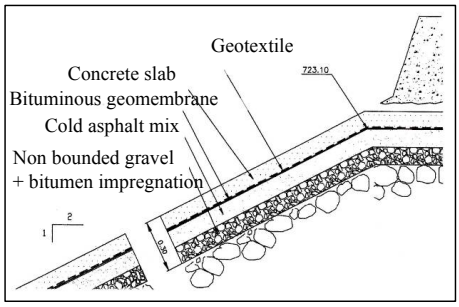


Figure 15. Typical cross section at La Galaube Dam (after Gautier, 2003).

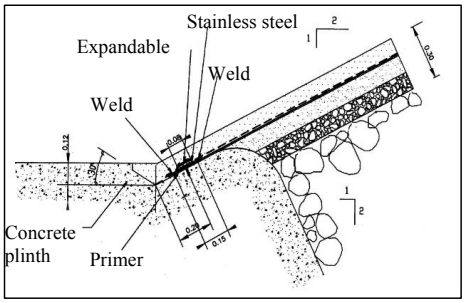


Figure 16. Typical cross section at La Galaube Dam (after Gautier, 2003).

3.4.4 *Examples of cover made by shotcrete*

When only a light cover is needed, a simple and effective low cost configuration consists of 50 to 70 mm of shotcrete reinforced with a PP geogrid. A needle-punched nonwoven geotextile protection is needed between the geomembrane and the shotcrete. This cover system has sufficient flexibility to withstand the settlements of the fill. The shotcrete may crack, but without significant alteration of its ballasting/protective function. Thanks to the needle-punched nonwoven geotextile, the system is free draining at the back, thereby relieving backpressure.

This type of cover has been used adopted at Mucone Dam in Italy, 1986, 7 m high, 880 m long. The 1.5 mm PVC geomembrane is covered by 0.06 m of shotcrete reinforced with a PP geogrid on a 500 g/m^2 PP needle-punched nonwoven geotextile between the geomembrane and the shotcrete (Figure 17).

Figure 17. Mucone Dam. Cover by shotcrete reinforced with a PP geogrid, placed over a geotextile

If the part of the upstream face being covered is such that the total weight of the cover does not generate excessive tension in the geosynthetics, the entire protection is anchored at crest. Otherwise it can be supported by an H steel beam placed horizontally at any given elevation, and anchored to the crest by steel cables.

3.4.5 *Examples of cover made by geocells*

At Mud Lake Dam, Nevada, USA, a 17 m high, 365 m long, dam built in 1900, the 1.14 mm reinforced PP geomembrane installed in 2000 on a 1V:2.0H slope is covered with a geocell filled with concrete. The construction sequence was as follows:

- A 540 g/m^2 polypropylene needle-punched nonwoven geotextile was placed on the polypropylene geomembrane and temporarily secured in the crest anchor trench.
- A 75 mm deep geocell was deployed on the slope.
- Tendons made of polyester or polyaramide ("Kevlar") passing through holes in the walls of the geocells, and attached to the geocell at all levels along the slope, were used to anchor the geocell at the crest of the dam. The tendons are attached to a bar embedded in the anchor trench.
- The geocell was filled with concrete.

Geomattresses (i.e. geotextile forms filled with grout or mortar) can also be used.

3.4.6 *Examples of cover made by using soil layer*

(1) General discussion

Geomembrane covers using a soil layer cannot be used on the steep slopes typical of rockfill dams because they would not be stable. Soil layers should only be used on slopes less steep than 1V:2.0H, preferably on 1V: 2.5H slopes and less steep. It is essential to check the stability of the soil cover under rapid drawdown conditions, the worst conditions for static stability. In relevant areas, the seismic stability should also be checked.

At Worster Dam, Colorado, USA, 22 m high, 215 m long, lined with a 1.5 mm textured HDPE geomembrane, the 0.3 m thick soil cover bulged at the toe of the 1V:3.0H slope at the first drawdown of the reservoir, as it was uplifted by water entrapped between the geomembrane and an old concrete slab located a few meters behind the geomembrane.

An example of soil cover was already presented in Section 3.2.5: Les Arcs Dams and Reservoir, with a 1V:3.0H slope. Additional examples are presented below.

(2) Example of soil cover: Aubrac Dam

Aubrac dam, France (1986), 15 m, 1V:2.5H has the following cross section from the upper layer to the lower layer:

- 0.5 m rockfill 100-300 mm
- 0.2 m gravel 0-25 mm
- needle-punched nonwoven geotextile (500 g/m^2)
- 1.2 mm PVC geomembrane
- needle-punched nonwoven geotextile (500 g/m^2)
- 0.2 m drainage layer, gravel 0-25 mm

During construction, a slide of the soil cover occurred over 1000 m^2 at the interface between the PVC geomembrane and the underlying geotextile.

This cover slide on a 22° slope was unexpected because the following values of the interface friction angle had been measured: 34° using a shear box (high normal stress) and 28° using inclined plane (low normal stress), which is more appropriate considering the depth of material on top of the interface. However, further investigations have shown that the interface friction angle could be reduced by 3° by a moist interface and 3° by vibrations (like vibrations potentially caused by construction equipment).

This case history illustrates that the measurement of interface shear strength is extremely delicate.

3.5 *Dams with internal geomembranes*

3.5.1 *General discussion*

Internal geomembranes (inclined, vertical, or quasi-vertical in "accordion" or "zigzag" shape) have the following advantages:

- They can be associated with a layer of low-permeability soil (compacted clay or silt), thereby forming a composite liner, which reduces the seepage in case of defect in the geomembrane.
- The geomembrane is well protected (assuming that the materials adjacent to the geomembrane have been placed carefully, without damaging the geomembrane, and assuming that these materials do not contain elements likely to damage the geomembrane in service).
- Uplift of the geomembrane by wind is obviously prevented.
- Uplift of the geomembrane by water in case of rapid drawdown of the reservoir is prevented if the soil pressure on the geomembrane is sufficient, which is always the case with vertical or quasi-vertical geomembranes and should be checked in the case of inclined geomembranes.
- The costs involved are smaller because the area of geomembrane is smaller.
- If the geomembrane has a zigzag shape, is it is less sensitive to settlements, unless it is connected to a rigid structure.

On the other hand, internal geomembranes have drawbacks: (1) internal geomembranes are generally not justified when the reservoir is lined (which is not frequent); (2) they are generally difficult to con-

struct; and (3) they require coordination between earthworks and geomembrane installation. For these reasons, internal geomembranes are rarely used. Internal geomembranes account for only 10% of the total of geomembranes used in embankment dams. Examples of embankment dams with internal geomembranes are presented in the following sections.

3.5.2 *Dams with inclined internal geomembrane*

As indicated in Section 2.3, the first embankment dam with an internal geomembrane was Odiel dam in Spain constructed in 1970 with a CPE geomembrane.

An inclined internal geomembrane was used at Valence d'Albi, France, 16 m high, constructed in 1988. The geomembrane is a 4 mm thick bituminous geomembrane. It was installed on a 1V:2.0H slope and was covered with a zone of soil having a final slope of 1V:2.5 h in the top third and 1V:3.0H in the lower two thirds (Girard et al. 1990; Alonso et al. 1990; Giroud 1991). The geomembrane is in contact with the compacted soil (a medium-permeability schist); as a result, the seepage rate in case of a defect in the geomembrane would be significantly less than it would be if the geomembrane had been in contact with a drainage material.

The bituminous geomembrane used at Valence d'Albi Dam (and at other dams) comprises on one of its faces (normally placed in contact with the supporting soil) a high-puncture resistance thin polyester film intended to prevent the growth of vegetation when the bituminous geomembrane is exposed. This film is rather smooth. During construction, this film was burnt with a flame to increase the interface friction angle with the adjacent material.

3.5.3 *Examples of dams with central geomembrane*

(1) Examples of central geomembranes in China

According to the ICOLD database, among the 43 embankment dams with a geomembrane constructed in China, six have the geomembrane in an internal position. Information available on these dams is limited.

In all of these dams, the geomembrane used was a PVC geomembrane. In two of the six dams, the geomembrane was used to repair an existing dam. For one of them, there is no detailed information. The other dam, Zhuwei Dam will be discussed in Section 3.6.

One of the four remaining dams is a 10 m high cofferdam built in 1995 with a 1.5 mm thick PVC geomembrane in central position. The three remaining dams are:

- Heihe Dam (1999), 18.6 m high, 220 m long, 0.3 mm PVC geomembrane in a central position, with no more details available.
- Wangfuzhu Dam (1999), 13 m high, 1251 m long, 0.5 mm PVC geomembrane in a central position with a zigzag configuration, and a 200 g/m^2 geotextile, with no more details available.
- Shirensigou Dam (2002), 41 m high, 94 m long, 0.8 mm PVC geomembrane in a central position, with a 300 g/m^2 geotextile, and no more details available.

It appears that very thin geomembranes are used in internal position in permanent embankment dams in China.

(2) Gibe III Cofferdam, Ethiopia

At Gibe III Cofferdam, in contrast with several dams in China, the central geomembrane has a thickness similar to the thickness of upstream geomembranes. Gibe III Cofferdam is a 50 m high rockfill cofferdam for the main 240 m high RCC dam of Gibe III Hydroelectric Project. The geomembrane is in a central position with an accordion shape (Figure 18).

According to the designer (Pietrangeli et al. 2009) "a central geomembrane core was adopted at Gibe III because of:

- Timing: it would allow completing the construction within the very short construction period.
- Simplicity: it would allow the realization of an embankment of homogeneous rockfill, with optimization in construction times and costs.
- No clay: lack of availability in the zone of material suitable for an impervious core.
- Safety: with such layout the impervious layer is embedded in the embankment, safer than any impervious layer (rockfill dams with concrete or bituminous concrete face).
- Settlement is not a problem.
- Permeability tests done during construction provide a guarantee unusual for this kind of structure."

The impervious core consists of a 3.5 mm thick PVC geomembrane "sandwiched" between two thick 1200 g/m^2 needle-punched nonwoven geotextiles acting as anti-puncture layers that protect the geomembrane against possible damage by the construction materials and equipment (Figure 19). Furthermore, two 50 mm thick sand filter layers are placed respectively at the upstream and downstream side of the geotextile-geomembrane-geotextile "sandwich".

The geomembrane has been installed from the bottom cut-off up to the crest, in a zigzag pattern so as to follow the step by step construction of the embankment and to be more flexible against possible settlements of the embankment. The waterproofing system thus creates a continuous impervious barrier running all along the longitudinal axis of the dam from the bottom cut-off up to the crest. The first constructed section of the cofferdam body was on

the downstream side with a height of 6 m. The next sections follow one upstream and one downstream and have each a constant height of 12 m.

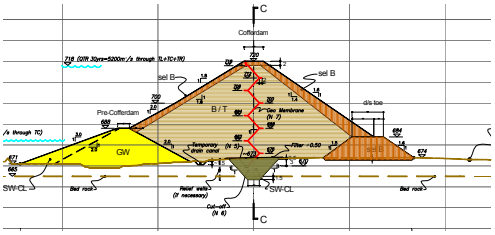


Figure 18. Typical cross section of Gibe III Cofferdam.

In the flat area at the crest of each section, the geomembrane lining the lower section is overlapped by the geomembrane lining the section above it, for a width of about 2 m, where the two geomembranes are connected by a double track automatic seam. Quality control of the seams was done with air pressure test using the air channel between the two tracks of the seam.

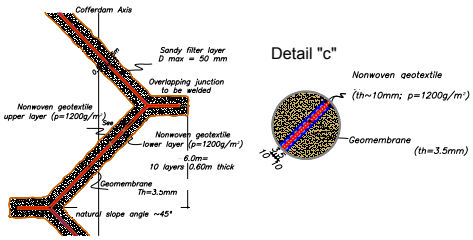


Figure 19. Detail of the impervious core of Gibe III Cofferdam.

The execution of this horizontal longitudinal connection seam, parallel to the axis of the dam, is made at the same time of installation of the PVC geomembrane sheets over the inclined slope of the upper section (Figure 20). Before the execution of this seam the protection placed to avoid damages on the geomembrane is removed, the area is cleaned, the integrity of the geomembrane is checked and, if needed, damages are repaired.

Figure 20. Gibe III Cofferdam. Placement of the geomembrane on the first section of the fill (left) and welding the geomembrane of the bottom section to the geomembrane of the upper section.

The alternate upstream-downstream phased embankment construction, with geomembrane seaming at the end of each phase is consistent with the strategy described by Giroud (1990, 1991). It has been used for the heightening of Middle Creek Dam in 1992 with a CSPE geomembrane.

The anchorage at the bottom boundaries is made by embedding the geomembrane in the 6 to 8 m deep clay cut-off and by backfilling with the same clay (Figure 21). At the two abutments, due to the difficulty of excavating the cut-off with the same depth because of the presence of surfacing rocks in the river bed, the geometry was slightly modified during construction by adapting the thickness of the clay layer below and above the geomembrane.

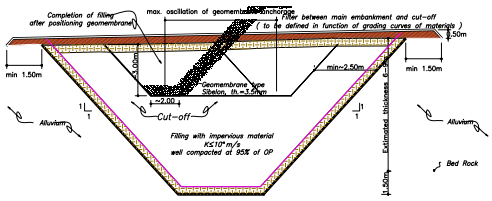


Figure 21. Gibe III Cofferdam. Anchorage of the impervious geomembrane core in the clay cut-off trench.

Top anchorage of the geomembrane is made with steel anchor and plates fixed to the reinforced concrete crest wall. Final stages of construction are shown in Figure 22.

Figure 22. Gibe III Cofferdam. Placement and compaction of the fill for the fourth Section of the cofferdam, and the 50 m high cofferdam nearing completion.

(3) Signal Buttes Flood Retention Structure

Signal Buttes Flood Retention Structure is a 14 m high, 2,100 m long, dam near Phoenix, Arizona USA, constructed in 1988. This dam controls floods that are relatively rare but could threaten populated areas. Due to the extremely dry climate, the reservoir of this dam is generally empty. The geomembrane central position was selected to ensure that the geomembrane would always be protected by sufficient soil even in case of intensive erosion. The geomembrane selected was an HDPE geomembrane, in part because HDPE was becoming a dominant geomembrane material in the early 1980s in the

USA when Signal Buttes dam was designed and in part because it was reportedly believed by the design engineer that HDPE geomembranes have high puncture resistance and could deform to follow potential large deformations of the embankment and its foundation. The 2.5 mm thickness was selected to increase the puncture resistance and resistance to stresses due to construction.

The geomembrane was installed with an accordion shape. However, contrary to the case of Gibe III Cofferdam discussed above, no horizontal seams were planned. According to the design and the construction plan, the HDPE geomembrane was to be installed as a continuous sheet from the bottom of the cutoff trench to the crest of the dam. At each stage of the alternate downstream-upstream embankment construction (similar to that of Gibe III Cofferdam), the extra length of geomembrane had to be stored at the crest of the constructed embankment section, waiting for the next embankment section to be constructed on the other side. The geomembrane was thus exposed to construction activities and subjected to temperature variations causing wrinkles. Handling the enormous weight of the geomembrane was a challenge for the contractor. Furthermore the dam was not straight but had a U shape. This further complicated the handling of the geomembrane. Also, connecting the accordion-shaped geomembrane to a vertical concrete structure was extremely difficult.

In spite of these problems, the construction of Signal Buttes Flood Retention Structure was completed. According to local engineers knowledgeable in flood retention structures, Signal Buttes Flood Retention Structure Signal Buttes Flood Retention Structure is now considered to be in better condition than conventional homogeneous flood retention structures in the same desert area, which are generally deeply cracked and could be unsafe in case of major flood. Using a geomembrane is clearly the best solution for earth dikes that are susceptible to cracking because they are not exposed to water most of the time and because of ground subsidence.

Construction of the Signal Buttes Dam would have been much easier if geomembrane panels had been welded at each embankment construction stage, rather than trying to install a geomembrane continuous from bottom to top. Also, a more flexible geomembrane would have made construction much easier. The difficulty of installing a geomembrane with an accordion shape without seaming at the end of each embankment construction stage was mentioned by Giroud (1990, 1991).

3.6 *Rehabilitation of embankment dams*

Geomembranes have been used for the rehabilitation or repair of all types of embankment dams: earth dams; concrete face rockfill dams and bituminous concrete face rockfill dams.

3.6.1 *Rehabilitation of earth dams*

(1) A typical example, Gold Camp Dam

Gold Camp Dam is a 265 m long earthfill dam with a total height of 30 m (but a face height of 20 m), built in 1954, in Colorado, USA, at elevation 1985 m. Three years after construction, a major leak emptied the reservoir. Grouting reduced the leakage rate, but for safety reasons, the dam was operated with reservoir at low level. In 1983, the dam was rehabilitated using a 1.5 mm CSPE geomembrane placed on the entire 1V:3.5H upstream slope and anchored at the crest and the periphery in an anchor trench.

The geomembrane has been left exposed in spite of the large upstream face area. Air vents in the upper part of the geomembrane contribute to the stabilization of the geomembrane in the case on wind. It should be noted that the wind-generated suction is less effective at high elevation.

The solution which consists in lining the upstream face with a geomembrane was preferred to the following solutions: (1) Adding a downstream zone of compacted soil and a toe drain; (2) Installing pressure relief wells from the crest; and (3) Adding a clay liner on the upstream slope. The rehabilitation has been successful and the seepage has been practically eliminated.

(2) Rehabilitation of flood retention structures

In western states of the USA, there are kilometers of flood retention structures that need to be repaired. They are relatively low dams, typically 10 m high, constructed in desert areas to control floods, which are rare but very dangerous, and increasingly dangerous with the development of populated areas. These dams are earth dams constructed in the 1950s and 1960s, and they are severely cracked because of the following reasons: (1) the extremely dry climate; (2) the fact that the reservoir of these dams is generally empty; and (3) the differential settlements due to subsidence caused by excessive pumping of ground water.

Cracks cannot be fixed. Therefore, using a geomembrane on the upstream slope is an obvious solution for the rehabilitation of cracked embankment dams. Studies undertaken for the selection of geomembrane to be used on the upstream slope of a cracked embankment dams have concluded that the geomembrane should be able to withstand the stresses caused by:

- The opening of a crack in the soil supporting the geomembrane after geomembrane installation; and
- Water pressure on the geomembrane bridging open cracks in case of sudden filling of the reservoir by a flood, a situation which is known to happen periodically.

3.6.2 *Rehabilitation of concrete or bituminous concrete face rockfill dams*

In the rehabilitation of concrete face rockfill dam dams and bituminous concrete face rockfill dams, the concept is similar to what will be discussed for concrete dams. Both are hard surfaces to which geomembranes can be attached. However, the anchorage system of the geomembrane to the dam face is designed depending on the type and strength of the existing facing (concrete or bituminous concrete). Therefore, the geomembrane is typically left exposed, and maintained to the dam face by face anchorage and perimeter anchorage.

Face anchorage can be made by gluing or by mechanical fixations.

Gluing has been done in the case of in situ geomembranes. In a number of these applications, gluing has resulted in failure, such as at Paradela Dam, Arcizans Dam, and Rouchain Dam. These failures can be attributed to the nature of the in-situ geomembranes. But, they may also be due to a fundamental conceptual mistake. As indicated in Section 3.2.3, two liners should not be located directly on top of each other, unless there is a sufficient load on them to counteract water pressure. Of course, there is no load on a geomembrane glued on a rigid support. Therefore, Consistent with the recommendations made in Section 3. 2.3, the ICOLD Bulletin 135 recommends that gluing "should not be continuous over the entire face to allow drainage behind the geomembrane and release of vapor pressure which would result in uplift pressure which could detach and ultimately damage the geomembrane or the supporting layer. *[. . .]* Rehabilitation of concrete or bituminous concrete facings with a geomembrane glued on the entire surface is not recommended." In fact, gluing has been abandoned since the 1980s.

A simple mechanical fixation consists of nailing the geomembrane to the supporting layer. This has been done for the partial repair of Heimbach Dam, a concrete dam. This application will be discussed in Section 4.3. Conceivably, a nailed geomembrane (or a geomembrane with other types of punctual anchors) could be used for the repair or rehabilitation of concrete face or bituminous concrete face rockfill dams. Only the currently used mechanical fixations are discussed hereafter.

Mechanical fixations are bolted to the dam with different methods depending on the characteristics of the face (concrete or bituminous concrete face). Mechanical fixations have the additional advantage of allowing a drainage system behind the geomembrane.

Rehabilitation can be performed on the entire facing (Salt Springs Concrete Face Rockfill Dam, Moravka Bituminous Face Rockfill Dam), as well as only on failing joints between concrete slabs (Strawberry Concrete Face Rockfill Dam). These specific cases are discussed in the following sections.

3.6.3 *Repair of concrete face rockfill dams*

(1) Complete rehabilitation

Salt Springs Dam, 101 m high, is an interesting example of repair of a concrete face rockfill dam because the installation of the geomembrane was carried out in two subsequent years and it was possible in the second year to inspect the geomembrane system installed in the first year.

Constructed from 1928 to 1931, Salt Springs is the 5th oldest concrete face rockfill dam in the world and the first concrete face rockfill dam to reach 100 meters in height. The initial construction procedure consisted of placing dumped rock in a lift approximately 50 m high at its maximum up to about elevation 1158 m, about 50 m below final crest elevation. Because of the large lift and rock placement techniques, the materials tended to segregate, with larger boulders (about 45 t) generally at the bottom and smaller boulders (about 3 t) on top. This resulted in poor consolidation that was soon evidenced by the large settlements measured during the first year of construction. In 1930, the dam construction was halted. The rock placement that followed in the remaining two years (1930 and 1931) of construction was changed. Lifts were reduced to about 23 m in height and significantly more water was used for sluicing the materials to achieve better consolidation. As a result, a weak zone was created in the dam at the transition zone (Larson et al. 2005).

The upstream face of Salt Springs Dam consists of reinforced concrete slabs, from 0.30 m thick at the crest to 0.90 m thick at bottom. Each slab has an approximate surface area of 18 m^2, on an average slope of 1V:1.3H. The initial rock placement procedures left the dam with an inherent settlement problem that caused repeated cracks to the upstream face. Repairs to the concrete face slab have been reported in 19 separate years from 1958 to 1995, with concrete patching, concrete overlay, joint repairs, and/or joint sealant. The continued attempt to use a rigid material (concrete) unable to accommodate continued movement and settlement of the dam, especially in the transition zone at elevation 1158 m, did not provide a long-term repair solution. As in other concrete face rockfill dams having problems with the cracking of the slabs, a flexible liner was more adequate to provide the desired leakage control and also accommodate any future settlement of the dam. In fact, the leakage rate per unit area was of the order of 100 liters/hr/m^2, which is two orders of magnitude above the leakage rate that can be expected with a geomembrane. Clearly a geomembrane had to be used.

The new liner is a composite geomembrane consisting of a 2.5 mm thick PVC geomembrane, laminated during fabrication to a 500 g/m^2 needle-

punched nonwoven geotextile. Due to the extreme roughness of the facing, a 2000 g/m^2 needle-punched nonwoven geotextile was placed directly on the deteriorated concrete to smooth irregularities of dam face and decrease surface preparation costs. The composite geomembrane, as it practically is always the case in dam rehabilitation, was left exposed.

Face anchorage and perimeter seals are the same that are generally used on concrete and as such discussed in Section 4.

Not to impact on operation of the dam, which is used for hydropower, the installation of the geomembrane system was scheduled in two years. As described by Larson et al. (2005), a seepage evaluation analysis was performed to define the optimum area to be waterproof to provide the best benefit/cost ratio.

The geomembrane installation was carried out in that area in February-March 2004. After summer filling, the reservoir was dewatered for the second waterproofing campaign and the geomembrane installed in 2004 could be inspected. Observations showed that the geomembrane had imprinted on the rough subgrade under the 60 m water head, accommodating the big holes and irregularities with no damage. Figure 23 shows the remarkable deformation of the geomembrane on the large cavities of the subgrade, with no rupture. The figure also shows how the surface preparation was limited, as significant voids did not have to be completely backfilled, but rather just smoothed with concrete and extra layers of geosynthetics.

Figure 23. Salt Springs dam, a 101 m high concrete face rockfill dam, USA: after repeated ineffective repairs with shotcrete and concrete, permanent repair was made with an exposed PVC geocomposite, installed in two phases to minimize impact on operation. At left, typical section of the concrete face after 70 years of service and 19 repairs, with the shotcrete overlay repairs in the background. At right, the PVC geocomposite installed in Phase 1 after exposure for one full season. Under the 60 m water head, the PVC geocomposite conformed without ruptures to the rough surface, which needed only minimum smoothening works. After installation, seepage has been reduced below the required acceptable level.

(2) Repair in a limited area

Repair can also be made only at critical areas. A patented external waterstop system has been successfully used for this purpose at Strawberry Concrete Face Rockfill Dam in USA (Scuero et al. 2005).

Strawberry dam is the second oldest concrete face rockfill dam in the world. Located in California it is owned and operated by Pacific Gas and Electric Company (PG&E). The dam, constructed from 1913 through 1916, is 43 m high. The nine vertical expansion joints have no waterstops. Each winter the face is fully exposed, leading to deterioration of the concrete for freeze/thaw. Leakage went up by 1998 to 595 l/s. In the spring of 1999, the California Division of Safety of Dams requested that PG&E take remediation measures to reduce the leakage. Temporary repairs using polyurethane caulking in the dam's nine joints reduced leakage to a maximum of just under 566 l/s.

Alternatives considered by PG & E for permanent joint repair were mastic, asphalt, concrete, or a synthetic liner. The acceptance criteria were a minimum reduction of 75 percent (for 100% coverage of all 9 joints) below leakage rates of 1998.

The synthetic liner option (exposed waterstop) was selected based on performance, on compatibility of installation with climatic conditions during wintertime, and on costs.

The work began in January 2002, with concrete surface preparation of the nine vertical joints. The liner system consisted of four layers, 3 for support and one for waterproofing.

The first support layer, anchored on one side by impact anchors into the new shotcrete or existing sound concrete, is a geocomposite (2.5 mm thick PVC geomembrane coupled to a 500 g/m² geotextile). This layer is 0.70 m wide to completely cover the joint. The layer was affixed to the face using drilled impact anchors to hold the geocomposite in place.

The second support layer is the same, 0.70 m wide and anchored by impact anchors installed on the opposite side.

The third support layer is a thick non-woven geotextile (1000 g/m²), installed over the two support geocomposites and anchored along its vertical edges. The geotextile provides cushioning for the final composite geomembrane.

The waterproofing geomembrane is a composite geomembrane made of the same material as used in the first two support layers.

The waterproofing geomembrane is anchored around the entire perimeter with a stainless steel batten strip, fastened to the concrete by anchor bolts at about 0.15 m spacing. The completed "external waterstop" system is about 1.0 m wide (Figure 24).

In May 2002, when 6 out of 9 joints had been repaired with the external waterstop system as scheduled, the reservoir filled in for the season. The recorded leakage has been about 85 percent below the 1998 leakage rates, more than adequately meeting the acceptance criteria and well below the historic levels. Consequently, PG&E proposed to the authorities that the completion of remaining 3 joints be de-

ferred until the seepage reached a predetermined level. This proposal was accepted and the work to complete the waterproofing of the joints was put on indefinite hold. PG&E will continue to measure the leakage; and, if it increases, installation of the external waterstop system on the remaining joints will be implemented.

Figure 24. Strawberry Concrete Face Rockfill Dam, USA: the second support layer is applied to the joint. Six joints out of a total of nine were waterproofed, reducing leakage below the target.

3.6.4 *Repair of bituminous concrete face rockfill dams*

If a geomembrane is used to repair dams having a bituminous concrete face, the anchorage system is designed in function of the particular characteristics of the subgrade, which being not a rigid material like concrete does not always allow using the type of anchors used on concrete.

For anchoring the profiles or batten strips that provide face anchorage, typically field testing is made to verify if chemical phials or grouted anchors or deep anchors must be adopted. Chemical phials require a stronger and not very viscous subgrade. Grouted anchors and deep anchors (Duckbill/Manta Ray type) can be used in alternative.

At Moravka, a 39 meter high earthfill dam in the Czech Republic used for hydropower, potable water supply, and flood control, an exposed PVC composite geomembrane was placed on the bituminous concrete facing that despite several repairs, including a new bituminous concrete layer, continued to exhibit leakage. An asset of geomembrane systems in this type of dams is that they do not require milling of the deteriorated bituminous concrete, which on the contrary is necessary if a new bituminous concrete layer is installed.

Pull out field testing was carried out at several locations of the facing to ascertain if chemical anchors could be used to fix the tensioning profiles for face anchorage. The tests were successful, but since the resistance of the bituminous concrete could vary over the year depending on atmospheric temperature, the conventional chemical anchors were modified to ensure stability.

Two bottom perimeter seals were installed at the concrete block where the drainage gallery is located. The primary seal confines the drainage system of the upstream face (geomembrane system); the secondary seal confines the drainage system for water coming from foundation/abutments/failing joints in the concrete. The two drainage systems discharge in the gallery with separate discharge pipes to allow monitoring the system (Figure 25).

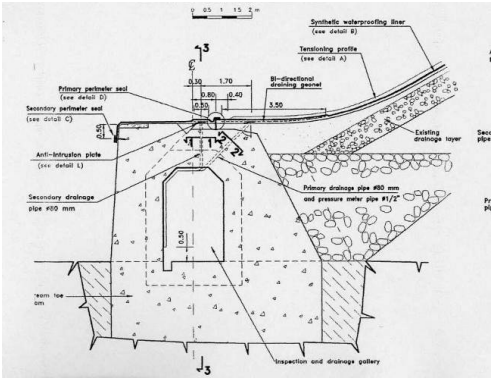


Figure 25. Moravka Bituminous Concrete Face Rockfill Dam, Czech Republic: the two bottom seals.

The secondary seal made on concrete is of the tie-down type (see Section 4.4.4). The primary seal made on bituminous concrete is of the insert type (Figure 26); the watertight resin making the seal allowed maintaining watertightness at the fissures intercepted by the seal.

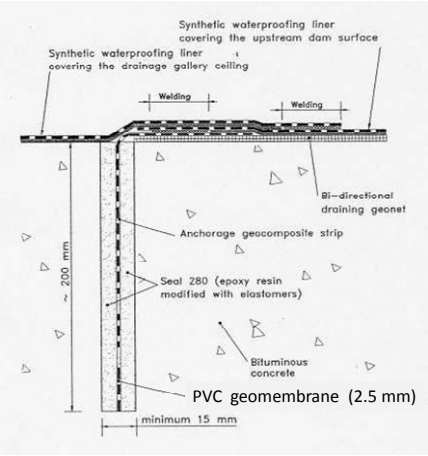


Figure 26. Moravka Dam: the insert type bottom seal on bituminous concrete

The exposed geomembrane system was released in 2009 from the ten years warranty. The geomembrane has resisted ice impact (Figure 27).

Figure 27. Moravka Dam: heavy ice impact did not damage the exposed PVC geomembrane.

3.7 *Performance of embankment dams with geomembrane*

A typical rate of leakage per unit area observed in the case of large earth and rockfill dams constructed with a geomembrane liner on the upstream face and covered is of the order of 1 liter/hr/m^2 (assuming that monitoring is accurate, which is not guaranteed) for the best cases and up to 10 liter/hr/m^2 for dams that do not have the best performance. For the sake of comparison, one defect per 1000 m^2 with a diameter of 2 mm gives a calculated leakage rate of the order of 0.1 liter/hr/m^2 with a typical water head. It is possible that leakage at peripheral connections may explain the difference.

However, in the case of geomembranes anchored on a concrete face rockfill dam, the rate of leakage is of the order of 0.1 liter/hr/m^2 for the best cases. It appears that the typical leakage rate in the case of geomembranes anchored on a concrete face rockfill dam is an order of magnitude times less than the typical rate of leakage through geomembranes installed conventionally in embankment dams. This may be due to better installation on rigid surface.

3.8 *Safety in case of geomembrane failure*

Exposed geomembranes can be damaged by accidental exceptional events (vandalism, terrorism) or by repeated or occasional aggression from the service environment (animals, ageing). Upstream geomembranes, exposed or covered, can be damaged by extraordinary events such as earthquakes or falling aircrafts.

As indicated in Section 3.2.2, the design of a dam with a liner (any type of liner) should be such that the seepage resulting from a major breach in the liner should not cause the rupture or a major distress of the dam. Therefore, when a geomembrane liner is considered for an embankment dam, it is essential to check that the appropriate design precautions have been taken. If it appears that the dam may be sensitive to internal erosion, the leakage control measures described in Sections 3.2.2, 3.23 and 3.2.4 should be taken.

It is important to monitor and maintain dams. Defects, if any, in geomembranes generally have no detrimental consequences, because a geomembrane with a small number of small punctures is still less pervious than other liners. Significant defects in geomembranes, due to the exceptional causes mentioned above, should be repaired if they are likely to have detrimental consequences. Fortunately, it is generally easy to repair geomembranes. Damage repair, at least for geomembranes having suitable characteristics, is feasible by simple patching if in the dry, or by underwater installation. This is possible for exposed geomembranes, and even, if necessary, for covered geomembranes.

In the case of internal geomembranes, the only cause of significant damage to the geomembrane could be a major earthquake. It is important, in seismic areas, to design the dam structure accordingly and to use a geomembrane with high elongation capability and high puncture resistance since the geomembrane could come in contact with rocks in case of malfunctioning of the protective layers. An internal geomembrane with high elongation capability is probably the safest possible liner in case of earthquake.

In conclusion, with appropriate dam design, and proper geomembrane selection, the geomembrane liners are very safe.

4 CONCRETE AND MASONRY DAMS

4.1 *Importance of imperviousness for concrete and masonry dams*

Maintaining imperviousness is critical for concrete dams, because water infiltration modifies the uplift pressure that was taken into account when designing the dam. A typical design approach in new conventional concrete dams, based on considering a relatively tight upstream face, is to assume a 50% uplift pressure reduction immediately behind the upstream face, and then a linear reduction to a zero pressure (or to the pressure due to the tailwater when present) at the downstream face, with a reduction of uplift at any line of drilled or formed internal drains, whose extent varies depending on the rules or standards in each country, but is approximately of the order of 2/3.

During the service life of the dam, the imperviousness of the upstream concrete generally decreases. Infiltrated water may wash the content of fines and of lime, reducing the strength of the concrete, clogging the drains, thereby reducing their effectiveness in providing the uplift reduction that was taken into account at design stage.

Even when seepage does not reduce the stability and safety of the dam to critical values, still another detrimental aspect is the appearance of seepage at the downstream face, which may be interpreted by the public as a sign of poor construction and poor maintenance, inducing a general feeling of lack of safety, which does not necessarily correspond to reality.

4.2 *Use of geomembranes in new concrete and masonry dams*

Considering the impressive success of geomembranes in the rehabilitation of old concrete and masonry dams (see Sections 4.3 to 4.7), geomembranes could conceivably be used for the construction of new concrete and masonry dams. However, to the best of our knowledge, geomembranes have only been used during construction at three concrete dams for a minor application. In these three dams, the geomembrane has been installed at the heel, to act as elastic waterstop at the peripheral joint between the upstream face and the plinth. (The plinth is the concrete beam located at the periphery of the upstream face to ensure proper connection between the face and the surrounding terrain. The plinth is often used as the starting point for grouting the dam foundation.)

The rest of Section 4 is devoted to the rehabilitation or repair of existing concrete and masonry dams.

4.3 *Use of geomembranes in the rehabilitation of concrete and masonry dams*

Extensive application of geomembranes on concrete and masonry dams was carried out initially in Europe, at the beginning of the 1970s. In almost all cases, the geomembrane is installed on the entire upstream face to restore imperviousness to the deteriorated facing. To the best of our knowledge, there are only two concrete and masonry dams on which a geomembrane was installed locally to repair a joint or a crack.

In rehabilitation of concrete and masonry dams, only PVC, LLDPE, CSPE and CPE-R geomembranes have been used, as listed in Table 1. Neither bituminous nor HDPE geomembranes were used.

Table 1. Geomembranes (GM) in concrete dams.

Dam type	GM	PVC	LLDPE	CSPE	CPE-R
Gravity	Expos.	31	0	0	0
	Cover.	1	0	0	0
Buttress	Exp.	3	0	0	0
	Cov.	0	0	0	0
Arch	Exp.	3	0	1	0
	Cov.	0	2	1	1
Multiple arch	Exp.	9	0	0	0
	Cov.	0	0	0	0
Total	Exp.	46	0	1	0
	Cov	1	2	1	1

As Table 1 shows, the geomembrane is almost always exposed. The only cases of covered geomembrane on concrete or masonry dams for which we have some data are two partial applications (joint at heel) on high arch dams in Austria.

Dams of remarkable height have been waterproofed using a geomembrane, such as Kölnbreinsperre Dam, an arch dam in Austria, 200 m, and Alpe Gera Dam in Italy, 174 m. In both dams, the geomembrane was installed in the lowest part of the dam face, sustaining the highest water head.

Geomembrane placement is generally carried out in the dry, after emptying all or part of the reservoir. Underwater repair has also been made and is discussed hereafter in Section 6.

One original case is that of Heimbach Dam in Germany. This concrete gravity dam, built in 1937, is 12 m high and 105 m long. A 500 m^2 area of the vertical face was repaired in 1974 using a 3 mm thick PVC geomembrane nailed to the face with one nail per m^2.

4.4 *The state-of-the-art design*

4.4.1 *Overview*

Since, in 87% of all cases of exposed geomembranes on concrete or masonry dams, the same system has been adopted, this system can be considered to be the state-of-the-art design. The state of the art is characterized by the type of geomembrane used, the anchorage system, the peripheral seal, and the drainage behind the geomembrane. These four features are discussed in the following sections.

4.4.2 *The geomembrane used*

The geomembrane used comprises two components: a geomembrane and a nonwoven geotextile. These two components are heat-bonded together at the manufacturing stage, thereby forming a composite geomembrane.

The geomembrane component is a high-stability PVC geomembrane. Its thickness is typically 2 to 3 mm. The geotextile component is a needle-punched nonwoven geotextile made of polypropylene or polyester, with a mass per unit area typically 200 to 700 g/m^2 depending on the conditions of the subgrade and the water head; polypropylene is generally used when contact with fresh concrete is foreseen, such as in case a new concrete or shotcrete layer is placed on the dam to increase its stability. While the geomembrane performs the waterproofing function, the geotextile performs several functions: it reinforces the geomembrane, thereby reducing sagging of the vertical geomembrane; it protects the geomembrane against mechanical damage by irregularities of the supporting medium; and it contributes to drainage behind the geomembrane.

4.4.3 *Anchorage*

A face anchorage system keeps the geomembrane attached to the upstream face avoiding displacement of the geomembrane by wind and waves, and preventing sagging of the geomembrane due to creep.

Anchorage is always mechanical, with only one exception, Zolezzi Dam in Italy, 1992, where anchorage was made by gluing (Cazzuffi & Sembenelli 1994). Face anchorage is almost always made with a well known patented anchoring and tensioning system (ICOLD 1991). The tensioning system consists of two stainless steel ribs (generally referred to as "profiles"), the first one, a U shaped internal profile, fastened to the dam upstream face, and the second one, an omega shaped external profile, installed over the composite geomembrane (Figure 28). The geometry of the two profiles is such that, when they are tightly connected, they secure the composite geomembrane to the upstream face and they pre-tension it. Pre-tensioning prevents the composite geomembrane from becoming loosened or wrinkled during service. Tensioning is preferable (ICOLD 2010) for the safety and durability of the system. If the composite geomembrane is not adequately tensioned, the repeated loads to which it will be subjected during its service life (waves and wind, varying water levels, etc.) will over time cause formation of slack areas and folds, which are places for potential concentration of stresses that can lead to faster ageing of the geomembrane. In cold climates, tensioning prevents ice from sticking to the geomembrane and dragging down the composite geomembrane.

The spacing between the vertical profiles is dictated by the load conditions and the width of the composite geomembrane rolls or geomembrane panels (sometimes two or more 2.10 m wide rolls are joined to form larger panels and expedite construction). Typical spacing is 1.80 m, 3.7 m or 5.7 m.

In addition to anchoring and tensioning the composite geomembrane, the vertical profiles create vertical channels that contribute to the drainage capacity of the system (see Section 4.4.4).

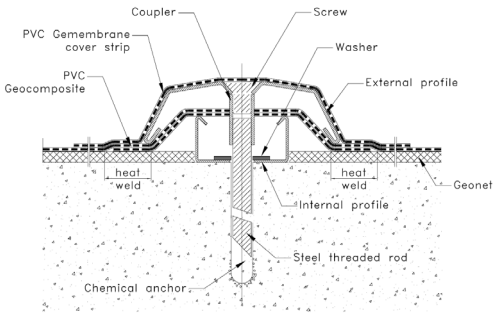


Figure 28. Cross section of the patented anchoring and tensioning system used for concrete dam rehabilitation.

4.4.4 *Peripheral seal*

A peripheral seal impedes water infiltration behind the geomembrane. The seal is of the tie-down type, that is to say a mechanical seal made by compressing the geomembrane on the concrete by stainless steel batten strips and appropriate gaskets and splice plates. It is designed to be watertight against the water pressure where the seal is positioned, i.e. rain and snowmelt at crest, and maximum water head at submersible boundaries.

4.4.5 *Drainage*

There is always a drainage system between the dam face and the geomembrane. The drainage system is composed of: (1) the geotextile component of the composite geomembrane: (2) a "face drainage layer", i.e. the gap created by the tensioning system between the composite geomembrane and the face of the dam; (3) when more flow capacity is needed, an additional layer of transmissive geosynthetic (such as a geonet); (4) the drainage collection conduits formed by the vertical tensioning profiles; (5) a bottom collector; and (6) one or more discharge pipes. Discharge is made in the gallery or downstream. Discharge inside the reservoir would also be possible, using one-way patented discharge valves as adopted in canals and pressure tunnels; however, to the best of our knowledge, this has not been done.

The drainage system performs three functions: (1) control seepage flow from the reservoir ("infiltration water"); (2) drain water from the dam ("saturation water"); and (3) monitor the performance of the geomembrane. These three functions are discussed below.

The imperviousness of the geomembrane and the associated drainage system provide an effective control of seepage flow from the reservoir through the upstream face of the dam.

Over the years prior to the rehabilitation of the dam using a geomembrane, water has progressively saturated the dam. The drainage system performs progressive dehydration of the dam. The mechanism is the following: water present in the dam migrates through the concrete towards the warmer upstream face of the dam, especially when the reservoir is empty and under the action of solar radiation. The impervious geomembrane stops the migration; the drainage layer intercepts the water behind the geomembrane, and conveys the water by gravity to bottom collection and discharge. Thus, the dam is progressively dehydrated of saturation water. As a result, pore pressure is reduced, the risk of clogging of drilled drains is reduced, and the phenomenon of concrete swelling due to alkali-aggregate reaction (AAR) is slowed down (Scuero & Vaschetti 2008).

The presence of a drainage system is of particular advantage in concrete dams affected by AAR, as it can deprive the dam of some of the water feeding the reaction. Also, drainage prevents water from ap-

plying pressure in existing cracks and causing uplift. At Pracana Dam, a 65 m high buttress dam in Portugal, affected by AAR, an exposed drained composite geomembrane system was installed in 1992. The drainage layer is a geonet located on the entire upstream face underneath the composite geomembrane. Monitoring the behavior of the drained waterproofing system has demonstrated its capability of dehydrating the dam. After 11 years of service of the dam, the owner reported "the dam waterproofing may be assumed to contribute for the reduction of the swelling process" (Liberal et al. 2003).

To monitor the performance of the geomembrane, the drainage system along the face of the dam is sometimes divided in sections. This makes it possible to locate a leak in the geomembrane.

4.5 *Rehabilitation of dams with concrete or shotcrete facing*

In dams with concrete or shotcrete facing, the composite geomembrane consists typically of a 2.5 mm thick PVC geomembrane coupled during fabrication to a 500 g/m^2 needle-punched nonwoven geotextile. The composite geomembrane is generally installed over the deteriorated face, after hydroblasting to remove unstable parts, and removal of foreign material. Excessively aggressive surfaces and deep voids may require patching with mortar, or using additional layers of supporting geosynthetics. Excessively damaged shotcrete layers may require total removal due to the instability of the shotcrete. In a few cases when additional weight was required for stability of the dam, the old shotcrete was removed, a new shotcrete layer was placed, and the composite geomembrane was installed over it not only to provide imperviousness, but also to protect the shotcrete from freeze-thaw deterioration.

In gravity and arch concrete dams, the composite geomembrane rolls, manufactured in adequate length to avoid transverse joining, are deployed vertically from the crest, and are secured to the dam by vertical parallel lines of anchoring and tensioning profiles (Figure 29).

Figure 29. Old and recent examples of drained tensioned exposed PVC geomembrane on concrete dams. At left, Lago Nero gravity dam, Italy, waterproofed in 1980 and pictured in 2003, where the exposed geomembrane was installed on the new shotcrete, In 2010, it is in still service. At right, Saint Marc gravity dam, France, rehabilitated in 2008.

When increased drainage capability is required, a drainage geonet is placed under the composite geomembrane. A cross section and a typical example are shown in Figure 30.

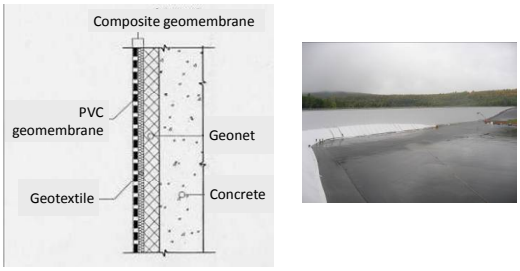


Figure 30. At Waldeck gravity dam in Germany, 2008, the composite geomembrane is placed on a drainage geonet.

Installation is made horizontally (Figure 31) if the geometry of the dam so requires, such as in multiple arch dams, because horizontal installation allows for better adapting the geomembrane sheets to the surface. A flexible geomembrane and good craftsmanship are crucial. Particular operational needs, such as at pumped storage schemes, may require installing the geomembrane system horizontally and in separate campaigns.

Figure 31. Examples of horizontal placement. At left La Girotte Dam, France, a multiple arch dam with a complex geometry; at right, Scais Dam, Italy, a buttress dam, for a pumped storage station.

4.6 *Rehabilitation of dams with masonry facing*

In dams with masonry face, the significant roughness of the dam face generally requires installing an additional anti-puncture layer underneath the composite geomembrane. Thick needle-punched nonwoven geotextiles (mass per unit area typically ranging from 1000 to 2000 g/m^2) or geonets are used, depending on the sharpness of the masonry. A remarkable example is Kadamparai Dam, a 67 m high, 478 m long, gravity dam in India. The dam is used as a forebay reservoir to the Kadamparai Pumped Storage Hydro Electric Project.

The waterproofing system installed on the gravity section is the same patented tensioning system described previously. A 2000 g/m^2 polyester needle-punched nonwoven geotextile was placed on the very sharp stone masonry, to protect the PVC composite geomembrane against puncturing.

The geomembrane system is integrated with a dedicated leak detection system, the Optical Fiber Cable system developed by the Technical University of Munich, Germany. The system can detect the area of a leak in the geomembrane, based on the difference in temperature that is caused by leaking water and is signaled by the optical fiber cable. The system has been adopted at other dams, such as Brändbach Dam and Waldeck Dam in Germany and Winscar Dam in UK.

Waterproofing works at Kadamparai Dam (Figure 32) involved installation of more than 17,300 m^2 of composite geomembrane and were completed in three months, i.e. six weeks ahead of schedule, allowing generating power earlier than expected. At full supply level, seepage that before installing the geomembrane was of the order of 3×10^4 l/min, has been reduced to around 1×10^2 l/min after installing the geomembrane. As reported by Sadagopan & Kolappan (2005): "The installation of the exposed PVC geocomposite mechanically anchored and drained, has more than confirmed the expectations of TNEB *[the owner]* when it selected the system as rehabilitation measure that could provide efficient seepage control."

Figure 32. At left, the face of Kadamparai Dam, a stone masonry dam, India, before waterproofing. At right, the completed works at Kadamparai Dam, 2005.

4.7 *Safety in case of geomembrane failure*

It is important in dam design to evaluate the consequences of a failure of the waterproofing element. In the case of concrete and masonry dams rehabilitated using the drained geomembrane system described in the preceding sections, the drainage system is designed so as to ensure that water seeping through defects in the geomembrane or at its peripheral connections is captured by the drainage system and removed before it can percolate into the dam. In the extreme case of a large breach in the geomembrane, the leakage rate may exceed the capacity of the drainage system (especially if there is no geosynthetic drain such as a geonet behind the composite geomembrane). However, the resulting water pressure on the dam cannot exceed the water pressure that existed before installation of the geomembrane when the dam reservoir was full. Therefore, even a total rupture of the geomembrane has no impact of safety. The only consequence would be to empty the reservoir and repair the geomembrane, unless repair can be done under water, as discussed in Section 6.

5 ROLLER COMPACTED CONCRETE DAMS

5.1 *Overview of uses of geomembranes in RCC dams*

A summary of uses of geomembranes in roller compacted concrete (RCC) dams is presented in Table 2. In RCC dams, geomembrane systems have been used mostly in new construction. In the case of new construction, the geomembrane has mostly been used to waterproof the entire upstream face, and only three cases of waterproofing of joints are reported. So far, to the best of our knowledge, geomembranes have been used only in three cases for the repair of existing RCC dams initially constructed without geomembrane. This may be due to the fact that RCC dams are relatively recent.

The geomembrane can be left exposed or be covered, as discussed in subsequent sections. With the possible exception of one RCC dam for which the geomembrane type is not known to us, all RCC dams where the geomembrane is exposed use a PVC geomembrane. With the exception of two of the 19 RCC dams where the geomembrane is covered, a PVC geomembrane is used.

Table 2. Geomembranes (GM) in RCC dams.

GM location	GM	PVC	LLDPE	HDPE
Entire face	Exposed	15	0	0
	Covered	19	1	1
Joints	Exposed	3	0	0
	Covered	0	0	0
Cracks	Exposed	3	0	0
	Covered	0	0	0

5.2 *Liner system concept in RCC dams*

It is beyond the scope of this paper to discuss all the methods for controlling seepage through RCC dams. Herein, only one approach is discussed. It is an approach that separates the static function and the waterproofing function: the RCC mass provides the stability of the dam, and an upstream geomembrane provides the watertightness.

In RCC dams, the watertightness provided by an upstream geomembrane has numerous beneficial effects, as discussed below.

In RCC dams the presence of an upstream geomembrane allows a conservative 50% reduction of the design uplift pressure at the upstream face. If there is a face drainage system behind the geomembrane, the reduction in uplift within the dam body occurs at the upstream face and is theoretically 100% (Scuero & Vaschetti 2005). Clearly, reduction of the design uplift is one of the advantages that a drained geomembrane system can provide.

Another benefit from the watertightness provided by geomembranes at the upstream face is the prevention of preferential seepage along compaction lifts. RCC dams are constructed by spreading and compacting the concrete in 0.30 m high lifts. The numerous (1 joint every 0.30 m) horizontal lift joints, if not perfectly treated, may create leakage paths. Only one seeping lift joint is enough to affect uplift and stability, especially in case of seismic events when water seepage can hydro-jack the lift.

Thermal cracking is a critical issue in RCC dams, especially in case of RCC mix having high cement content. The deterioration mechanism described for concrete dams can be expected to develop over time also in RCC dams. Constant seepage can lead to the deterioration of the RCC by leaching out of cement. This can affect stability in the long term, especially in case of a seismic event.

In summary, an upstream drained geomembrane prevents seepage in lift joints, allows a reduction of the design uplift, prevents intrusion of water in cracks that could form due to thermal constraints, thereby avoiding the risk of internal water pressure, and avoids future concerns about the watertightness of the upstream face, including contraction joints and joints between RCC and conventional concrete. The numerous design and construction constraints that can be significantly reduced thanks to the use of an upstream geomembrane in RCC dams have been widely discussed in the literature (Herweynen 2006).

There are two options: exposed geomembrane and covered geomembrane. These two options are discussed in subsequent sections.

5.3 *RCC dams with exposed geomembrane system*

5.3.1 *The system*

The exposed geomembrane system for RCC dams is illustrated in Figure 33. Out of 21 dams lined by an exposed PVC upstream geomembrane system, 3 are in China and the others have adopted the same patented system. The exposed system has been applied in very different climates, ranging from humid equatorial climates (Colombia), to hot desert climates (Jordan), to very cold climates with temperatures down to -50°C and large temperature excursion (Mongolia).

The composite geomembrane consists of a 2.5 mm or 3.0 mm thick high performance PVC geomembrane, laminated during manufacturing to a needle-punched nonwoven geotextile. The composite geomembrane is placed against the completed RCC lifts, covering the upstream face including all joints, from crest down to the heel. The composite geomembrane is fastened by a face anchorage system and by perimeter sealing.

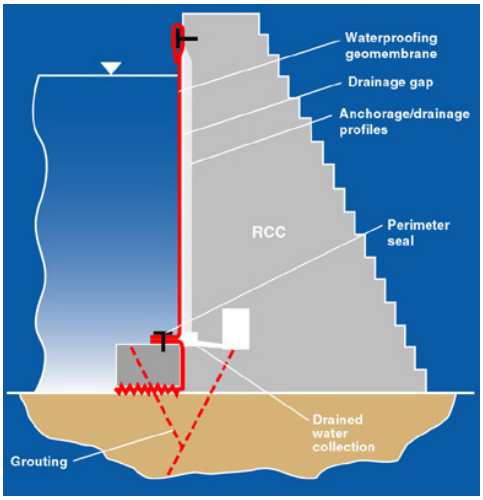


Figure 33. Concept of the exposed geomembrane system in RCC dams.

The face anchorage system uses the concept adopted for the rehabilitation of concrete dams: anchorage by vertical lines, with a tensioning system consisting of two stainless steel profiles, the first one, U shaped, fastened to the dam upstream face, and the second one, omega shaped, installed over the composite geomembrane.

In RCC dams the tensioning system can be installed in two possible configurations: the U profile can be embedded into the dam during construction ("embedded profile", Figure 34), or it can be installed after the dam is completed, on the finished surface ("external profile"), and, in such a case, the configuration is identical to the one described for the rehabilitation of concrete dams (see Figure 28).

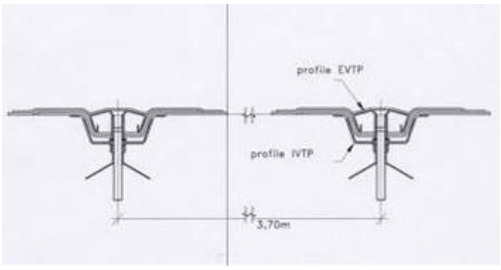


Figure 34. The tensioning system with embedded "U" profile.

The two options are technically equivalent. While in the external configuration the time for installing the external U profiles is under the full control and responsibility of the waterproofing contractor, in the embedded configuration the time for embedding the U profiles is under the responsibility of the main

contractor and is subject to uncertainties: if the main contractor embeds the profiles in the proper way while placing the RCC, the total time for installation of the entire waterproofing system may be lower; but if, on the contrary, the profiles are not embedded in the proper way, the main contractor must make the necessary corrective actions to comply with specifications, which may entail significant additional construction time (and costs). A further advantage of the external configuration is that it is less dependent on the skills and production rate of the main contractor and it requires less quality control on the civil works.

The profiles are placed vertically at regular spacing, typically 3.7 or 5.7 m depending on the design loads. Since they keep the liner fastened but not intimately adherent to the dam face, as discussed for the rehabilitation of concrete dams, the profiles allow the formation of a "face drainage layer" behind the composite geomembrane. The face drainage layer, and the vertical conduits formed by the tensioning profiles, facilitate the flow of drainage water by gravity to the drainage collection and discharge system at the bottom of the upstream face of the dam. If it is necessary to increase the flow capacity, a geosynthetic drain can be used between the RCC and the composite geomembrane. The geosynthetic drain can be a geonet. The drainage system is ventilated, to prevent the development of suction behind the composite geomembrane.

The composite geomembrane is anchored along its periphery by a perimeter seal, which prevents water from infiltrating behind it, as discussed for the rehabilitation of concrete dams.

The rolls of composite geomembrane cover the full height of the dam face, without horizontal joints unless required by site-specific reasons (e. g. in case of very high dams or in case of installation carried out in separate horizontal sections). Adjacent rolls are vertically joined by watertight welds. The rolls of composite geomembrane are placed directly over the upstream face, or over a layer of drainage geosynthetic. In front of the contraction joints of the dam, one or more layers of geosynthetics are placed behind the composite geomembrane, to support it over the joint. The number and type of these layers depend on the anticipated width of opening of the joint, rotation of blocks, and hydrostatic pressure, and depend on the factor of safety, etc.

Miel I RCC Dam and Boussiaba RCC Dam are examples of the two systems.

5.3.2 *Examples of RCC dams with embedded profiles*

At 188 m, Miel I is the world's highest RCC dam at present in full operation. It is a straight gravity dam constructed in a narrow gorge in Colombia. To meet contractual schedule, the original design of an upstream face made of slip formed reinforced concrete was changed to a drained exposed PVC composite geomembrane system, placed on a 0.4 m wide zone of grout-enriched vibrated RCC. This double water shield was considered necessary due to the height of the dam (Marulanda et. al. 2002). The use of grout enriched RCC allowed applying good compaction of RCC mix at the dam face, thereby ensuring a good finishing of the upstream concrete surface. The RCC mix has a cement content of 85 to 160 kg/m^3; the contraction joints (which are vertical) are placed every 18.5 m.

The waterproofing liner is a composite geomembrane, consisting of a PVC geomembrane laminated to a 500 g/m^2 polypropylene nonwoven needle-punched geotextile in the factory. In the lower 62 m of the dam, from elevation 268 m to elevation 330 m, the PVC geomembrane is 3 mm thick, and in the upper 120 m, from elevation 330 m to elevation 450 m, it is 2.5 mm thick. The entire upstream face has a surface area of 31,500 m^2.

The attachment system for the composite geomembrane on the dam face is made by parallel vertical anchoring and tensioning profiles, placed at 3.70 m spacing. The first component of the tensioning profile assembly was attached to the formworks, and embedded in the 0.3 m high RCC lifts (Figures 35 and 36).

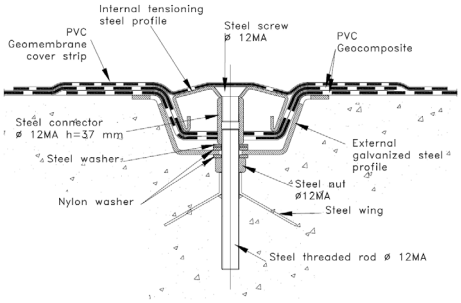


Figure 35. Embedded tensioning profile at Miel I RCC Dam.

Figure 36. Miel I RCC Dam, Colombia 2002. At left and middle, the U profiles and drainage collector attached to the formworks are embedded in the RCC. At right, the U profiles after embedment appear as vertical grooves in the face of the dam.

The integrated face drainage system behind the composite geomembrane consists of

- the face drainage layer created by the gap between the liner and the dam face, and by the geotextile laminated to the PVC geomembrane;
- the vertical conduits formed by the tensioning profiles;
- a peripheral collector embedded in the RCC and the transverse discharge pipes discharging into the gallery; and
- a ventilation pipe assuring water flow at ambient pressure.

The drainage system is divided into four horizontal sections, each discharging in the gallery located at its bottom. Each horizontal section is in turn divided into vertical compartments with separate discharge. In total there are 45 separate compartments, achieving very accurate monitoring of the behavior of the waterproofing system.

In front of the contraction joints, the composite geomembrane is supported by two layers of the same composite material. The PVC composite geomembrane was installed in six sections, to allow early impounding while the dam was still under construction, and to follow at best the division of the drainage system into horizontal sections. A movable railing system was used to install the composite geomembrane concurrent and independent of RCC activities (Figure 37). The railing system was attached to the dam face at first at approximately 90 m above foundation, and then moved to approximately 140 m above foundation. The travelling platforms, from which all activities were carried out, were suspended from the railing system.

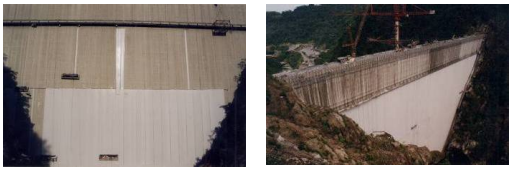


Figure 37. Miel I RCC Dam, Colombia 2002. At left, the composite geomembrane installation almost completed in the lower section, and starting in the section above, where placement of the support geocomposite on the contraction joints is being performed from the travelling platform. At right, installation of the composite geomembrane with travelling platforms suspended at a movable railing system placed at elevation 407 m, while RCC placement is ongoing above the railing system.

Construction of the grouting plinth at the heel of the upstream face took place after the placement of the RCC. A PVC composite geomembrane, placed against the completed RCC lifts and over the natural excavation rock, waterproofs the plinth. The composite geomembrane waterproofing the plinth is watertightly connected to the composite geomembrane waterproofing the upstream face by a mechanical seal. The seal achieves watertightness by compressing the PVC geocomposite with 80 × 8 mm stainless steel batten strips on the concrete of the plinth leveled out with epoxy resin; rubber gaskets, and splice plates placed at abutting batten strips to ensure that compression is evenly distributed. This type of seal, tested at 2.4 MPa, is placed also at crest to resist water overtopping.

The second component of the tensioning profile assembly, i.e. the omega-shaped profile placed over the installed composite geomembrane and connected to the first component (i.e. the U-shaped profile), secures and tensions the composite geomembrane liner on the upstream face. Where the water head is higher, from elevation 268 m to 358 m, the profiles have a central reinforcement. The profiles are waterproofed with PVC cover strips (Figure 38).

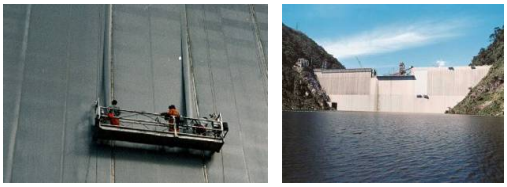


Figure 38. Miel I RCC Dam, Colombia 2002. At left, the stainless steel profiles are waterproofed with PVC cover strips. At right, the composite geomembrane already installed in the lower sections allowed impounding the reservoir while RCC placement and waterproofing works were ongoing above.

The construction of the dam started in April 2000 and ended in June 2002, hence a total of 26 months. The change in design allowed meeting the schedule, and saved several tens of millions US$, because of reduced cement content, faster completion, and earlier power generation.

Olivenhain Dam in California is an example of the same exposed system adopted at Miel I Dam, but with a geonet placed between the dam face and the PVC composite geomembrane to increase the drainage capacity. The dam, 788 m long and 97 m high, at present the highest RCC dam in the USA, is a key element of the Emergency Storage Project of the San Diego County Water Authority, owner of the dam. About 90% of water is brought to San Diego from hundreds of kilometers away, and the aqueducts cross several large active faults, including the San Andreas Fault. The reservoir will provide water to the San Diego region in case of an interruption in water delivery due to an earthquake or a drought.

Evaluation of the alternatives for the upstream face, considering the magnitude of the design earthquake and the critical function of the dam to provide water during an emergency, placed emphasis on seismic stability and seepage control. In a range of 1 to 3, these features were assigned the maximum weighting factor of 3 (Kline et al. 2002). Special consideration was also given to construction se-

quence because the dam had to be fully operational within a certain date.

In the stability analysis, the exposed geomembrane liner and its face drainage system received the highest score among the 11 considered alternatives.

Shaping blocks and plinth are waterproofed with the same composite geomembrane used for the upstream face, watertightly connected to the composite geomembrane of the upstream face with the same seal as used at Miel I Dam.

The face drainage system is similar to the one adopted at Miel I Dam, with the addition of a drainage geonet installed against the RCC (Figure 39), which will enhance discharge capabilities should accidental damage occur to the impervious geomembrane. Under the hydrostatic pressure of the reservoir water, the geonet will maintain high transmissivity, thereby preventing water from migrating through lift joints in the body of the dam. As a result, saturation levels and pore pressures in the dam will be lowered, with beneficial effects on the stability safety factors, and on appearance at the downstream face.

Figure 39. Olivenhain RCC dam, USA 2003. At left, the black drainage geonet is placed on the RCC to enhance drainage collection and discharge; at right, the composite geomembrane is placed over the geonet.

The composite geomembrane, consisting of a 2.5 mm thick PVC geomembrane coupled to a 500 g/m^2 needled-punched nonwoven polypropylene geotextile, is fastened by vertical tensioning profiles at 3.70 m spacing. The embedded profiles have been designed larger than standard, to create larger vertical conduits and further enhance drainage transmission. Stainless steel plates, 60 × 6 mm, placed every 0.40 m inside the profiles, avoid that the profiles deform under the hydrostatic pressure. Each line of profiles discharges separately into the gallery, and can be individually monitored. Theoretically each line of profiles should intercept water of its area of influence and thus constitute a compartment that can be monitored separately from the adjacent compartments. In reality, there is no certainty that water in between two lines of profiles will flow half in one profile and half in the next one. A watertight seal is necessary to separate compartments. The upstream face of Olivenhain Dam has been divided into 12 compartments, separated by a vertical watertight seal, to allow defining the area of the leak in case of damage to the geomembrane. The peripheral seals are of the type adopted at Miel I Dam.

Waterproofing works were completed in five months; the cost of the exposed geosynthetic system was about 5% of the US$124,959,204 contract for the construction of the dam. The reservoir started filling on 7 August 2003 (Figure 40).

Figure 40. Olivenhain RCC dam at completion of waterproofing system and at first filling on 7 August 2003.

On 16 June 2004, with reservoir almost full (Figure 41), a 5.5 Richter scale earthquake occurred at about 100 km from the dam. The blocks of the dam shook and moved, but no damage has been reported and watertightness of the dam has been totally maintained, fully meeting the design and safety requirements.

Figure 41. Olivenhain RCC dam and reservoir, USA.

5.3.3 *Example of RCC dam with external profiles*

Boussiaba Dam, an RCC dam in Algeria, provides an example of the use of external U profiles. Completed in the summer of 2009, it is part of the Beni Haroun-Boussiaba system and is located about 400 km east of Algiers. The dam will form a reservoir having a capacity of 120 million m^3, and a regulating volume of 80 million m^3. The regulating volume will be used to increase the capacity of the Beni Haroun Transfer Station and the supply of water to the people of the region (Cazzuffi et al. 2009).

The dam is approximately 51 m high and 310 m long, with a 100 m long spillway section, a concrete intake tower at the right abutment, and a grouting plinth along the bottom perimeter. The exposed drained geomembrane system covers the entire upstream face, with the exclusion of the concrete intake tower, from crest down to the plinth.

The waterproofing system has the same components as adopted at Olivenhain Dam, but the U profiles have been installed over the completed dam

body (external configuration) (Figure 42) instead of being embedded in the dam as it was done at Miel I Dam and Olivenhain Dam. Additionally, the drainage bottom collector has been modified: instead of the embedded box drain planned in the original design, a 1m high strip of drainage geonet, of the same type as the one constituting the drainage layer, has been placed as bottom collector along the bottom perimeter of the waterproofing system (Figure 42). This modification simplified construction because the only embedment required was that of the transverse discharge pipes of the five drainage compartments.

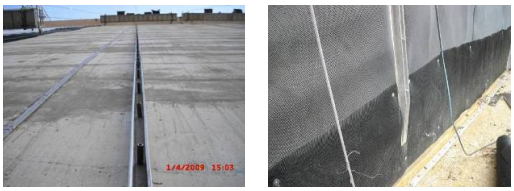


Figure 42. Boussiaba RCC Dam, Algeria 2009. At left, the U profiles installed external to the dam, on the completed RCC lifts. At right, detail of the drainage geonet placed on the upstream face, and of the drainage geonet band placed as longitudinal drainage collector at heel.

The waterproofing system was installed starting at the left abutment and installation proceeded according to the section of the dam completed by the main contractor (Figure 43). Installation of the geosynthetic system started on 24 January 2009 and was completed on 22 May 2009 on schedule, notwithstanding interruptions caused by more than 20% days of bad weather.

The use of geosynthetics, as adopted in the final design, achieved the objectives of simplifying the construction of the dam and of its waterproofing system. The external U profiles configuration made it possible to carry out the waterproofing works without affecting the construction works that were ongoing at the spillway, thereby reducing construction constraints and quality control, and kept installation times and costs within those announced.

Figure 43. Boussiaba RCC Dam, Algeria 2009. At left, installation starting at the spillway section on 30 March 2009; at right, a view of the dam at crews' demobilization on 28 May 2009.

5.4 *The covered geomembrane system*

5.4.1 *The system*

The covered system is also known as Winchester system, from the original name of the dam in the USA where it was first adopted (see Section 2.6.2). The concept is illustrated in Figure 44. The waterproofing geomembrane is attached to the prefabricated concrete panels that are used as the permanent formworks for construction of the RCC dam. The panels are placed so that their concrete layer faces the reservoir, and their geomembrane layer faces the RCC lifts. When construction is completed, the geomembrane is thus embedded between the concrete face and the RCC body.

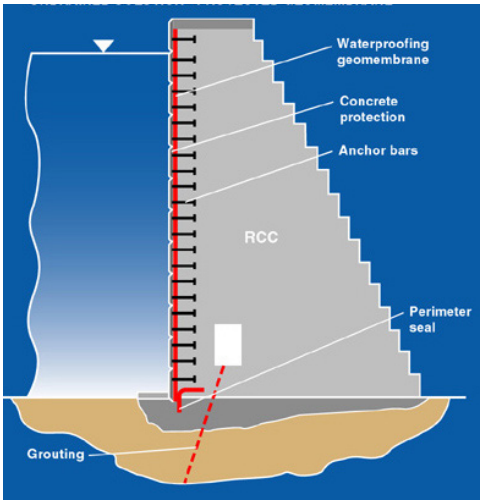


Figure 44. Concept of the covered geomembrane system in RCC dams.

The concrete panels are anchored to the body of the dam using anchor bars (Figure 44) that are placed between lifts of RCC.

Embedment in the concrete provides permanent protection of the geomembrane from all environmental aggressions: mechanical actions (transported floating material, ice, rocks, vandalism), or chemical and climatic aggressions, do not directly affect the geomembrane. On the other hand, a crucial aspect of the covered system is that strict quality control must be implemented to ensure that the geomembrane is not damaged during construction activities related to placement of RCC.

The waterproofing material generally used is a PVC geomembrane, typically 2 mm thick, attached to the concrete of the panels during prefabrication. Originally, geomembranes were equipped with PVC ribs (ribbed geomembranes) to provide attachment to the concrete of the panels. Most recent projects starting from 1998 (Penn Forest Dam, Buckhorn

Dam, Hughes River dam, Hunting Run Dam, Blalock Dam, Hickory Log Dam and Elkwater Fork Dam in the USA, and Paradise Dam in Australia) have substituted the PVC ribs with a geotextile coupled to the geomembrane during fabrication as detailed below.

5.4.2 *Example of covered geomembrane on RCC dam*

An example of application of the covered system on a RCC dam is Paradise Dam (aka Burnett River Dam), an RCC dam, built in Australia in 2004. The dam, 35 m high and used for water supply, is owned by the government of Queensland and is Australia's largest volume RCC dam. Innovation was a key aspect for the design and construction of Paradise Dam. The alliance among the owner, the designer and the contractors produced several innovative approaches, including the use of a 2 mm thick PVC geomembrane that was incorporated in the upstream facing system. The designer (Herweynen 2006) reports that "by adopting an upstream membrane we could guarantee watertightness and we could fundamentally change the design requirements for the RCC mix design. As the membrane provides the watertight function, the concrete provides the stability function." The RCC mix design consists of a lean mix with a final cement content of only 65 kg/m^3. No fly ash is used in the mix, because "the fly ash does essentially nothing except add cost and require additional material to deliver, control and mix" (Herweynen 2006).

At Paradise Dam, the following patented system for attaching the PVC geomembrane to the concrete panels was used: the PVC geomembrane is laminated during fabrication to a 200 g/m^2 polypropylene needled-punched nonwoven geotextile, thereby forming a composite geomembrane that is deployed over the fresh concrete of the panels. The fresh concrete impregnates the geotextile, providing the temporary attachment needed before placement of the panels.

The fastening system by geotextile laminated to the geomembrane allows large displacement of the panels that would not be possible with the rigid blocking typical of ribbed geomembranes that were adopted in former projects. When, during service, the dam body and the panels are subjected to relative movements, excessive shear stresses may develop at the concrete-composite geomembrane interface. The geotextile component of the composite geomembrane will detach from the PVC geomembrane component to an extent that is function of the magnitude of the shear stress. Ultimately, the PVC geomembrane could be attached to the panels only at the location of the anchor bars, and, for the rest of its surface, would be totally independent from the panels. As a result, the PVC geomembrane will be "sandwiched" between two protective and independent concrete layers: the concrete+geotextile on one side, and the RCC, to which it does not stick, on the other side.

Construction is illustrated in Figure 45. Construction of the covered geomembrane system started with preparation for the perimeter seal at the heel of the dam. A PVC geomembrane sheet was embedded in the conventional concrete at the foundation, with an extra width that extends horizontally in the body of the dam, to constitute a horizontal barrier to upward migration of water seeping from the foundation. The first rows of panels were then set in place, with the geomembrane side facing the dam body and the concrete side facing the reservoir. The panels were shaped to allow easy placement and interlocking. After panels had been placed, adjoining panels were connected along their entire perimeter by PVC geomembrane strips heat-welded on the geomembrane of the panels.

The panels were then attached with watertight fittings to the steel bars that anchor them to the dam, and placement of RCC started. As the RCC lifts were being placed, the anchor bars were embedded within them, thus securing the panels to the dam body. Construction proceeded with placement of subsequent rows of panels, repeating the sequential steps described for the first row, until the dam body was completed and the construction of traditional concrete appurtenances could start.

Figure 45. Paradise RCC Dam, Australia 2004. Erection of precast panels, and view from the upstream side.

The designer reports a total estimated seepage at 65% impoundment to be less than 2 l/s, and considers that a majority of this seepage is coming through the foundation rather than the geomembrane.

5.5 *Geomembranes used as external waterstops in RCC dams*

5.5.1 *The system*

Another application of geomembranes in construction of RCC dams is the use of geomembrane strips as external waterstops on the vertical contraction joints. A geomembrane strip (Figure 46) is installed on the completed RCC, and is supported over the joint by a support structure, which prevents the geomembrane strip from intruding in the active joint. The support structure must be effective at maximum opening of a joint under the maximum hydrostatic pressure from the water in the reservoir. The support

structure is generally composed of several independent layers, whose configuration is a function of the water pressure and of the anticipated movements of the joint.

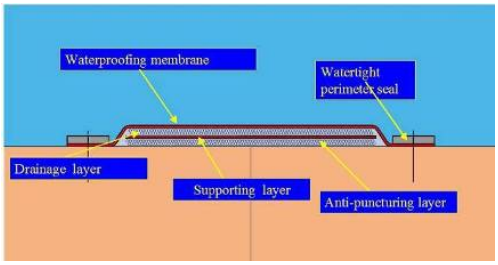


Figure 46. Schematic view of external waterstop.

Different from conventional embedded waterstops, which allow deformation only in the central portion of their bulb, the external waterstop can exhibit large deformation (typically 0.40 to 0.60 m). Therefore, it is capable of accommodating significant movements that may occur in the joint, which would cause failure of conventional embedded waterstops.

When required, a protective layer (anti-puncture layer) and/or a drainage layer can be incorporated in the system (Figure 46). Also, if required, a protective layer can be installed over the geomembrane. Those additional layers have no impact on the effectiveness of the waterproofing system.

5.5.2 *Example of use of external waterstop on an RCC dam*

External waterstops were used at Porce II Dam, a 118 m high RCC dam constructed in 2000 in Colombia (Figure 47). The upstream face is formed by curb extruder (see Section 3.3.5). The contraction joints, at average spacing of 35 m, were waterproofed with the external patented waterstop system.

The final design of the waterstop was verified in full scale testing in a pressure vessel under the simultaneous conditions of a hydrostatic head of 2.4 MPa and an opening of the joint of 35 mm. Ismes, a major engineering institute in Italy, supervised and approved the system (Scuero & Vaschetti 2006).

In detail, the system adopted at Porce II Dam consists of:

- Support structure. Proceeding from the dam body towards the reservoir, support is provided by a rigid part impeding intrusion of the waterproofing liner into the active joint, and by a flexible part providing extra support and low friction, so that the movements of the joint can take place without affecting the waterproofing composite geomembrane. The rigid part consists of two stainless steel plates, each anchored along one edge, and of a strip of Teflon to decrease friction in the area of overlapping, thereby allowing sliding of the plates. The flexible part consists of one layer of 2000 g/m² polyester needle-punched nonwoven geotextile, anchored along one edge and providing anti-puncture protection against the sliding edges of the steel plates, and one layer of composite geomembrane anchored along one edge.
- Waterproofing liner. Composite geomembrane, consisting of a 3.5 mm thick PVC geomembrane heat-coupled during extrusion to a 500 g/m^2 polyester needle-punched nonwoven geotextile. The composite geomembrane is centered on the joint, covering it for a total width of about 0.40 m. The composite geomembrane is watertightly anchored at the periphery by flat stainless steel batten strips compressing it against the curbs, locally leveled out by trimming the offsets and by applying a layer of epoxy resin. Synthetic gaskets distribute stress to achieve even compression. The composite geomembrane is exposed. At plinth level, the composite geomembrane waterproofing the vertical joints is connected directly against the rock.

Figure 47. Detailed view of external waterstop at Porce II RCC Dam contraction joints, and general view of the dam.

5.6 *Repair of RCC dams*

In RCC dams, repair has been so far carried out on cracks, using the external waterstop system described in the preceding section for use in new construction. As for total rehabilitation of dams, repairs can be and have been made in the dry or underwater.

Dona Francisca Dam (Figure 48) is an example of repair in the dry. It is a 50 m high RCC dam with a 0.5 m thick impervious concrete facing. At the end of concrete pouring, before the dam was impounded, cracks appeared in the RCC. Two major cracks developed through almost the whole height of the dam; other three or four cracks were of smaller length.

The two major cracks and one of the smaller cracks were essentially vertical and seemed to be of thermal shrinkage origin. However, the temperatures measured in the dam were lower than the design limit values. The other cracks were inclined and seemed to be of foundation settlement origin. Since the

cracks cause was ultimately not well defined, a geomembrane system was selected because it would have the capability of bridging also future increase in the cracks opening.

The external waterstop was placed on the large and small cracks for a total length of approximately 150 m. Both the support layer and the waterproofing layer are of the same material, a 2.5 mm thick PVC geomembrane heat-coupled during extrusion to a 500 g/m^2 polyester geotextile.

Figure 48. Dona Francisca RCC dam, Brazil, 2000: the external waterstop was installed on the cracks in the dry, before the dam was impounded.

An example of underwater repair of RCC dams is discussed in Section 6.

5.7 *Performance of RCC dams with geomembrane*

RCC dams lined with PVC geomembranes typically show insignificant seepage as compared to RCC dams with conventional concrete facing. At Miel I dam (Marulanda et al. 2003), total leakage from the dam body at fully impounded reservoir is less than 2.5 l/s; the surface area of the Miel Dam face being 31,453 m^2, even if we presume that all leakage comes through the geomembrane, and not as it is generally the case also from the abutments or foundations, the average leakage rate per unit area is less than 0.3 l/hr/m^2. At Balambano Dam, a 95 m high RCC dam, Indonesia, the exposed PVC composite geomembrane installed in 1999, has a total water flow for all six box drains of the upstream face varying from 0.012 l/s to a maximum of 0.965 l/s at full supply level; the surface area of the dam face being 15,490 m^2, the average leakage rate per unit area is about 0.22 l/hr/m^2.

5.8 *Safety in case of geomembrane failure*

As indicated in Section 4.7, it is important in dam design to evaluate the consequences of a failure of the waterproofing element. In the case of RCC dams constructed using the drained geomembrane system, the drainage system is designed so as to ensure that water seeping through defects in the geomembrane or at its peripheral connections is captured by the drainage system and removed before it can percolate into the RCC dam.

In the extreme case of a large breach in the geomembrane, the leakage rate may exceed the capacity of the drainage system (especially if there is no geosynthetic drain such as a geonet behind the geomembrane). In this case, some water may percolate through the RCC dam. However, this will not pose a stability problem. Therefore, even a total rupture of the geomembrane has no impact of safety. The only consequence would be to empty the reservoir and repair the geomembrane, unless repair can be done under water, as discussed in Section 6.

6 UNDERWATER REPAIR OF DAMS

6.1 *The concept of underwater repair*

The underwater installation of a geomembrane system, unlike underwater construction with concrete, does not require a complicated site organization and the use of heavy equipment that may cause significant environmental impact.

The technology for underwater installation has been developed based on experience accumulated by the main specialist contractor in the field of dam waterproofing, and through a research program carried out jointly by this contractor and the US Army Corps of Engineers. This research program is discussed in Section 7.2.2.

Underwater repair can be, and has been, made to restore watertightness to the entire upstream face (as at Lost Creek Arch Dam), to repair cracks (as at Platanovryssi RCC Dam), and to repair most critical areas of the upstream face (as at Turimiquire Concrete Face Rockfill Dam).

6.2 *Examples of underwater repair*

6.2.1 *Example of underwater repair of the entire face*

Lost Creek Dam is a 36 m high arch dam, with a crest length of 134 meters, constructed in 1924, and owned by Oroville-Wyandotte Irrigation District in USA. The deterioration of the dam due to freeze/thaw cycles had resulted in loss of more than 0.30 m of depth of concrete at the downstream side. The upstream tensioned geomembrane option was selected out of evaluation of seven repair options, restricted to three because of environmental concerns related to the reservoir level. The three options considered were a downstream drainage system covered with shotcrete, an RCC buttress, and an upstream exposed geomembrane. The geomembrane system had the lowest cost acquisition price. Key to the decision was the projected service life of the geomembrane (more than 50 years, a durability that was also estimated for the RCC buttress,) and the

lack of maintenance required for the geomembrane, which resulted in a much lower yearly amortization cost. The only ongoing cost with the geomembrane system is to take readings of the water level behind the membrane and the water discharge from the drainage system. The owner projected to save more than $2.8 million over the minimum predicted service life by selecting the geomembrane option (Scuero et al. 2005).

Since heat-welding cannot be made underwater, joining of adjacent composite geomembrane sheets was mechanical, with the patented drained system presented in Section 4.4.5, slightly modified to adapt it to underwater installation. The underwater working environment also required modifying some minor components of the fastening system.

The geomembrane installation was scheduled to coincide with the rewinding of the generator at the powerhouse. This timing allowed a significant drawdown of the reservoir without suffering an additional loss of energy production. Drawdown of the reservoir meant the dive depths were reduced for the underwater installation, allowing longer time underwater per dive and achieving significant economy. The reservoir was lowered to about half the maximum water depth at the dam (Figure 49).

The geomembrane system was installed in 1997. Piezometer readings have consistently indicated that there is no water standing in the geonet drain. The downstream face of the dam appears dry. The owner reports (Harlan & Onken 2003) that leakage "is only 2.3 liters per minute with the reservoir at spillway crest". The total surface area of the dam face being 2,800 m^2, the average leakage rate per unit area is 0.05 $l/hr/m^2$, which is very low compared to other measured leakage rates.

Figure 49. PVC geomembrane panels under installation at Lost Creek Dam, USA, in 1997. In 2003 (right), the dam in service.

6.2.2 *Example of underwater repair of a crack*

Platanovryssi Dam, Greece, is an example of underwater repair of a crack. At 95 m, it is the highest RCC dam in Europe. The dam, of the high cement content type, was designed to be impermeable in its whole RCC mass. The vertical contraction joints, placed at variable spacing not superior to 30 m, were waterproofed during construction of the dam with the external waterstop system described in Section 5.5.2.

On first filling, seepage started increasing, then decreased, and then again increased, attaining a maximum of 30.56 l/s on 10 October 2000. The cause of the seepage was an approximately 20 m long, 25 mm wide crack, extending from the upstream to the downstream face. Due to an unusually dry season, the owner could not afford to lose the volume of the water already stored in the reservoir, and, therefore, could not empty the reservoir in order to work in dry conditions.

In addition to that restriction, when Platanovryssi Reservoir is empty, the pumped storage scheme of Thissavros cannot operate, which has serious implications to the production system. It was considerably more cost effective to do the work underwater. Repair works were scheduled for the spring of 2002.

The system selected by the owner for the underwater installation on the crack in 2002 is the same conceptual system that had been selected and installed on the vertical contraction joints during construction of the dam in 1998; however, the materials have lower thickness due to the lower hydrostatic pressure at the crack. The repair concept is illustrated in Figure 50. The waterproofing system was installed in the dry from crest level at 227.50 m to elevation 225 m, and underwater from elevation 225 m down to elevation 208 m. To facilitate underwater works, a special steel frame was constructed and lowered into the reservoir to serve as template for placement of the perimeter seal, which is the same as installed at Miel I Dam. Works started on 22 April 2002 and were completed on 23 May 2002. The repair work has been extremely successful: the leak through the dam has been fully stopped and the downstream face (Figure 50) dried out a few hours after installation of the geomembrane system (Papadopoulos 2002).

Figure 50. At left artist's impression of the repair of the crack executed underwater at Platanovryssi RCC Dam, Greece 2002. At right, the downstream appearance of the crack before and a few hours after installation of the external waterstop: the leakage stopped and the crack dried out.

6.2.3 *Example of underwater repair of critical areas*

Turimiquire Dam is an example of underwater repair carried out in crucial areas of the upstream face. Designed by Barry Cooke and built from 1976 to 1980,

Turimiquire Dam is a 113 m high concrete face rockfill dam located in Venezuela and used for potable water supply.

Due to interruption of works, delays, and modifications of parts of the original design, the dam since impounding in 1988 suffered severe leakage that was not stopped by the repeated repair measures carried out over the years and consisting basically in backfilling the cracks and cavities of the upstream face with various types of materials.

In 2008, when leakage had attained 9,800 l/s, the owner decided to adopt a geomembrane system to repair the most deteriorated part of the upstream face (Figure 51), on a surface of about 14,500 m^2. (The total surface area of the concrete face being about 55,000 m^2.)

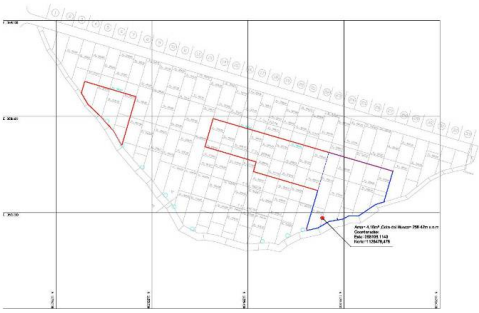


Figure 51. Turimiquire Dam: the critical areas to be waterproofed with the exposed PVC geomembrane system.

A general view can be seen on Figure 52. Installation will be made with the reservoir in operation, partly in the dry and partly underwater. The waterproofing system used for the dry and the underwater parts is similar to the ones already discussed in the previous case histories. The diving depth at Turimiquire Dam will however be much superior, attaining more than 50 meters. Divers will carry out works as the water level varies depending on operation needs. Works have started in the fall of 2009 with cleaning of the slabs, removal of the sediments and placement of the face anchorage profiles (Figure 52).

Figure 52. At left, upstream slope of Turimiquire Dam; at right, the anchorage profiles being lowered into the water at Turimiquire Dam.

7 GEOSYNTHETIC BARRIER MATERIAL SELECTION

7.1 *Geomembrane selection criteria*

Geomembranes used in dams must be designed and produced adequately. Modern high-performance geomembranes are produced with sophisticated design and high-quality components; the manufacturing and quality control procedures and the installation technology are sophisticated; and a much longer durability is expected if compared to the durability of old geomembranes.

Of the two main "families" of geomembranes, polymeric geomembranes and bituminous geomembranes, polymeric geomembranes are by far the most widely used in hydraulic applications worldwide. In particular on dams, polymeric geomembranes account for approximately 91% of the total, while bituminous geomembranes (used mainly in France and in some French-speaking countries) account for approximately 9%, according to the ICOLD data base.

A detailed discussion of the chemical composition and properties of polymeric geomembranes is beyond the scope of this paper. According to the ICOLD database, thermoplastic geomembranes are by far (about 85%) more used than thermoset geomembranes, and, more than 70% of the thermoplastic geomembranes are the most flexible ones. Thus, PVC geomembranes, which are very flexible, account for more than 60% of the total of dams lined with a geomembrane.

One of the key points in designing a geosynthetic barrier system is the selection of the geomembrane. In the first pioneer applications in the 50s, 60s and 70s, a wide variety of geomembranes were used. Then, based on experience acquired in early applications, and developments in research, testing and manufacturing, geomembranes with lower performance have been gradually abandoned in favor of more performing ones.

From the viewpoint of imperviousness, all geomembranes are extremely effective compared to conventional liners, as clay, cement concrete or bituminous concrete. The criteria for selection are different depending on the type of structure to be waterproofed. Hydraulic structures, and dams in particular, require materials more performing than the ones that can be used for example in roofing because hydraulic structures face more demanding service conditions.

Design of the geomembrane shall be made taking into consideration the environmental factors involved such as extreme temperatures and exposition (the ageing of geomembranes being generally quicker in areas facing South in the northern hemisphere and North in the southern hemisphere), and the loads exerted on the geomembrane during service, such as:

- hydrostatic pressure due to water in the reservoir; backpressure due to water table in some cases; impact by floating debris, ice, boats etc.; action of waves and wind;
- concentrated stresses, often related to the applied hydrostatic pressure, causing puncture and burst over irregular subgrade; tensile stresses due to differential settlements in embankments and at connections between deformable embankments and rigid structures; active joints and localized differential deformation that may occur in concrete and RCC dams in case of weakness of foundation or in case of seismic events; etc.; and
- water chemistry, in some cases.

In general, these loads require geomembranes with high performance with respect to elongation, flexibility, and resistance to puncture and burst. Good elongation properties in 3-dimensional loading and, preferably, no yield on the geomembrane tension-strain curve are important properties to ensure that the geomembrane is capable of accommodating high deformations of the subgrade, and repeated stresses exerted for example by waves and wind. The presence of a yield point on the geomembrane tension-strain curve, even if the geomembrane has high elongation at failure, may be crucial in hydraulic applications. High flexibility and elongation capability contribute to high resistance to puncture: the more a geomembrane can adapt to the subgrade, the more the load exerted by water is distributed, thus reducing the risk of damage by puncture due to concentrated loads at protrusions.

To better bridge cavities (i.e. to decrease the risk of rupture by burst) and to better resist rupture in case of concentrated differential movements such as the connections between compressible embankment and rigid structure, a geomembrane should have an optimum combination of strength and deformation capability. This combination is expressed by the co-energy of the geomembrane tension-strain curve (Giroud & Soderman 1995; Giroud 2005). From this viewpoint, an excellent tension-strain curve is that of a composite geomembrane that consists of a geomembrane with high elongation capability and no yield point bonded to a needle-punched nonwoven geotextile that provides strength without significantly reducing elongation. An analysis has been done for the design of a dam where it was assumed that cracks could develop after geomembrane installation (Giroud, to be published). Extensive theoretical developments were required due to the different tension-strain curves considered: linear, bi-linear, and parabolic. The analysis showed that the geomembrane performance was governed by its co-energy. Four geomembranes were compared in the analysis. Numerical calculations were conducted assuming a crack width of 50 mm, based on the geotechnical study of the site. The following factors of safety against rupture in tension were obtained: 1.4 for 1.5 mm LLDPE, 1.6 for both 1.5 mm HDPE and 4 mm bituminous geomembrane, and 5.8 for a composite geomembrane consisting of a 2 mm PVC geomembrane bonded to a 500 g/m^2 polyester needle-punched nonwoven geotextile. This study confirms that the best performance is achieved with a combination of strength and elongation; the analysis also shows that the mechanical behavior of an LLDPE geomembrane is not better than, and in fact is inferior to, the mechanical behavior of an HDPE geomembrane, contrary to a widespread belief; the study finally shows that a variety of geomembranes can be used depending on the required factor of safety.

The behavior of geomembranes in real scale performance testing for adaptation to the subgrade and resistance to puncture is quite different from one type of geomembrane to another. Since experience has shown that an inadequate geomembrane may exhibit premature ageing, and possibly cause failure of the waterproofing system, several agencies and research centers have in the last decade conducted research and issued recommendations on selection of geomembranes for rehabilitation and new construction of hydraulic structures. The following section summarizes some significant findings.

7.2 *Research programs*

7.2.1 *Research program on geomembranes for concrete dams in cold climates*

IREQ is the Research Institute of Hydro Québec, the largest dam owner in Canada. The extremely cold climate in some parts of Canada, in particular frequent wetting-dehydrating and freeze-thaw cycles, high daily and seasonal variations of temperature and of reservoir water level, action of ice, of debris and of acid rains, can heavily deteriorate concrete dams. Since repair with new concrete sooner or later entails facing the same problems, IREQ dedicated intensive research to investigate alternative repair methods and evaluate which are the most dependable ones for concrete dams experiencing extremely severe climatic conditions.

The one-year-long study, presented in a 200-page Final Report (Durand et al. 1995), was executed for the Department for Maintenance of Equipments and Safety of Dams of Hydro-Québec, and was carried out in the laboratories of the Department of Materials' Technology and of the Structural Department of the Ecole Polytechnique de Montréal.

The characteristics estimated by IREQ as mandatory for waterproofing liners in cold climates were:

- be waterproof;
- recover from deformations induced by thermal gradients and hydrostatic heads;

- resist tearing and abrasion due to ice and debris action;
- be resistant to UV and ozone;
- if glued, perfectly adhere to the substrate (cement concrete);
- stretch and recover the initial shape if cracks appear in the concrete behind the liner (crack bridging); and
- have realistic and economical installation requirements.

The study was structured into three tasks: Task 1, review of literature on existing liners; Task 2, a first experimental phase including standardized tests to check the impact of different environmental conditions on mechanical properties of the materials; and Task 3, a second experimental phase consisting of specially designed tests simulating the ice action (adherence) on the geomembranes, and evaluating the shear strengths of the materials selected after the tests of the first experimental phase.

The review of existing liners included several rigid conventional liners: shotcrete, metal sheets and bituminous concrete. Although these liners were theoretically deemed valuable, work done under Task 1 pointed out some drawbacks of these rigid liners, mainly that they are difficult to install, their cost is high, and the resistance of some of them to cold climates is disputable. The research team therefore focused on less conventional solutions, basically geomembranes (prefabricated or sprayed). Due to the large variety of products, it was decided to focus on geomembranes which had already a proven record of satisfactory performance, while being readily available on the market, namely PVC, HDPE, CSPE, CPE, butyl rubber, Styrene-Butadiene-Styrene (SBS) rubber, bituminous geomembranes, and sprayed materials, which, although not to strictly belonging to the family of geomembranes, had found some applications in the past.

Further investigation showed that bitumen-based geomembranes were subject to non-negligible deterioration in the long term. Based on the report, the formation of small cracks and the growing of bacteria affect the permeability and resistance of these geomembranes. Following the investigations conducted under Task 1 and discussions with manufacturers or suppliers, eight products were selected for the two experimental phases of the study:

- four prefabricated geomembranes: PVC-A (unsupported plain PVC geomembrane), PVC-B (PVC geomembrane laminated to a nonwoven geotextile), HDPE, SBS; and
- four sprayed geomembranes, mixed and sprayed by the manufacturers concerned and then supplied in form of a sheet: Polyurethane-A, Polyurethane-B, Methacrylate and Neoprene.

In the first experimental phase (Task 2), samples of each geomembrane were exposed, for a predetermined period of time, to what were considered the most severe types of exposure, namely:

- freeze and thaw cycles;
- ultraviolet radiations; and
- low temperatures.

At the end of the exposure periods, samples were tested to ascertain how exposure had affected the following main properties:

- tensile strength;
- puncture strength; and
- tear strength.

Results of the tests showed that freeze-thaw cycles did not significantly affect most prefabricated geomembranes. However, HDPE, Polyurethane-B and, to a greater degree, Neoprene showed slight decrease in tensile strength, Consequently, Neoprene was judged unsuitable for practical applications. Freeze-thaw cycles on the contrary greatly affected the adherence of sprayed or glued products, whose tear strength was significantly reduced. UV exposure did not cause significant variations in the behavior of geomembranes after different exposure times: the conclusions drawn were that the treatment applied to geomembranes to prevent UV deterioration had been effective.

Low temperatures had a different impact on different geomembranes. Details can be found in the Final Report of the study (Durand et al. 1995), and in an abridged English version that has been published under the aegis of ICOLD (Durand et al. 1998).

The first testing phase indicated that the four best performing products to be selected for the second testing phase (Task 3) were: PVC-B, Polyurethane-A, Polyurethane-B, and Methacrylate (this last product was not tested in Task 3 because the manufacturer failed to deliver samples on time). A test assembly was designed and constructed to simulate ice action, more specifically to ascertain adherence of ice on the geomembranes and to determine the shear strength. At the end, the PVC-B (i.e. PVC geomembrane with backing geotextile) and Polyurethane-A (a sprayed material) geomembranes showed the best properties.

IREQ then evaluated two further aspects considered crucial for application on dams: the possibility of having a drainage layer behind the geomembrane, and previous successful experience. Based on these crucial aspects, geomembrane PVC-B was finally deemed by IREQ more adequate than Polyurethane-A, because it could fulfill both requirements, while Polyurethane-A could not.

7.2.2 *Research program on geomembranes for hydraulic structures*

The US Army Corps of Engineers (USACE) operates and maintains a large number of dams and major hydraulic structures, often made of concrete. The USACE is constantly searching for new concepts in maintenance and repair of concrete structures, since traditional repair materials such as cement concrete, shotcrete, bituminous concrete, resins, and steel plates have presented many limitations, especially when underwater installation is at stake.

With an approach similar to that of IREQ (see Section 7.2.1), a two-phase research program was carried out by the Army Corps of Engineers Waterways Experiment Station in Vicksburg, USA, in 1995 and 1996.

The first part of the research carried out in 1995 focused on geomembrane liners. The characteristics that the Corps deemed crucial for a reliable geomembrane liner were, in order of decreasing importance:

- very low permeability;
- high resistance to tensile stress, pressure, puncture;
- elastic behavior, with high-percentage elastic elongation;
- high resistance to the service environment;
- ease of junctions;
- satisfactory performance in previous applications;
- repairability;
- acceptable cost; and
- availability.

Since successful previous application was one of the requirements, the Corps considered only the geomembranes that had already proven to be suitable for hydraulic structures, namely PVC, PVC-R (with backing reinforcement), CSPE-S (with scrim reinforcement), CSPE, PP, PP-R (with backing reinforcement), EPDM, and HDPE. A total of 21 of these geomembranes having various thicknesses were tested under two types of tests: standardized uni-axial tests on small specimens, and large scale, non-standardized multi-axial tests. These latter tests, internationally considered more adequate to better evaluate resistance and flexibility characteristics, focused on resistance to puncture and burst, on homogeneity and isotropy of the material (necessary to ensure that the geomembrane will not fail on irregularities of the substrate under the hydraulic head), on capability to conform to the substrate and on elastic recovery (essential to reduce the stresses on the anchorage system). The results are listed in the Technical Report REMR-CS-50 (Christensen et al. 1995).

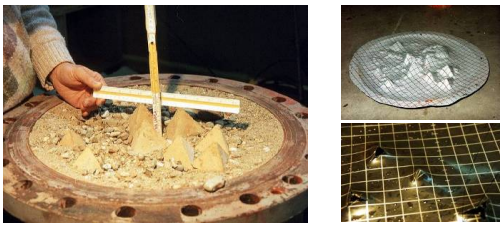


Figure 53. Real scale testing for puncture and burst resistance carried out by the US Army Corps of Engineers on 21 geomembranes confirmed the excellent performance of PVC in respect to all other types of geomembranes. PVC composite geomembrane (2.5 mm thickness, upper photo) conformed to the substrate and resisted to repeated cycles of loading-downloading at 1 MPa (100 m). HDPE (2.5 mm thickness, lower photo) did not conform and ruptured at 0.35 MPa at the first loading (*Courtesy of the US Army Corps of Engineers – Waterways Experiment Station*).

Results of the puncture test (Figure 53) showed that

- PVC geomembranes, having a low modulus of elasticity, were flexible and elastic, better conformed to the substrate, and resisted repeated loading cycles at water heads superior to 1 MPa (100 m);
- HDPE geomembranes, having a high modulus of elasticity, were rigid, did not conform to the substrate, and broke at low water heads, in the 0.15 to 0.35 MPa (15 to 35 m) range; and
- PP geomembranes, having medium to high modulus of elasticity, failed at 1 MPa, showing excessive permanent deformation.

A test for resistance to burst (Figure 54) was a destructive test intended to assess the elongation at break and the mode of failure. Results of this test showed that:

- PVC geomembranes are capable to deform more, which reduces stresses in the geomembrane, and break in a star-like mode, which indicates that the geomembrane is isotropic, i.e. with no preferential failure direction;
- HDPE geomembranes deform less, and, as a result, stresses are higher; furthermore, the geomembrane breaks in a linear mode, indicating that the geomembrane has a preferential failure direction; and
- PP geomembranes are more capable to deform than HDPE, but deformation is not isotropic, indicating a potentially weaker direction.

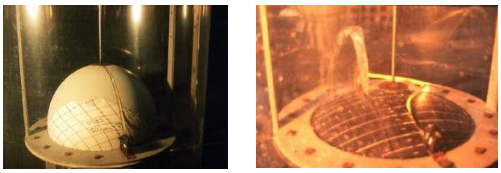


Figure 54. Testing burst resistance. A composite geomembrane (3 mm PVC geomembrane bonded to a 700 g/m^2 geotextile, left) resisted 160 kPa, with a 271 % surface elongation (i.e. increase in surface area). An HDPE geomembrane (right) failed at lower elongation, along a line indicating a zone of lower strength, and a preferential path for propagation of tear *(Courtesy of the US Army Corps of Engineers – Waterways Experiment Station).*

Table 3 in the USACE report summarizes the test results and the evaluation of the 21 geomembranes after testing for puncture and burst. Testing evaluated the conformability to substrate, failures at protrusions and depressions, elastic recovery, homogeneity and isotropy, and capability to deform. The maximum total available score was 35. Some of the typical scores obtained were as follows:

- PVC geomembrane: 34
- Composite geomembrane (PVC bonded to needle-punched nonwoven geotextile): 35
- EPDM geomembrane: 32
- CSPE geomembrane (scrim reinforced): 28
- PP geomembrane: 22
- HDPE geomembrane: 6

To complete the evaluation, all other relevant characteristics of the geomembranes (imperviousness, tear resistance, dimensional stability, seamability, etc.), as well as other parameters considered important by the Corps to select an adequate material (constructability, previous applications, durability, availability, repairability, and costs) were investigated. The following list, derived from Table 4 in the USACE report, presents the main results of the overall geomembrane' evaluation where the maximum possible score was 260:

- PVC geomembrane: 235
- PVC composite geomembrane: 256
- CSPE geomembrane (scrim reinforced): 220
- EPDM geomembrane: 209
- PP geomembrane: 189
- HDPE geomembrane: 140.

Field evidence confirms laboratory results. PVC geomembranes have been successfully used in a very large number of dams and hydraulic structures where their performance benefits from their properties. Their low modulus of elasticity, hence their high flexibility, makes them more performing with respect to puncture and burst. Furthermore, the weathering resistance of properly formulated PVC geomembranes has been proven by decades of field experience in exposed applications (first installation in 1960). In addition, they are easy to install, in particular due to their flexibility. On the contrary, geomembranes with a high modulus of elasticity are stiff and less performing in the field with respect to puncture and burst by irregular rough substrates. This is confirmed by the statistics on the use of different types of geomembranes in dams. The following table lists geomembranes in dams by type of geomembrane, and if exposed or covered, according to the ICOLD database.

Table 3. Geomembranes (GM) in dams.

GM	PVC	LLDPE	Bituminous		HDPE
Expos.	92	0	7		3
Cover.	77	29	11		12
	Elastom.	CSPE	PP	CPE-R	Other
Exp.	5	2	3	0	0
Cov.	5	5	3	3	3

Composite geomembranes, which consist of a geomembrane and of a geotextile coupled to it during fabrication, are the most widely adopted type of geomembrane in dams. The geomembrane component provides imperviousness; the geotextile component provides anti-puncture protection reducing the need for surface preparation, higher dimensional stability, higher friction angle allowing self-stability on inclined surfaces, and some drainage capacity. A composite geomembrane is more performing than a geomembrane with a separate geotextile, because it has higher friction angle and it better transfers the stresses to the anchoring system and substrate.

The thickness of a geomembrane to be used in a project is selected depending on the water head, the roughness of the subgrade, the extent of exposure to UV, the required service life, the survivability at installation, etc. The ICOLD database reports typical thickness varying for bituminous geomembranes from 4 to 5 mm, and for polymeric geomembranes from 1 to 3 mm, exceptionally 3.5 mm (HDPE) and 4 mm (CSPE). In a non negligible number (60) of dams, the geomembrane thickness is inferior to 1 mm (some in the range of 0.25-0.35 mm): these cases refer mostly to small dams, or to old dams, or to dams constructed in areas where performance criteria are different from the criteria used in industrial and populated areas.

In practice however, in modern dam projects the recommended minimum thickness values, for example for a PVC geomembrane, are generally 2 to 2.5 mm. The choice of using a thicker geomembrane is based on requirements about durability, which for an exposed geomembrane is related to its thickness. Testing has shown that, for the same formulation of a PVC geomembrane, doubling the thickness increases the expected lifetime by a factor of approximately four. In dam projects, the cost of supply of the geomembrane is normally only 10 to 20 % of the

cost of the entire waterproofing system, which is normally only 3 to 5 % of the cost of the entire dam project. Thus, it is financially prudent to increase the thickness of the geomembrane, because the cost increase due to a thicker geomembrane is a very small portion of the project cost, and the service life is significantly increased.

7.3 *Selection criteria for the associated geosynthetics*

The design of the use of the various geosynthetics associated with geomembranes in the applications described in this paper would require another paper as long as this one. Only some general comments are made in this section.

The geosynthetics associated to the geomembrane in a geosynthetic barrier system are selected based on the function to be provided. Numerous design methods are available in the geosynthetic literature for a number of applications relevant to liner systems such as: drainage design, cavity bridging design, stability of liner systems on slopes, uplift of geomembrane by wind, geotextile filter criteria, design of geotextile as protective layer, etc. In all cases, it is important to obtain adequate property values from the manufacturers. Some comments on geosynthetic selection are made below.

When the subgrade presents large/deep cavities, like in dams with masonry facings or in concrete face rockfill dams that have undergone large settlements or deterioration, geogrids can be used to support the impervious liner.

Geonets are used when an additional drainage layer is required. If the surface being lined is concrete, the geonet is placed directly over it; if the surface is soil or in general loose material, a drainage geocomposite (geonet bonded to geotextile) is used, to prevent the loose particles of the subgrade from clogging the geonet. Drainage materials consisting of coarse filaments or cuspated or otherwise deformed plastic plates are generally not recommended because their compressive strength is often low and, as a result, their hydraulic transmissivity can be significantly reduced by the high hydrostatic pressures that exist in most dams.

Geotextiles used either as a separate anti-puncture layer under the geomembrane, or associated to the geomembrane, are typically made of polyester or polypropylene fibers. Polyester has high strength, but it deteriorates in alkaline environment. Therefore, polypropylene must be used when contact with fresh concrete is foreseen, as in construction of new RCC dams, or when shotcrete is used. The same applies to geotextiles used as a protection when a cover layer is placed over a geomembrane: the geotextile must be made of polypropylene fibers when the cover layer is cast-in-place concrete.

8 BEHAVIOR OF GEOMEMBRANES AS A FUNCTION OF TIME

8.1 *Durability of geomembranes in dams*

Durability is based on the weathering properties, and on the resistance of the geomembrane to specific loads during service (extreme temperatures, frost, freeze/thaw, ice, impacts by floating debris and boats, wind and waves, fauna and flora, vandalism etc.).

Taken for granted that not all geomembranes have the same behavior due to their chemistry, basic ingredients and manufacturing process, it is important for dam projects to select an existing geomembrane or to design a new geomembrane that can best perform according to the type of environment in which it will be used, and that can provide adequate durability for the required application.

Standard accelerated ageing tests are available and are being used all over the world to predict the behavior of geomembranes in service. These tests, although accelerated, would however require too long a time to give indication of long term behavior.

The most practical way to ascertain if a geomembrane will be resistant in the long term to the environmental loading expected in a dam project, is to exhume samples of the same geomembrane that have already been in service, in a similar environment and project, for a period of time that should be as long as possible, ideally as long as the required service life of the geomembrane in the considered dam, and perform tests to determine to which extent their properties have changed. Testing of the physical and mechanical properties of the exhumed samples indicates if the geomembrane properties are at the time of the test are within acceptable limits, and extrapolation allows the determination of the expected remaining service life.

This approach has been adopted in Italy (Cazzuffi 1987 and 1996), using data from several dams rehabilitated with exposed PVC composite geomembrane. The oldest reported application of exposed PVC geomembranes on dams dates back to the mid 1970s, and since then many dams have been rehabilitated with exposed geomembranes of the same type, which makes it possible to obtain dependable results. Furthermore, many of these dams are at high elevation (typically greater than 2000 m) where UV radiation is intensive and the weather conditions are harsh. From this database, it has been concluded that the service life of PVC composite geomembranes in such harsh environment is superior to 50 years (Hsuan et al. 2008). It should be noted that geomembranes of the same type installed today are of better quality than the geomembranes installed 30 years ago and included in the data base used to predict durability. Therefore, the durability of PVC

composite geomembranes installed today may be even greater than the predicted durability of at least 50 years.

The approach just described for PVC geomembranes should be used with other geomembranes to evaluate if they have the appropriate durability for use in dams.

8.2 *Leakage rates*

Typically, leakage from a geomembrane system on a dam is very low, in the order of a few l/s for surfaces in the range of 10,000 to 30,000 m^2. Figure 55 compares leakage from roller compacted concrete dams with (the line just above zero) and without upstream geomembrane (the five lines on top of the bottom one). From Figure 55, it is clear that the rate of seepage is significantly less through RCC dams with geomembrane than without geomembrane.

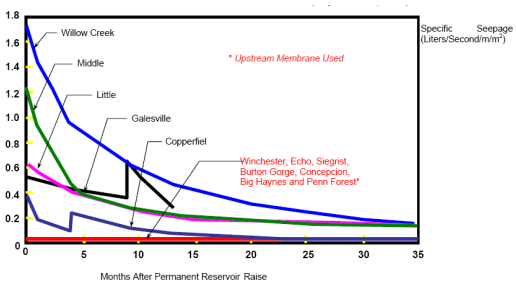


Figure 55. Leakage in roller compacted concrete dams with and without upstream geomembrane (*Courtesy Ernie Schrader*).

The rate of leakage depends also on the design and on the goals that the owner/designer intend to achieve. There have been cases in which partial repair has been carried out, accepting a certain amount of leakage based on the benefit/cost ratio. An example is Strawberry Dam, a concrete face rockfill dam in the USA: after that six out of a total of nine joints had been repaired with an exposed PVC geomembrane, the owner has postponed the repair of the remaining three joints because the leakage rate has already been lowered to acceptable values.

Lack of treatment at boundaries due to site-specific or financial reasons may also lead to accept higher leakage rates.

9 CONCLUSIONS

The range of possible applications of geomembranes as water barriers in dams is quite wide. Geomembranes can be applied to all types of dams, in new construction and in rehabilitation, in the dry and underwater.

Design and installation systems of the various components of geomembrane systems according to the type of application have been discussed in this paper.

Geomembrane selection and behavior of geomembrane systems vs. time have been investigated based on various research projects, and on results from laboratory tests and tests on exhumed samples.

Data on the performance of dams constructed or rehabilitated using geomembranes have been provided. These data show the excellent performance of geomembranes in dams of all types.

This paper shows clearly that the use of geomembranes in dams is now a well established technique.

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