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Design of canals lined with geomembranes

Jean-Pierre Giroud[*1], *Hervé* Plusquellec[2] and *Eric* Blond[3]

[1]Consulting Engineer, Paris, France
[2]Consulting Engineer, Washington, USA
[3]Consulting Engineer, Montreal, Canada

Abstract. First, a review of important issues that must be addressed when designing the use of geomembrane linings in canals is presented: water flow characteristics affecting exposed geomembranes and mechanisms that could cause failure of geomembrane linings. The reviewed mechanisms include: deformation of the supporting soil; uplift of geomembrane by wind, flowing water, or groundwater; mechanical damage to the geomembrane lining, e.g. by puncture or excessive stresses Recommendations are presented to address these issues. Then, design methods are presented for the sizing of canal components such as geomembrane protection, geomembrane ballasting, and geomembrane anchorage. It is shown that, in many cases, the association of geomembrane watertight lining and concrete protection layer is a successful solution.

1 Introduction

1.1 Continuity from a preceding paper at a GeoAfrica Conference

A general introduction to the use of geosynthetics, in particular geomembranes, in canals can be found in a paper by Giroud and Plusquellec (2017) [1] presented at the Third African Regional Conference on Geosynthetics, in Marrakech, Morocco. In the present paper, the focus is on the design of geomembrane linings in canals. It is assumed herein that readers are already familiar with geomembranes and other geosynthetics.

1.2 Leakage/seepage control

The term ‘leakage/seepage’ is used in the present paper as a generic term to encompass the following mechanisms of liquid migration: migration (i.e. seepage) through the pores of a permeable medium (e.g. the soil underlying a canal, lined or unlined, and a clay lining); and migration (i.e. leakage) through discrete passageways or openings (e.g. cracks in concrete, defective joints between concrete panels, holes in geomembranes, and inadequate connection between a lining and an appurtenant structure).

Leakage/seepage control is the primary function of a canal lining. However, this subject is not addressed in the present paper because it has been extensively discussed specifically

* Corresponding author: jpg@jpgiroud.com

for canals in other publications, in particular Giroud and Plusquellec (2017) [1] and Giroud (2022) [2]. The focus of the present paper is on aspects of design that are essential to ensure that canal linings, in particular geomembrane linings, can safely perform the leakage/seepage control function. To that end, the present paper addresses issues such as: prevention of geomembrane lining failure, stability of linings, evaluation of stresses on geomembrane linings, protection of geomembranes, etc.

1.3 Materials used for lining canals

Canals are unlined, or lined with a variety of materials such as concrete, clay, bricks, bituminous concrete, and geosynthetics (including geomembranes). A canal lining is intended to perform one or several functions, including: leakage/seepage control; protection of the canal banks and bottom against erosion and other forms of deterioration; smooth surface to foster rapid flow of water; control of vegetation; and hard surface to facilitate cleaning and maintenance.

Concrete linings can perform (to a certain degree) the above-mentioned functions, at least before they deteriorate, which happens more or less rapidly, e.g. a few years to more than 50 years in case of high quality of construction, but much less in many instances. As a result, concrete linings have been routinely used before the advent of geomembranes and other geosynthetics, and are still often used.

Geomembranes are the material of choice when leakage/seepage control is a priority. However, in many cases, geomembrane linings need to be covered for protection and/or ballasting. Various materials can be used to cover geomembrane, for example: concrete, soil or stone layer, and a variety of geosynthetics. A geomembrane that is not covered is said to be exposed.

In addition to geomembranes, a variety of geosynthetics are used in canals:

- Bentonite geocomposites, also called geosynthetic clay liners (GCLs), and concrete geocomposites, also called geosynthetics cementitious composite mats (GCCMs), are used for leakage/seepage control.
- Concrete-filled geomattresses are typically used for slope protection, but may also be used to provide some degree of leakage/seepage control.
- Sand-filled geomattresses are used for slope protection.
- Soil-filled or concrete-filled geocells are typically used for slope protection, but concrete-filled geocells may also be used to provide some degree of leakage/seepage control.
- Geomats and geotextile-supported articulated concrete blocks are used for slope protection.

The present paper is devoted to geomembranes, and more specifically to the design of geomembrane linings for canals. Other materials are mentioned when they are associated with geomembrane linings.

1.4 Complementarity of concrete and geomembrane linings

In many instances, concrete linings and geomembrane linings are the two options considered for lining a canal. While geomembranes can be damaged by external actions (e.g. falling objects, boats, animals, hail, operation and maintenance activities), concrete linings are robust. On the other hand, geomembrane linings are far more watertight than concrete linings and, being flexible and extensible, they are not affected by systematic cracking as concrete linings do. A detailed comparison of the watertightness of geomembrane and concrete linings has been published by Giroud and Plusquellec (2023a) [3].

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The watertightness of geomembranes and the robustness of concrete are complementary properties. As shown in the present paper, linings that associate geomembrane watertightness and concrete robustness are often the best solution to achieve effective leakage/seepage control and long-term performance.

1.5 Organization of the present paper

The present paper is organized as follows:
1. Introduction
2. Flow characteristics affecting canal linings
3. Possible failure mechanisms
4. Design of canal characteristics and lining components
5. Conclusions

The present paper addresses many design issues, but not all design issues. For example, geomembrane selection, which is an important design issue (considering the significant differences between the available geomembranes in mechanical properties and ease of installation, in particular), is not addressed in the present paper which is focused on design methods. For the same reason, geomembrane lining installation is not discussed in the present paper. However, it is recognized that many aspects of geomembrane lining installation have an impact on geomembrane selection and on design.

2 Flow characteristics affecting canal linings

2.1 Relationship between flow velocity and lining characteristics

2.1.1 Manning's equation

Several relationships have been proposed between the flow velocity and the canal characteristics. Only the Manning's equation is mentioned herein:

$$v_{avrg} = \frac{R_H^{2/3}\, S^{1/2}}{n_M} \tag{1}$$

where v_{avrg} is the average flow velocity, R_H is the hydraulic radius of the canal, S is the longitudinal slope of the canal, n_M is the Manning's roughness coefficient. It is important to note that the velocity thus calculated is the average flow velocity. The actual velocity of water is typically lower than the average value in the vicinity of the bottom and the side slopes of the canal and higher than average in the upper central part of the water cross section. Also, the water velocity may significantly vary locally in case of turbulence generated downstream of obstacles such as weirs, gates, bridge piers, in irrigation canals, and turbulence generated by boat propellers in navigation canals.

The Manning's roughness coefficient is generally presented as dimensionless, but it is not. Its SI unit is s $m^{-1/3}$. If the Manning's roughness coefficient is considered dimensionless, the above equation must be used with the following SI units: v_{avrg} (m/s) and R_H (m); S is dimensionless.

The hydraulic radius is defined by the following equation:

$$R_H = \frac{A_w}{P_w} \tag{2}$$

where A_w is the cross-section area of water in the canal, and P_w is the wetted perimeter, i.e. the length of lining in contact with water.

In the case of a trapezoidal canal:

$$A_w = d_w \left(B_{invert} + \frac{d_w}{\tan \beta} \right) \tag{3}$$

$$P_w = B_{invert} + \frac{2\, d_w}{\sin \beta} \tag{4}$$

where d_w is the depth of water in the canal, B_{invert} is the width of the canal invert (i.e. the canal bottom), and β is the canal side-slope angle.

Hence, the following expression for the hydraulic radius in the case of a trapezoidal canal:

$$R_H = \frac{A_w}{P_w} = d_w \left(\frac{B_{invert} \sin \beta + d_w \cos \beta}{B_{invert} \sin \beta + 2\, d_w} \right) \tag{5}$$

The flow rate, Q, is defined and expressed as follows:

$$Q = v_{avrg} A_w = \frac{A_w R_H^{2/3} S^{1/2}}{n_M} = \frac{A_w^{5/3} S^{1/2}}{P_w^{2/3} n_M} \tag{6}$$

2.1.2 Manning's coefficient values for geomembranes

If a geomembrane is exposed (i.e. not covered by a protective layer), the following values of the Manning's coefficient, n_M, can be considered for geomembranes that are perfectly flat and clean: from 0.011 for a smooth geomembrane to 0.013 for a rough geomembrane. These ideal values of roughness coefficients (e.g. Manning's coefficient, n_M) may not be representative of the actual situation, especially after years in service.

Some geomembranes do not lay flat because they exhibit wrinkles due to thermal expansion. Most geomembranes have a large coefficient of thermal expansion and wrinkles develop in geomembranes that have a high bending modulus and a low interface friction angle with the underlying material, such as HDPE geomembranes with a smooth lower face. This was demonstrated theoretically by Giroud and Morel (1992) [4] and confirmed by observations in the field. Research is needed to provide data on the roughness coefficient of wrinkled geomembranes. In the meantime, the following method is proposed: if the geomembrane exhibits repeated wrinkles in the canal transverse direction, the Manning's coefficient value for the flat geomembrane should be increased by a significant value, such as 0.010 (for example, $n_M = 0.021$ can be used for a smooth geomembrane with repeated wrinkles in the transverse direction).

The values of Manning's coefficient generally cited for geomembranes are based on the assumption that the geomembrane surface is free from materials that would slow down the flow of water (e.g. debris, sediments, vegetation, etc.). If vegetation grows in a canal, the Manning's coefficient may be high, such as 0.030-0.100. As a result, the flow rate is significantly reduced.

2.1.3 Manning's coefficient values for cover materials

If a geomembrane is covered, the Manning's coefficient for the cover material should be used. Values are readily available in the literature, in particular in Chow (1959) [5]. Examples are: concrete in very good condition, 0.013; shotcrete, 0.016-0.025; gravel, 0.020-0.030; cobbles, 0.030-0.045; rip-rap, 0.050 or more; concrete geocomposite, 0.011-0.015; and geomattresses, 0.015-0.030.

The following equation derived from the original equation proposed by Strickler (1923) [6] can be used to calculate the Manning's coefficient of narrowly-graded particulate materials (sand, gravel, cobbles, stones, rocks) as a function of the particle diameter, D, assuming an equivalent spherical shape:

$$n_M = 0.015\, D^{1/6} \text{ with } D \text{ in mm} \tag{7}$$

Manning's coefficient values calculated using Equation 7 are presented in Table 1.

Table 1. Manning's coefficient values for the soil cover as a function of particle diameter.

D (mm)	0.2	0.4	10	30	50	100	300
n_M (s m$^{-1/3}$)	0.011	0.013	0.022	0.026	0.029	0.032	0.039

The following comments can be made on Table 1:

- The Manning's coefficient values calculated for 0.2 and 0.4 mm particles are consistent with the values typically measured on geomembranes (0.011 to 0.013) and indicated at the beginning of this section. This may be a coincidence, or one may consider that 0.2 to 0.4 mm are an indication of the surface condition of smooth to rough geomembranes.
- The Manning's coefficient values calculated for 10 to 30 mm particles are consistent with the values mentioned earlier in this section for gravel. However, it seems that Equation 7 underestimates the Manning's coefficient for 50 to 300 mm stones.

The Manning's coefficient values presented above should only be considered as indicative for preliminary design. Values specific to the actual lining of a canal project should be used in the actual design.

2.2 Drag stress exerted by flowing water

2.2.1 General expression of drag stress

Any lining is subjected, on its upper face, to a normal stress due to the static pressure of water which is expressed by the following equation:

$$p_{stat} = \rho_w \, g \, h_w \tag{8}$$

where ρ_w is the water density, g is the gravitational acceleration, and h_w is the depth of water above the considered point

The upper face of the lining is also subjected to a shear stress (often called drag stress or tractive stress) due to the flowing water. This shear stress varies from zero at the level of the water surface to its maximum at the bottom of the canal. In the case of a canal with a trapezoidal cross section, the drag stress at the bottom of the canal can be expressed by the following equation:

$$\tau_{bottom} = \lambda \, \rho_w \, g \; d_w \sin\alpha \approx \lambda \, \rho_w \, g \; d_w \tan\alpha = \lambda \, \rho_w \, g \; d_w \, S \tag{9}$$

where λ is a dimensionless factor, d_w is the depth of water in the canal, α is the longitudinal slope angle of the canal, and S is the canal longitudinal slope, with $S = \tan\alpha$.

In the case of a trapezoidal canal, the dimensionless factor, λ, can be expressed as follows:

$$\lambda \approx \frac{B_{invert} \sin\beta + d_w \cos\beta}{B_{invert} \sin\beta + d_w} \tag{10}$$

Equation 10 is only approximate because it was established assuming that the shear stress varies linearly from 0 at the water level to τ_{bottom} at the toe of the side slope. The value of λ is between 0.9 and 1.0 for most typical canal cross sections. Therefore, the following equation is often used:

$$\tau_{bottom} \approx \tau \approx \rho_w \, g \; d_w \, S \tag{11}$$

The force, calculated as the integral of shear stress over the considered surface area of the geomembrane, is designated as 'drag force' or 'tractive force'. This force tends to displace the geomembrane in the downstream direction, but it is counteracted by the resisting force acting beneath the geomembrane, as discussed hereafter in Section 2.2.5.

2.2.2 Comment on the drag stress

The following assumptions were made for the development of Equations 9 and 11: (1) The longitudinal slope is assumed to be constant over a long reach of canal. (2) There is no water loss and the flow rate is assumed to be constant in the considered reach. (3) The canal lining is assumed to be motionless for any appropriate reason (own weight, friction with underlying material, anchorage, etc.). (4) It is assumed that there are no obstacles across the flow in the considered reach of canal. (5) The surface condition (e.g. roughness) of the lining is assumed to be constant over the considered reach of the canal, in particular the lining is assumed to exhibit no localized irregularities such as steps or wrinkles. As a result of these assumptions, the flow velocity is constant along the considered reach. Therefore, there is no acceleration and the only two forces are balanced: the weight of the water in the canal and the resistance opposed by the lining. Therefore, the tractive force (also called drag force) results only from the weight of the water.

It may appear that the drag stress, expressed by Equations 9 and 11, and, therefore, the drag force, do not depend on the flow velocity and the roughness of the lining. In fact, the flow velocity and the roughness of the lining are hidden parameters, as shown in the following three-step demonstration: (1) Equation 1 shows that the hydraulic radius is increased in the following two cases: (i) if the flow velocity is increased for a given lining roughness; and (ii) if the lining roughness is increased for a given flow velocity. (2) Then, Equation 5 shows that the water depth, d_w, is increased if the hydraulic radius is increased. (3) Then, Equation 9 shows that the drag stress is increased if the water depth, d_w, is increased. This three-step demonstration shows that the drag stress (and, therefore, the drag force) increases if the flow velocity and/or the roughness of the lining increase.

Another demonstration consists in combining Equations 1, 5, 9 and 10 and simplifying, which gives the following equation:

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$$\tau_{bottom} \approx \frac{2+\left(B_{invert}/d_w\right)\sin\beta}{1+\left(B_{invert}/d_w\right)\sin\beta}\rho_w\, g\, n_M^{3/2}\, v^{3/2}\, S^{1/4} \tag{12}$$

This equation shows that the drag stress increases if the roughness, n_M, and the flow velocity, v, increase, which confirms the above three-step demonstration.

2.2.3 Impact of canal bend on drag stress

The drag stress (in particular at the canal bottom, τ_{bottom}) should be increased in a canal bend by a factor K_{bend}, which depends on the radius of curvature of the canal centerline. Values of K_{bend} are typically given by the following relationships:

$$K_{bend} = 2 \text{ if } R_c/W_w \text{ is lower than 2} \tag{13}$$

$$K_{bend} = 1.05 \text{ if } R_c/W_w \text{ is higher than 10} \tag{14}$$

$$K_{bend} = 1.0 \text{ for a straight canal} \tag{15}$$

where R_c is the radius of curvature of the canal centerline and W_w is the width of the canal at water level in normal operation. For other values of R_c/W_w, K_b can be interpolated.

2.2.4 Relationship between drag stress and flow velocity

Eliminating the longitudinal slope, S, between Equations 1 and 11 gives:

$$\tau \approx \frac{\rho_w\, g\, n_M^2\, v^2}{d_w^{1/3}} \tag{16}$$

In the case of a granular material such as gravel, n_M is approximately 0.025, and Equation 16 becomes:

$$\tau \approx \frac{6v^2}{d_w^{1/3}} \text{ with } \tau(\text{Pa}), d_w(\text{m}) \text{ and } v(\text{m/s}) \tag{17}$$

Hence:

$$v \approx 0.4\sqrt{\tau\, d_w^{1/3}} \text{ with } v(\text{m/s}),\ \tau(\text{Pa}), \text{ and } d_w(\text{m}) \tag{18}$$

In the case of smooth concrete, n_M is approximately 0.014, and Equation 16 becomes:

$$\tau \approx \frac{2v^2}{d_w^{1/3}} \text{ with } \tau(\text{Pa}), d_w(\text{m}) \text{ and } v(\text{m/s}) \tag{19}$$

Hence:

$$v \approx 0.7 \sqrt{\tau \, d_w^{1/3}} \; \text{with} \, v\left(\text{m/s}\right), \, \tau\left(\text{Pa}\right), \, \text{and} \, d_w\left(\text{m}\right) \tag{20}$$

In the case of rough concrete, n_M is approximately 0.0175, and Equation 16 becomes:

$$\tau \approx \frac{3 v^2}{d_w^{1/3}} \; \text{with} \, \tau\left(\text{Pa}\right), d_w\left(\text{m}\right) \; \text{and} \; v\left(\text{m/s}\right) \tag{21}$$

Hence:

$$v \approx 0.6 \sqrt{\tau \, d_w^{1/3}} \; \text{with} \, v\left(\text{m/s}\right), \, \tau\left(\text{Pa}\right), \, \text{and} \, d_w\left(\text{m}\right) \tag{22}$$

2.2.5 Resistance to drag stress

Beneath a geomembrane in contact with the underlying material at the bottom of a canal, the stresses are as follows: the normal stress is practically the same as above the geomembrane (Equation 8 with $h_w = d_w$) since the weight of the geomembrane is negligible compared to the weight of water above the geomembrane; and the shear resistance at the bottom of the canal is expressed by the following equation:

$$\tau_{resist} = \rho_w \, g \; d_w \tan \delta \tag{23}$$

where δ is the interface friction angle between the geomembrane and the underlying material.
The shear resistance beneath the geomembrane, given by Equation 23, is greater than the shear stress above the geomembrane, given by Equation 11, because $\tan\delta$, which is of the order of 0.2 to 0.5, is far superior to S (with $S = \tan\alpha$, where α is the canal longitudinal slope angle), S being typically of the order of 0.0001 to 0.001. (In other words, the interface friction angle, δ, which is of the order of 10 to 30°, is much greater than the canal longitudinal slope angle, α, which is of the order of 0.006 to 0.06°). Therefore, if the geomembrane is flat and in contact with the underlying material, and for the range of interface friction angles and slopes, S, considered above as typical, the drag forces are not sufficient to displace the geomembrane. However, in a chute or a spillway with a steep slope, the drag force applied by water on an exposed geomembrane may exceed the resisting force beneath the geomembrane, but exposed geomembranes are not used in chutes or spillways without specially designed concrete protection.

2.3 Action of flowing water on geomembrane irregularities

2.3.1 Purpose of this section

The estimate of the shear stress acting on an exposed geomembrane lining, given by Equation 11, is based on the assumption that the geomembrane is flat. This is an ideal case. In reality, there are two situations where the geomembrane departs from a flat condition: some geomembranes exhibit wrinkles, and any geomembrane can be uplifted, as explained in Sections 3.3 to 3.6. These two geomembrane irregularities can be considered to act as obstacles across the flow of water. As explained hereafter in Section 2.3.3, cross-flow obstacles are subjected to the dynamic pressure exerted by flowing water. This is discussed in Section 2.3.4 for wrinkles and Section 2.3.5 for uplifted geomembranes. Prior to the presentation of these two sections, it is useful to understand the formation of geomembrane wrinkles. This is addressed in the following section.

2.3.2 Geomembrane wrinkles

The formation of geomembrane wrinkles as a result of thermal expansion of a geomembrane is complex. Three types of parameters are involved: parameters related to the mechanical properties of the geomembrane, parameters related to the thermal expansion of the geomembrane, and parameters related to the manufacturing and installation of the geomembrane. These three types of parameters are discussed below.

The influence of the mechanical properties of the geomembrane on the formation of wrinkles was the subject of a theoretical analysis by Giroud and Morel (1992) [4]. This theoretical analysis has demonstrated and quantified the essential role of two mechanical parameters in the formation of geomembrane wrinkles, the bending modulus of the geomembrane and the interface shear strength between the geomembrane and the underlying material. The theoretical predictions have been verified in the field. The influence of these two mechanical parameters is discussed below:

- Two geomembranes, having different bending moduli but having the same coefficient of thermal expansion and subjected to the same increase in temperature, exhibit the same amount of expansion. The extra length of geomembrane generated by thermal expansion ends up in wrinkles. However, the geomembrane with a high bending modulus exhibits high wrinkles whereas the geomembrane with the low bending modulus does not. One may wonder where the extra length of the low-modulus geomembrane is. The explanation is that the extra length of the high-modulus geomembrane is in a small number of high wrinkles whereas the same extra length of the low-modulus geomembrane is in a very large number of wrinkles that are hardly visible in the field. Observers who report seeing many wrinkles in the case of high-modulus geomembranes (often referred to as stiff geomembranes), such as HDPE geomembranes, have, in fact, seen a limited number of high wrinkles. The bending modulus depends on the composition of the geomembrane, its thickness, and its temperature. The greater the geomembrane thickness, the higher the bending modulus and, therefore, the higher the wrinkles. The lower the temperature, the higher the bending modulus, but not necessarily the higher wrinkles because, at the same time, the thermal expansion is low.
- The interface shear strength between the geomembrane and the soil or other material underlying the geomembrane has a significant influence on the formation of wrinkles. The theoretical analysis has shown that: the lower the interface shear strength between the geomembrane and the underlying material, the greater the required distance between wrinkles to ensure that the interface shear stresses balance the wrinkle bending force; and, the greater the distance between wrinkles, the higher the wrinkles necessary to accommodate a given extra length of geomembrane with fewer wrinkles. Essentially, the more slippery the geomembrane/subgrade interface, the higher the wrinkles. Therefore, HDPE geomembranes with a textured lower face exhibit wrinkles that are lower and more closely spaced than the wrinkles of smooth HDPE geomembranes with the same thickness.

The magnitude of the wrinkles depends on the magnitude of the thermal expansion that generates the extra length of geomembrane that ends up in the wrinkles. Three factors are involved, which are discussed below:

- The coefficient of thermal expansion depends on the type of geomembrane. It is high (approximately 1 to 2 × 10–4/ ºC) for thermoplastic geomembranes such as HDPE and PVC geomembranes. It is much lower for elastomeric geomembranes such as

EPDM geomembranes and geomembranes reinforced with a woven textile fabric. However, thermally bonding a polymeric geomembrane with a nonwoven geotextile has no significant effect on its coefficient of thermal expansion.

- The temperature of an exposed geomembrane depends on air temperature, wind, and the amount of solar radiation converted into heat, which depends on the geomembrane color and solar radiation reflectance. On a sunny day in the field, the temperature of black geomembranes may exceed 80ºC, whereas geomembranes with a white surface reach a maximum temperature of 50 to 60ºC, and possibly less with a surface treatment to increase reflectance. The geomembrane temperature is a key factor of thermal expansion and it has a significant effect on the bending modulus of thermoplastic geomembranes, the higher the temperature the lower the bending modulus. Since air temperature and solar radiation vary during the day, the installation of wrinkle-prone geomembranes may be done in early morning (or even at night) in hot countries or hot season.
- The difference of geomembrane temperature between the moment the geomembrane is installed and a given time of interest (e.g. the time the cover material is placed, the time water enters the canal, etc.) controls the geomembrane expansion.

Some aspects of geomembrane manufacturing and installation have an impact on the formation of wrinkles, as discussed below:

- The manufacturing process of the geomembrane may have an impact on the formation of wrinkles. Thus, polyethylene geomembranes manufactured using the blown-film process have two folds located at 1/4 and 3/4 of the panel width, which are preferred locations for wrinkle formation in the field. Since these folds affect the spacing between wrinkles, they also affect the height of the wrinkles. All other properties being identical, stiff geomembranes manufactured with the blown film process typically exhibit higher wrinkles than similar geomembranes manufactured with a flat die. Similarly, seams between geomembrane panels also have an impact on the location of geomembrane wrinkles.
- Installing wrinkle-prone geomembranes is a challenge. A wrinkle-prone geomembrane installed without wrinkles in early morning will exhibit growing wrinkles as the air temperature increases. A wrinkle-prone geomembrane installed without wrinkles during a sunny day will rapidly exhibit wrinkles at its temperature increases due to solar radiation. Clearly, a wrinkle-prone geomembrane will exhibit wrinkles during warm and/or sunny days unless it is covered with a heavy layer as soon as it is installed.

Considering the great difference in bending modulus, coefficient of thermal expansion, color and solar radiation reflectance between different geomembranes, there are clearly geomembranes that favor the formation of high wrinkles and geomembranes that do not. It is important to control the formation of wrinkles because they have several detrimental consequences for example: high wrinkles complexify the seaming of geomembrane panels during geomembrane installation; high wrinkles make it practically impossible to place a cast-in-situ concrete cover or prefabricated concrete panels on a geomembrane; and wrinkling of exposed geomembranes may affect their hydraulic performance, as indicated in Sections 2.1.2 and 2.3.4. Below are examples of precautions:

- When environmental conditions are particularly critical, such as in arid regions, preventing high wrinkles may drive the selection of the type of geomembrane that will be used, i.e. polymer, thickness, surface color, solar radiation reflectance, and friction of the geomembrane face in contact with the underlying material.
- Under hot climate, if a geomembrane prone to exhibit wrinkles has been selected, it is often necessary to install this geomembrane in the early hours of the day, which

was done, for example, for the installation of an HDPE geomembrane at the Toshka Canal in Egypt.

- When the geomembrane is to be covered by concrete, a successful strategy has been to place the concrete a shortly after seaming the geomembrane, i.e. before a significant change of temperature could occur. To do so, a concrete paver was closely following the seaming operations at the Toshka Canal.

Regardless of the precautions taken, exposed geomembranes with a high bending modulus, such as HDPE, geomembranes, will generally exhibit wrinkles. Therefore, it is preferable to place a cover layer on top of high-bending-modulus geomembranes, installed as flat as possible and as soon as they are installed, i.e. before a temperature change triggers the formation of wrinkles.

2.3.3 Flowing water dynamic pressure

Obstacles across the water flow are exposed to the dynamic pressure of water, p_{dyn}, which is expressed by the following equation:

$$p_{dyn} = \alpha_{dyn}\, \rho_w \frac{v^2}{2} \tag{24}$$

where α_{dyn} is a coefficient that quantifies the effectiveness of the dynamic pressure, generally (but not always) lower than 1, and v is the flow velocity.

2.3.4 Dynamic pressure on geomembrane wrinkle

An example of obstacle is a geomembrane wrinkle perpendicular to the flow direction. A conservative estimate of the force per unit width applied by the dynamic pressure in the wrinkle can be obtained by using Equation 24 as follows, with the factor, α_{dyn}, equal to 1.0:

$$F / W = \rho_w \frac{v^2}{2} H_{wrink} \tag{25}$$

where F/W is the force per unit width exerted on the wrinkle by the flowing water, and H_{wrink} is the height of the wrinkle after bending under the effect of the flowing water. (Note: the height of the wrinkle is assumed to be measured using as a reference the plane of the geomembrane in contact with the supporting material, away from wrinkles.)

Calculations were performed assuming that the height of a high wrinkle was reduced to 50 mm due to bending under the action of flowing water. For H_{wink} = 0.05 m, and a high flow velocity of 5 m/s (which may exist at locations where the flow velocity is increased, for example downstream of some obstacles), Equation 25 gives a force by unit width of 0.6 kN/m. It is interesting to compare the force per unit width thus calculated with the strength of a geomembrane seam to evaluate if a seam could fail if it were located next to a wrinkle subjected to dynamic pressure from the flowing water. Typical order of magnitudes of peel strength of well-executed seams on 1.5 mm thick geomembranes are: 16 kN/m (HDPE), 13 kN/m (LLDPE), 6 kN/m (PP), 3 kN/m (PVC), and 1 kN/m (EPDM). It should be noted that flexible geomembranes (such as PP, PVC and EPDM) are less likely to exhibit large wrinkles than stiff geomembranes (such as HDPE), as shown by Giroud and Morel (1992) [4]. It appears from this calculation that a well-executed seam, located next to a geomembrane wrinkle, can be expected to resist stresses generated by the action of flowing water on the

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wrinkle. However, poorly executed seams could fail as a result of the action of flowing water on a geomembrane wrinkle.

2.3.5 Dynamic pressure on uplifted geomembrane lining

The ultimate case of geomembrane departure from a flat condition occurs when the geomembrane is uplifted. (See the uplift mechanisms in Sections 3.3 to 3.6.) An uplifted geomembrane exhibits a significant cross-section area across the flow. The force obtained by multiplying this cross-section area by the dynamic pressure (as expressed by Equation 24) is large (as indicated in Section 3.3.3). If the geomembrane is properly anchored laterally, in particular at the toes of the side slopes, the central part of the geomembrane lining moves downstream while the sides where the geomembrane is anchored cannot move. As a result, the geomembrane is distorted, in particular in the vicinity of the lateral anchorages. The stresses associated with the distortion of the geomembrane are added to the tensile stresses caused by uplift-generated deformation and the stresses due to the drag force. Clearly, a geomembrane lining that has been uplifted (for any reason) is subjected to high stresses, which may result in geomembrane rupture.

2.4 Interaction between flowing water and obstacles

Any obstacle across a canal disturbs the flow of water. Thus, localized high flow velocities and turbulence may exist downstream of bridge piers and flow-regulating structures (such as gates and weirs). Turbulence is particularly marked in the case of orifice gates, i.e. gates that control the passage of water through an orifice, as opposed to weir gates that control flow of water on their top.

In some cases, a hydraulic jump, which is a major disturbance of the flow causing major turbulence, takes place downstream of gates, weirs and bridge piers. A hydraulic jump is a rather abrupt rise in the liquid surface which occurs at the transition from supercritical flow to subcritical flow, i.e. when water flowing at high velocity (i.e. above the critical velocity) discharges into a zone of water flowing at low velocity (i.e. below the critical velocity). The critical velocity (which corresponds to a Froude number equal to 1) is given by the following equation:

$$v_{crit} = \sqrt{\frac{g\,A_w}{W_v}} \tag{26}$$

where A_w is the cross section of water in the canal, and W_w is the width of the water surface.

In the case of a trapezoidal canal, Equation 26 becomes:

$$v_{crit} = \sqrt{\frac{g\,d_w\left(B_{invert}\tan\beta + d_w\right)}{B_{invert}\tan\beta + 2\,d_w}} \tag{27}$$

Hence for a rectangular canal:

$$v_{crit} = \sqrt{g\,d_w} \tag{28}$$

Damage to exposed geomembrane linings due to turbulence has been observed downstream of gates, weirs and bridge piers in several canals. Turbulence can also be caused by a sudden change in the longitudinal slope of a canal, from a steeper slope (e.g. a chute) to a flatter slope, or by a drop (i.e. an intentional localized, step-shaped, decrease in canal bottom elevation used to maintain the canal bottom longitudinal slope constant on each side of the drop).

High flow velocities significantly increase the detrimental effects of the dynamic pressure because it is proportional to the square of the velocity (Equation 24). Turbulence creates repeated stresses on portions of geomembrane that can move, for example portions of geomembrane that are uplifted. Tests have shown that repeated loading accelerates the development of cracks in geomembranes that are susceptible to cracking (Scheirs 2022) [7].

3 Possible failure mechanisms

3.1 Overview of possible failure mechanisms

The authors of the present paper are aware of cases where inadequate design and, even, absence of design, was a major contributor to the failure of a geomembrane lining. Therefore, it is important, at the design stage, to identify possible causes of failures and to develop solutions to prevent possible failures.

In the following sections, different mechanisms that could impair the integrity of a geomembrane lining in a canal are discussed. It is important to realize that, even though geomembranes are less susceptible to rupture than concrete linings due to their flexibility and extensibility, there are failure mechanisms that may affect geomembrane linings in canals. It should be noted that the list of failure mechanisms addressed in the following sections is not exhaustive. In other words, other situations detrimental to geomembrane linings in canals may exist.

3.2 Geotechnical problems affecting canal linings

3.2.1 Detrimental effect of geotechnical problems on geomembrane linings

The geotechnical problems that may affect canal linings, and geomembrane linings in particular, are essentially characterized by deformation of the soil and differential displacements between soil and structures connected with the linings. These deformations and displacements can be small (e.g. of the order of centimeters) or can be large, such as a large breach in a dike or collapse of soil that causes localized lack of support of the lining.

Concrete linings often crack even in the case of small deformation of the supporting soil, whereas geomembrane linings, thanks to their flexibility and extensibility, can accommodate relatively large deformations (to a degree which depends on the type of geomembrane). The deformations that cause cracking of concrete and the large deformations that may tear a geomembrane result in leakage that further deteriorate the soil underlying the lining, thereby increasing the size of cracks and tears, which results in more leakage, and so on, until it may be necessary to interrupt the operation of the canal.

The design of canals to prevent geotechnical problems is addressed in Section 4.4.

3.2.2 Origin of geotechnical problems affecting canal linings

A number of geotechnical problems that affect canal linings are caused, or aggravated, by leakage/seepage of canal water through and/or around the lining. The mechanisms involved include: settlement of poorly compacted soils (i.e. soils compacted in excessively thick layers and with excessively low water content); erosion of certain types of soils; settlement of collapsible soils such as loess; dissolution of soils containing gypsum; cavity formation in karstic terrain; swelling of some clays; and instability of soil in the case of canals located on hillsides.

Some geotechnical problems likely to affect canal linings may also exist in the absence of leakage/seepage. For example, this is the case of: excessive soil settlement due to insufficient compaction of the soil supporting the lining; differential settlement between the lining and appurtenant structures such as gates and bridge piers; and instability of soil in the case of canals located on hillsides.

3.2.3 Swelling clays

Swelling clays (also called ‘expansive clays’) are present in different parts of the world. They contain montmorillonite and, as a result, they exhibit significant swelling when their water content is increased and significant shrinkage when their water content is reduced. As long as swelling clays are not cracked, their permeability is low and, therefore, changes of water content are slow.

An important property of a swelling clay is its swelling pressure, which is defined as the compressive stress generated when a swelling clay is hydrated in a fixed volume. In other words, the swelling pressure is the compressive stress that prevents swelling of the clay as it is being hydrated. The swelling pressures of different swelling clays span over a wide range from 10 kPa to hundreds of kPa, depending on the composition, density and initial water content of the swelling clay.

If the water content of a swelling clay is increased, a volume increase of the order of 50% is not uncommon. The pressure of the water in the canal (plus the pressure due to any layer of soil or concrete in the canal) is lower than most of the above-mentioned swelling pressures and, therefore, is not sufficient to prevent the swelling of most swelling clays. A geomembrane, thanks to its flexibility and extensibility (to a degree which depends on the type of geomembrane), can accommodate the strains associated with this volume increase. However, less swelling is sufficient to significantly deteriorate a concrete lining. As a result of concrete lining deterioration, water leaks from the canal and causes more swelling. In India where brick lining is often used in traditional canals, lining deterioration due to swelling clays is frequently observed because brick linings are even more prone to cracking than concrete linings.

If the water content of a swelling clay decreases, the clay shrinks. As a result, deep cracks typically develop in the clay. Cracks in the clay that are 10 mm wide and more than one meter deep can be observed after wet-dry cycles. Soil movements due to swelling-shrinkage associated with wet-dry cycles can be accommodated by geomembrane linings but may cause cracking of concrete linings and, therefore, leakage. Leaking water flowing in the cracks may cause internal erosion of the clay, resulting in piping (i.e. free flow of water in the clay cracks, which causes more erosion). Wet-dry cycles may be caused by canal operation such as on-off irrigation, a mode of canal operation which consists in managing irrigation water by supplying water to some canals for a certain period of time (e.g. two weeks) while other canals do not get water, and vice-versa.

Description of swelling clays and detrimental consequences to canals can be found in Kapadia (2018a) [8]. Design measures to prevent the detrimental effects of swelling clays are addressed hereafter in Section 4.4.3.

3.2.4 Action of freezing and thawing on linings

Upward movement of the soil surface caused by freezing, or 'frost heave', is commonly observed in cold parts of the world. For example, frost heave in the range of 150 to 300 mm is typically observed on roadways, with inadequate base layers, built on frost-susceptible soils. The mechanism leading to frost heave can be summarized as follows:

- If the ambient temperature is significantly below 0ºC for some time, the temperature of the lining, and of a certain thickness of soil beneath the lining, is below 0ºC.
- Water vapor present in the soil tends to migrate from relatively warm areas toward relatively cold areas. Therefore, vapor migrates toward the cold lining or the already frozen soil.
- In contact with the lining or the frozen soil just beneath the lining, the vapor condenses and freezes. As a result, ice accumulates, and pockets of ice (called ice lenses) are formed immediately beneath the lining or in the vicinity.
- If there is a supply of water in the soil near the lining, the phenomenon of vapor migration continues as long as freezing temperatures prevail. As a result, thick ice lenses are formed and the lining is uplifted.

The following comments can be made on the mechanism described above:

- It is important to note that the uplifting of the lining (i.e. the so-called frost heave) is not due to a volume increase of the water fraction of the soil. Indeed, the volume of water increases by only 9% upon freezing. The water content of a saturated soil being of the order of 30% by volume, the resulting swelling of the soil is less than 3%, which alone would not cause significant heave of the soil surface. When frost heave is significant, it is essentially due to the formation and growth of ice lenses.
- Frost heave occurs mostly with silty soils. Indeed, soils finer than silts (e.g. clays) have a low permeability, which slows down vapor migration, whereas capillarity is absent or insignificant in soils coarser than silts (e.g. sands), which precludes vapor migration.

The following comments can be made on the behavior of different lining materials in case of freezing and thawing:

- Geomembranes, because of their flexibility and extensibility can accommodate large deformations (to a degree which depends on the type of geomembrane). However, if a geomembrane lining is covered by concrete, this concrete cover is likely to behave the same way as concrete lining (see below). Sharp edges of a cracked concrete cover can mechanically damage the geomembrane it is intended to protect.
- Cast-in-situ concrete linings and concrete layers used to cover geomembrane linings lack flexibility and, therefore, can be cracked by cycles of frost heave and soil softening due to thawing.
- Prefabricated concrete panels are more likely to crack than cast-in-situ concrete because they are thinner (50 mm, sometimes less) than cast-in-situ concrete (typically 75 to 100 mm, sometimes 150 mm, and even 200 mm at Toshka Canal, in Egypt).
- Prefabricated concrete panels, since they are light, can easily be uplifted by frost heave. As a result, the panel pattern can be disorganized and, on the side slopes, some panels can slide downward on top of adjacent lower panels in case of differential frost heave between panels. If prefabricated concrete panels are used to cover a geomembrane and if they are displaced in a disorganized manner, the corners of the panels can cause severe mechanical damage to the geomembrane.

In addition to the problems caused by frost heave, frost has a direct action on concrete. Indeed, water, present in the pore volume of concrete, expands when it freezes. If the resulting pore pressure exceeds the tensile strength of the concrete, cracks develop. This is a classical mode of deterioration of concrete.

3.3 Geomembrane uplift by groundwater

3.3.1 Occurrence of high groundwater

Groundwater surface may be high, generally for hydrogeological reasons, and, sometimes, because of leakage/seepage, either from unlined canals in the vicinity of the considered canal or because of leakage upstream in the same canal. Unless there is a record of groundwater level specific to the site where a canal is to be located, it is often difficult to predict groundwater level. It is recommended to use the services of experienced hydrogeologists to evaluate the risk of high groundwater at a canal location.

Groundwater that rises above the level of the bottom of a canal has detrimental effects during installation of the geomembrane lining and when the geomembrane is in service. This is discussed in the following sections.

3.3.2 Impact of high groundwater on geomembrane installation

Seaming geomembranes requires the two faces to be assembled to be perfectly dry and clean. Therefore, during geomembrane installation, the presence of water (rainwater, runoff water, groundwater, etc.) at the bottom of a canal can make seaming of geomembranes more challenging, and sometimes impossible. Strategies must be implemented to maintain the geomembrane faces dry and clean. In extreme situations, lowering the groundwater table by pumping may become necessary. This requires substantial pumping equipment if the soil permeability is high. For the above reasons, adequate prediction of the depth of the groundwater table at the season a geomembrane lining is to be installed is often necessary.

3.3.3 Impact of high groundwater on geomembrane lining performance

Regarding the detrimental effect of high groundwater on a geomembrane lining in service, two cases can be considered: (1) the groundwater surface is higher than the canal bottom geomembrane lining but is lower than the water surface in the canal; and (2) the groundwater surface is higher than the water surface in the canal. In the first case, the geomembrane lining is not uplifted by the groundwater, whereas, in the second case, the geomembrane lining is uplifted by groundwater. It should be noted that, in both cases, the geomembrane may also be uplifted for other reasons, but only the effect of groundwater is addressed in this section.

If the groundwater surface is higher than the canal bottom geomembrane lining but is lower than the water surface in the canal, an upward pressure is applied by groundwater on the lower face of the geomembrane lining. This upward pressure is not sufficient to uplift the geomembrane lining, but the normal stress that presses the geomembrane lining against the underlying soil is reduced. Consequently, the magnitude of the resisting shear stress beneath the geomembrane lining is reduced, which increases the likelihood for the geomembrane to be displaced by the drag forces exerted by the flowing water.

If the groundwater surface is higher than the water surface in the canal, the upward pressure applied by the groundwater under the geomembrane is higher than the downward pressure applied on the geomembrane by the canal water. As a result, the geomembrane is uplifted. The fact that the geomembrane lining is uplifted has two detrimental consequences: the resisting stress under the geomembrane is zero and the geomembrane lining is subjected to the dynamic pressure exerted by the flowing water, in addition to the drag force which exists whether the geomembrane is uplifted or not. The magnitude of these two forces can be compared as follows.

If the geomembrane lining is uplifted over a width B (which may be the width of the canal bottom) and a length L, the drag force can be expressed as follows, based on Equation 16:

$$F_{drag} \approx \frac{\rho_w\, g\, n_M^2\, v^2}{d_w^{1/3}} B\, L = K_{drag}\, \rho_w\, v^2\, B \tag{29}$$

where d_w is the depth of water on top of the uplifted geomembrane, and:

$$K_{drag} \approx \frac{g\, n_M^2\, L}{d_w^{1/3}} \tag{30}$$

The force due to the dynamic pressure can be expressed as follows, based on Equation 24 conservatively used with $\alpha_{dyn} = 1$:

$$F_{dyn} \approx \frac{\rho_w\, v^2}{2} B\, d_{uplift} = K_{dyn}\, \rho_w\, v^2\, B \tag{31}$$

where d_{uplift} is the distance between the uplifted geomembrane and the canal bottom, and:

$$K_{dyn} = \frac{d_{uplift}}{2} \tag{32}$$

Comparing the effects of drag force and dynamic force boils down to comparing K_{drag} and K_{dyn}. Numerical calculations show that the dynamic force is generally far superior to the drag force. For example, if a geomembrane with a Manning's coefficient $n_M = 0.012$ is uplifted by 0.5 m over a 12 m length and the depth of water on top of the uplifted geomembrane is 0.3 m, the dynamic force is approximately ten times the drag force.

A situation where an uplifted geomembrane is particularly exposed to dynamic pressure by the flowing water occurs when the geomembrane lining if uplifted by high groundwater when a canal is empty and the canal is suddenly put back in service. Since the geomembrane was uplifted when the canal was empty, it was more uplifted than when the canal is in service and the pressure of water in the canal counteracts the uplift pressure. As a result, it is more exposed to the dynamic pressure of the flowing water.

If there is a risk of geomembrane uplift by groundwater, it is recommended to inspect the geomembrane lining before putting the canal back in service. If possible, it may be preferable to put the canal in service progressively. Also, before a canal is emptied for any reason (operation, maintenance, repair), it is recommended to check the groundwater level.

3.3.4 Precautions in case of possible high groundwater

At the construction time, for all of the above reasons, it is preferable to avoid installing a lining in canals where the groundwater surface is at the level of the canal bottom or higher. At the design stage, in areas where high groundwater can be predicted, possible solutions include: (1) raising the canal bottom and widening the canal to retain the same flow capacity (while ensuring proper connection between the stretch of canal where the bottom has been raised and the rest of the canal); and (2) leaving the canal unlined. If these solutions are impractical, then the geomembrane should be ballasted and possibly equipped with a pressure-relief system, as discussed hereafter in Section 4.5.6. It is important to note that ballasting the bottom of a canal does not prevent geomembrane uplift on the side slopes; however, the span of geomembrane able to be uplifted on side slopes is substantially lower than on the canal bottom.

3.3.5 The myth of the floating geomembrane lining

Regarding geomembrane uplift, it is necessary to dispel the myth of geomembrane linings uplifted because their density is lower than the density of water. Two cases should be considered: a piece of geomembrane immersed in water; and a geomembrane lining containing water.

A piece of geomembrane immersed in water floats if its density is lower than the density of water, and sinks if its density is higher than the density of water. This statement results from the Archimedes principle and it can be demonstrated as follows in the simple case of a horizontal piece of geomembrane immersed under a depth, d_w, of still water. The uplift pressure is expressed as follows:

$$p_{uplift} = \rho_w\, g\, \left(d_w + t_{GMB}\right) - \rho_w\, g\, d_w - \rho_{GMB}\, g\, t_{GMB} \tag{33}$$

where t_{GMB} is the geomembrane thickness and ρ_{GMB} is the geomembrane density. On the right side of the above equation, the first term is the upward static pressure below the geomembrane, the second term is the downward static pressure above the geomembrane, and the third term is the weight of geomembrane per unit area. Equation 33 is simplified as follows:

$$p_{uplift} = \left(\rho_w - \rho_{GMB}\right) g\, t_{GMB} \tag{34}$$

Equation 34 shows that the uplift pressure is positive, therefore the geomembrane is uplifted, if the geomembrane density is lower than the water density.

The above equation is for the case of a piece of geomembrane immersed in water. In the case of a geomembrane lining containing water and subjected to groundwater pressure on its lower face, the uplift pressure exerted on the horizontal part of the geomembrane lining is expressed by the following equation where the first term is different from the first term of Equation 33:

$$p_{uplift} = \rho_w\, g\, \left(d_{gw} + t_{GMB}\right) - \rho_w\, g\, d_w - \rho_{GMB}\, g\, t_{GMB} \tag{35}$$

where d_{gw} is the height of groundwater measured above the level of the top of the geomembrane. Equation 35 is simplified as follows:

$$p_{uplift} = \rho_w\, g\left(d_{gw} - d_w\right) + \left(\rho_w - \rho_{GMB}\right) g\, t_{GMB} \tag{36}$$

On the right side of Equation 36, the second term is generally negligible with respect to the first term. For example, if the groundwater level is half a meter above the water level in the canal ($d_{gw} - d_w = 0.5$ m), the second term is three orders of magnitude lower than the first term, considering typical geomembrane thicknesses and densities. Therefore, regardless of whether the second term is negative or positive (i.e. geomembrane denser or not than water), the geomembrane lining is uplifted by groundwater if the groundwater level is higher than the water level in the canal.

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In conclusion, there is a fundamental difference between a piece of geomembrane, which is immersed in a body of water, and a geomembrane lining, which separates two bodies of water. A piece of geomembrane floats or sinks depending on its density relative to the density of water. A geomembrane lining is uplifted if the pressure from underneath (e.g. from groundwater) is higher than the pressure from above, i.e. the pressure from the water in the canal.

The above demonstration is related to well-known facts as follows:

- A piece of steel sinks if it is immersed in water, whereas a steel boat floats because it is uplifted by the pressure of water on the hull until the force resulting from the water pressure on the hull (known as Archimedes thrust) balances the weight of the boat.
- If geomembrane uplift was governed by the density difference between the geomembrane and the surrounding fluid, geomembranes would not be uplifted by wind since the density of geomembranes is three orders of magnitude higher than the density of air.

In Section 3.3, only static pressures are considered. However, a geomembrane lining can also be uplifted by dynamic pressure exerted by flowing water, as discussed in the following section.

3.4 Geomembrane uplift by flowing water

3.4.1 Mechanism of geomembrane uplift by flowing water

If there is no hole in the geomembrane and no high groundwater, the static pressure applied by the water present in the canal keeps the geomembrane in contact with the underlying medium and the drag force is not sufficient to displace the geomembrane due to the resisting force beneath the geomembrane, as discussed above in Section 2.2.5. If there is a hole in the geomembrane, and if water leaking through this hole accumulates under the geomembrane (in other words, if the material under the geomembrane is not draining), water stagnates under the geomembrane and, as a result, the water pressure beneath the geomembrane is then the total pressure or stagnation pressure, which is the static pressure, given above by Equation 8, plus the dynamic pressure, given above by Equation 24, hence:

$$p_{total} = p_{satgn} = \rho_w \, g \, h_w + \alpha_{dyn} \, \rho_w \, \frac{v^2}{2} \tag{37}$$

As a result, the geomembrane is uplifted by the dynamic pressure, p_{dyn}, given above by Equation 24, which is the difference between the pressure beneath the geomembrane (i.e. the stagnation pressure, p_{total}) and the pressure above the geomembrane (i.e. the static pressure given by Equation 8).

3.4.2 Full-scale test of geomembrane uplift by flowing water

The mechanism of geomembrane uplift by flowing water dynamic pressure has been investigated at the Technical University of Munich, Germany, in full-scale tests conducted in a concrete canal, 95 m long, 4 m wide at the bottom, 1.5 m deep, with 1.25H:1V side slopes (TUM 2005) [9]. The geomembranes tested were: a 2.5 mm thick HDPE geomembrane, which was placed on a polypropylene needle-punched nonwoven geotextile with a mass per unit area of 500 g/m^2; and a composite geomembrane that consisted of a 2.5 mm thick PVC geomembrane heatbonded to a polyester geotextile with a mass per unit area of 500 g/m^2. The flow velocity was 1 m/s. In each test, the geomembrane was longitudinally anchored to the underlying concrete at the toe of the two side slopes. In some of the tests, the geomembrane was intentionally cut transversally over approximately 1 m thereby creating a hole in the geomembrane.

The full-scale test at the Technical University of Munich (TUM 2005) [9] confirmed that the present of a hole in the geomembrane resulted in uplifting the geomembrane. The test was also used to show the effectiveness of pressure-relief valves.

3.4.3 Effect of drainage on geomembrane uplift by flowing water

It should be noted that, in the full-scale tests described above, the geomembrane was underlain by concrete. As a result, the water that leaked through the intentional holes could accumulate under the geomembrane. If an effective drainage layer had been placed between the geomembrane and concrete, it can be predicted that water leaking through the geomembrane hole would not have been able to accumulate under the geomembrane and the geomembrane would not have been uplifted, but could have experienced some displacement in its central portion, assuming that the anchorage of the geomembrane on each side of the canal bottom was effective.

It is important to note that the above discussion is based on the assumption that the hypothetical drainage layer is effective. In other words, it is assumed that the drainage capacity remains significantly higher than the rate of leakage through the geomembrane, even if the hole that initiated the leakage becomes a large tear in the geomembrane. It is also assumed that the hypothetical drainage includes an adequate outlet, so the collected water is promptly evacuated without pressure buildup.

3.5 Geomembrane uplift by wind

3.5.1 Actions of wind on geomembrane linings

If a canal is lined with an exposed geomembrane, the wind can have the following actions above the water level, or on the entire geomembrane lining when the canal is empty: (1) the geomembrane can be uplifted by negative air pressure (‘suction’) generated by the wind (see Section 3.5.2 below); (2) wind gusts may cause repeated displacement of the geomembrane lining; and (3) in case of wind, air may enter under the geomembrane lining if it is torn or not sealed on a portion of its periphery. This last action of wind may occur in particular during installation of the geomembrane, whether the geomembrane lining is intended to be exposed of covered, if the geomembrane is not secured at its periphery by temporary ballast such as sandbags.

The above-described actions may have the following consequences: (1) when the wind recedes, the geomembrane may not return exactly to its original position, with results in localized stresses; and (2) the geomembrane may be subjected to a variety of concentrated stresses often resulting in tears, tensile ruptures, seam failures, creases, etc. In extreme cases, the geomembrane, or a portion of it, must be replaced.

The risk of uplift of geomembrane by wind increases with the size of the lining. Therefore, the risk of uplift of geomembrane by wind increases as the size of the canal increases. Also, considering the geometry and the relatively small size of many canals, the risk of uplift by wind of exposed geomembrane linings in canals is lower than in the case of large exposed geomembrane linings installed in reservoirs, waste storage landfills and mining facilities. However, the risk of uplift by wind during geomembrane installation should be carefully considered in the case of large canals.

3.5.2 Mechanism of geomembrane uplift by wind

When the wind blows, a positive or a negative pressure is applied on portions of a geomembrane lining. Whether the pressure is negative ("suction") or positive depends on the direction of the wind and the geomembrane lining configuration, as discussed in Giroud *et al.* (1995a) [10]. The geomembrane may be uplifted in all areas subjected to suction if the magnitude of the suction exceeds the component nor mal to the slope of the weight per unit area of the geomembrane, as expressed by the following equation from Zornberg and Giroud (1997) [11]:

$$\Delta p_{suction} = \lambda_{wind}\ \rho_{air}\ \frac{v_{wind}^2}{2}\ e^{-(1.25\times10^{-4})z} - \mu_{GMB}\ g\ \cos\beta_{GMB} \tag{38}$$

where λ_{wind} is a factor, generally lower than 1, depending on the exposure of the geomembrane, ρ_{air} is the density of air, v_{wind} is the wind velocity, z is the elevation above sea level, and β_{GMB} is the inclination (with respect to a horizontal plane) of the geomembrane subjected to the suction. It should be noted that the influence of elevation is not negligible; for example, the value of the exponential factor is 0.78 for an elevation of 2000 m. Values of λ_{wind} can be found in Giroud *et al.* (1995a) [10].

Conservatively at sea water level (z = 0), with the factor λ_{wind} = 1.0 for the maximum exposure, which is at or near the canal crest, and neglecting the geomembrane weight, the following approximate equations derived from Equation 38 can be used:

$$\Delta p_{suction} = 0.65\ v_{wind}^2 \quad \text{with } \Delta p_{suction}\left(\text{Pa}\right) \text{ and } v_{wind}\left(\text{m/s}\right) \tag{39}$$

$$\Delta p_{suction} = 0.05\ v_{wind}^2 \quad \text{with } \Delta p_{suction}\left(\text{Pa}\right) \text{ and } v_{wind}\left(\text{km/h}\right) \tag{40}$$

At the bottom of the canal, the above suctions can be divided by a factor 2. The suction calculated using the above equations is used to size the anchor trenches or anchor benches used to secure the geomembrane lining, as described in Giroud *et al*. (1995a, 1999) [10, 12]. The method is also used to calculate the deformation and strain of the geomembrane (Giroud 2009) [13].

3.5.3 Influence of the geomembrane on the risk of uplift by wind

It is often mentioned that heavy geomembranes, such as bituminous geomembranes, are less likely to be uplifted by wind than light geomembranes. This statement can be quantified by using Equation 38, which shows that, at elevations between 0 and 2000 m, geomembranes with a mass per unit area of the order of 4 to 6 kg/m^2 (such as bituminous geomembranes) start being uplifted for wind velocities of the order of 40 km/h whereas geomembranes with a mass per unit area of the order of 1 to 2 kg/m^2 (such as polymeric geomembranes) start being uplifted for wind velocities of the order of 20 km/h. Clearly, heavy geomembranes can be uplifted by wind, but the threshold velocity beyond which they are uplifted is higher than for light geomembranes. It should be noted that wind velocities higher than 40 km/h are frequent. Therefore, all geomembranes should be properly anchored of ballasted.

3.5.4 Uplift by wind compared with uplift by flowing water

It is interesting to compare geomembrane uplift by water flowing in canals with the well-known phenomenon of geomembrane uplift by wind. In both cases, the pressure that tends to uplift the geomembrane has the same expression, $\rho v^2/2$, multiplied by similar coefficients. This expression shows that the uplift pressure is proportional to the fluid density and to the square of the fluid velocity. However, the densities of air and water are very different, and the velocities of wind and water are very different.

The density of water is 800 times the density of air. In a canal, the water flow velocity is low (e.g. of the order of 1 to 4 m/s). If the wind velocity is 100 to 200 km/h (28 to 56 m/s), the squared velocity range for wind is approximately 800 times the squared water velocity range in a canal. The two 800 ratios cancel out, which shows that the risk of geomembrane uplift by water flowing in a canal is equivalent to the risk of geomembrane uplift by wind. This comparison should help design engineers to take geomembrane uplift in a canal as seriously as they take geomembrane uplift by wind.

3.6 Mechanical damage to the geomembrane

An exposed geomembrane can be mechanically damaged by the following actions: hail, falling objects, boats, animals, humans, construction, and operation and maintenance activities. To prevent mechanical damage, geomembranes can be protected by a cover layer. However, it is important to ensure that the cover layer (whether it is designed for protection or ballasting) does not cause mechanical damage to the geomembrane. This is addressed hereafter in Section 4.5.11.

4 Design of canal characteristics and lining components

4.1 Design overview

4.1.1 Avoiding failure

As discussed above in Section 3, several mechanisms could cause failure of a geomembrane lining in a canal. The failure mechanisms can be counteracted and their consequences avoided by design measures aimed at optimizing the different characteristics and components of a canal, such as: canal location and geometry; lining options; soil supporting the geomembrane and appurtenant structures; layer covering the geomembrane; and anchorage systems. These are topics addressed in the following sections.

4.1.2 The two aspects of design

It is important to note that the design of each canal component should include two aspects: design to ensure that the considered component performs its function; and design to ensure that the considered component does not harm the geomembrane lining. Also, it is necessary, in each design phase, to evaluate the stresses and strains in the geomembrane lining to make sure that these stresses and strains are compatible with the properties of the geomembrane. For example, it is pointless to design a strong anchorage system if the geomembrane undergoes excessive stresses and strains, and ruptures next to the anchorage system.

4.1.3 Importance of design engineer

Based on the above, it is important that canal linings be designed by a design engineer, with knowledge on canal lining and geomembranes. The authors of the present paper are aware of cases where the absence of a design engineer was a major contributor to the failure of a geomembrane lining. In small canal projects, the canal owner may be tempted to contract directly with an installer. As a result, the role of the design engineer is played by the owner, the installer, or both. In these cases, both the owner and the installer take a significant risk.

4.1.4 Scope of design

As indicated in Section 1.2, leakage/seepage control is not addressed in the present paper. Another important aspect of design is not addressed in the present paper: geomembrane selection. The scope of Section 4 is essentially to describe and quantify the methods that can be used to prevent the failures discussed above in Section 3. Accordingly, the subjects dealt with in Section 4 are: Canal location and geometry (Section 4.2); lining options, including not lining (Section 4.3); design related to the soil associated with the lining (Section 4.4); design of the geomembrane cover (Section 4.5); and, finally, design of exposed geomembrane (Section 4.6).

4.2 Canal location and geometry

4.2.1 Canal location and elevation

As discussed hereafter in Section 4.6, designing a canal lining against geomembrane uplift by high groundwater is not easy and solutions may not be effective or may be too expensive. Therefore, it is wise to consider raising the canal bottom in areas where high groundwater can be predicted and widening the canal to retain the same flow capacity, while ensuring proper connection between the stretch of canal where the bottom has been raised and the rest of the canal, as mentioned in Section 3.3.4. Another option (which is rarely possible) is to change the location of the canal to avoid areas where high groundwater can be expected.

Some stretches of canals may be elevated on earth embankments to cross low areas. It is important to design an embankment that supports a canal as an earth dam would be designed, to avoid the development of high hydraulic gradient and high pore-water pressure in case of leakage of water from the canal. This precaution would minimize the risk of occurrence of two mechanisms that could cause failure of the embankment: internal erosion (‘piping’) due to high hydraulic gradient and slope instability due to high pore-water pressure.

4.2.2 Freeboard

In most canals, there is a freeboard. Three cases should be considered: (1) there is no lining; (2) the lining extends to the side slope crest; and (3), the lining does not extend to the side slope crest and is terminated at some level on the slope.

If there is no lining, in other words if the canal is unlined, the freeboard is the vertical distance between the design water level and the crest of the side slopes.

If the lining extends to the side slope crest (which is the general case for lined canals), the freeboard is the vertical distance between the design water level and the crest of the side slopes. In this case, the freeboard should be sufficient to prevent excess canal water from overtopping the lining and the canal berm. The detrimental effects from such overtopping are: accumulation of water behind the lining, erosion of the canal berm, and, if the lining is a geomembrane and there is an anchor trench, saturation of the anchor trench backfill, which could weaken the anchorage of the geomembrane lining.

If the lining does not extend to the side slope crest, the freeboard is the vertical distance between the design water level and the top of the lining. In this case, the freeboard should be sufficient to prevent excess canal water from overtopping the lining. The detrimental effects from such overtopping are: accumulation of water behind the lining, and erosion of the canal bank above the top of the lining (which would weaken the lining anchorage, if the lining is a geomembrane).

Excess water in the canal may occur as a result of surges during canal operation, waves due to wind or canal operation. precipitation, runoff, or flooding.

4.2.3 Canal side slopes

The selection of the inclination of the side slopes should take into account not only the flow capacity of the canal and the geotechnical stability of the slope supporting the geomembrane lining, but also the stability (during construction and in service) of the material covering the geomembrane and its potential displacement relative to the underlying geomembrane. The stability and displacement of the material covering the geomembrane lining is addressed in Section 4.5.10.

4.2.4 Canal longitudinal slope

The longitudinal slope of canals is generally rather uniform along the length of the canal, and can be very small, such as 0.1 m/km (i.e. a longitudinal slope $S = \tan\alpha = 1\times 10^{-4}$, hence a longitudinal slope angle $\alpha = 0.006^{o}$). However, it should be remembered that a localized change in the longitudinal slope or the presence of a control structure (e.g. a weir) or any obstacle in the canal may cause locally a significant increase in the flow velocity and a development of flow turbulence. As a result of flow turbulence, the flow velocity fluctuates locally, which induces repeated stresses in an exposed geomembrane lining. Therefore, it is recommended to protect the geomembrane with a cover material in all areas where turbulence can be predicted based on hydraulic analyses, in particular downstream of structures and obstacles, over a certain distance (e.g. tens of meters) based on a hydraulic analysis. Also, a geomembrane needs to be protected from turbulence generated by boat propellers in navigation canals.

4.3 Lining options

Essential decisions at the beginning of the design phase consist in selecting between ‘to line or not to line’ and between ‘to cover or not to cover’. These two dilemmas are discussed below.

4.3.1 To line or not to line

Regarding the dilemma ‘to line or not to line’, the decision should be made on the basis of technical and economic considerations.

Technical considerations include: (1) to line to decrease leakage/seepage; (2) not to line because of the presence of high groundwater; and (3) to line to decrease the roughness of the canal surface to promote higher flow velocity and, therefore, greater flow capacity. In the case of areas where the soil has a very low permeability, it is appropriate to use unlined canals, such as in the Gezira Irrigation Scheme, in Sudan. In cases where a canal is located alternatively in areas of low permeability and areas of high permeability, it is appropriate to line the canal only in areas of high permeability. This is the case, for example, of the Canal Bas Service des Doukkala, in Morocco, where approximately 15% of the length is lined (while 85% of the canal length is unlined, being in zones of low-permeability clays). Unlined canals are also appropriate in the deltas of large rivers because of high groundwater level, which is the case worldwide.

Economic considerations are different for hydroelectric canals (which are generally short, e.g. a few km) and irrigation canals (which are generally long, e.g. 100 km or more). In the case of hydroelectric canals, it is important to minimize water losses and maximize flow velocity to maximize electricity production; therefore, the cost of lining is generally justified. The situation is different in the case of irrigation canals where the budget is generally limited and the use of a lining may, in some cases, increase the cost of a canal by up to 50%. Thus, there are several irrigation projects with limited budget where canals are lined only in areas where the soil has a high permeability. This strategy consists in limiting leakage/seepage only in areas where it is likely to be very large. Rather than deciding at the design stage which canal stretch is to be lined, some designers opt for putting in service the entire canal unlined and deciding which canal stretch should be lined based on observed leakage/seepage. This option may have a significant cost, because earthwork is necessary to reduce the canal cross section prior to lining since flow is faster in a lined canal than in an unlined canal. Finally, it should be noted that, with the increasing demand for water, it becomes more and more appropriate to line longer stretches of canals and, in a greater number of cases, the entire length of canals, depending on soil permeability.

Despite the many reasons for lining canals, it cannot be ignored that some water resources experts claim that irrigation canals should not be lined so leaking water can recharge aquifers for the benefit of people, including farmers, who can thus extract groundwater by pumping. Discussion of this approach is beyond the scope of the present paper because: (1) it would require an analysis of the differences in quality of irrigation service between water distribution by canals (depending on canal water management sophistication) and groundwater extraction by pumping; and (2) it would lead to mentioning social, economic, environmental and political motivations that may underlie this approach. This complex matter is addressed in the book by Giroud and Plusquellec (2023b) [14] where it is concluded that the best approach to ensure equitable water distribution to farmers while saving energy is to combine leakage/seepage control by lining canals with geomembranes and efficient water distribution with the use of modern systems of canal water management, which is a major challenge in many areas where traditional methods prevail.

4.3.2 To cover or not to cover

A geomembrane lining may be exposed or covered. Therefore, at the design stage there is a dilemma 'to cover or not to cover a geomembrane lining'. The decision should be made on a case-by-case basis.

Leaving a geomembrane lining exposed (i.e. non-covered) has some advantages: the cost is minimized; the geomembrane can be inspected and repaired; and the flow of water is typically faster (in particular if a smooth geomembrane is used and if the cleanliness of the geomembrane is maintained over the years). On the other hand, covering a geomembrane has many advantages: protection against geomembrane deterioration by animals, humans, boats, floating debris, floating ice, falling objects, and hail; protection against geomembrane abrasion by particles in suspension; protection against geomembrane expansion and contraction due to temperature variations; protection against geomembrane degradation by solar radiation; prevention of adhesion of ice and mollusks to the geomembrane; ballasting the geomembrane against uplift by wind; ballasting the geomembrane against uplift by groundwater; ballasting the geomembrane against displacement by action of flowing water; and ballasting the geomembrane seams against peeling. Covers also reduce the temperature of the geomembrane above the water level, thereby reducing the rate of aging of the geomembrane. Essentially, the benefits of a cover layer are protecting, ballasting, and increasing the durability of geomembranes. In addition, a layer of concrete or bituminous concrete is sometimes used at the bottom of canals to allow for traffic of light vehicles and to facilitate operations of service and maintenance.

4.4 Design related to the soil associated with the lining

4.4.1 Design to control soil settlement

To minimize the risk of soil settlement, technical specifications for earthworks should contain detailed prescriptions about compaction of the soil supporting the geomembrane. The prescriptions should include construction requirements (e.g. soil layer thickness and water content) and soil density to be achieved. Quality control of earthwork, in particular compaction, is essential. However, in some irrigation projects, quality control is neglected. Lack of quality control or ineffective quality control during construction often results in compacting excessively thick layers with excessively low water content, especially in arid regions. This results in large soil settlement if the soil thus compacted becomes saturated by water leaking from the canal. Also, poorly compacted soils are susceptible to erosion. An example of canal dike failure due to insufficient compaction is presented in Kapadia (2018b) [15].

To minimize differential settlement between soil and adjacent structures, compaction with special equipment (e.g. hand-held plate compactor) should be specified next to appurtenant structures. Also, concrete structures should not have a vertical face in contact with soil supporting a geomembrane; instead the face should have a slight batter, as recommended by Giroud and Soderman (1995a) [16]. Geomembranes likely to better withstand differential settlements can be selected using the concept of co-energy which combines strength and elongation, as proposed by Giroud and Soderman (1995b) [17] and Giroud (2005) [18].

Deformation of soil is influenced by its water content. Therefore, a geotechnical engineer should evaluate the impact on soil deformation of water content increase due to leakage/seepage and water content decrease due to the presence of an effective lining.

4.4.2 Design to control soil subsidence

While the term ‘settlement’ refers to the downward displacement of the soil surface due to volume decrease of the soil under the action of applied stresses or change in water content (see Section 3.2.2), the term ‘subsidence’ is used herein to refer to the formation of cavities just below the soil surface. Such cavities may result from erosion of the soil, dissolution of the soil by water, or collapse of loess (a type of soil with high porosity). Soils with a high content of gypsum (which is very soluble in water) occur typically in arid areas, which are areas where canals are needed for irrigation. Several occurrences of severe damage to concrete lining due to the formation of cavities caused by gypsum dissolution have been reported.

If there is a possibility of formation of cavities (also called ‘sinkholes’) in the soil supporting a geomembrane lining (see Section 3.2), soil reinforcement can be used and designed using the cavity-bridging method published by Giroud *et al*. (1990) [19]. However, this type of soil reinforcement is expensive. If the possibility of formation of cavities is due to leakage/seepage, it is more cost-effective to minimize leakage/seepage (see Giroud 2022) [2].

4.4.3 Design to control swelling clays

Problems caused by swelling clays are described above in Section 3.2.3. In some parts of India, where swelling soils are frequent, the following practice has been developed. The upper part of the swelling clay is replaced by a layer of compacted "cohesive non-swelling soil" (often referred to as CNS). The cohesive non-swelling soil typically consists of 15-20% non-swelling clay, 30-40% silt, 30-40% sand, and, possibly, some gravel. This soil has a low permeability, a minimum cohesion of 10 kPa and a maximum swelling pressure of 10 kPa.

The configuration is such that the canal lining, at the bottom and on the side slopes of the canal, is underlain by the required thickness of cohesive non-swelling soil. Small canals are excavated in the cohesive non-swelling soil. The required thickness of the cohesive non-swelling soil is selected using standard tables based on the swelling pressure of the swelling clay and the size of the canal. The thickness is typically of 0.5 m to 0.75 m for small canals (flow rate lower than 2 m^3/s) and 0.75 m to 1.0 m for larger canals, depending on the swelling pressure of the swelling clay. It should be noted that a soil thickness of 1 m can only counteract a swelling pressure of 20 kPa, which is lower than the swelling pressure of most swelling clays. Clearly, the role of the cohesive non-swelling soil is essentially to act as a moisture barrier to limit the water content fluctuations in the swelling clay.

As indicated in Kapadia (2018a) [8], since the appropriate way to prevent the detrimental effects of swelling clays is to avoid moisture fluctuations in the swelling clay, the most effective solution is to line the canal with a geomembrane. Indeed, a geomembrane lining limits the risk of swelling by controlling leakage/seepage and limits the risk of shrinkage by preventing evaporation of water from the clay.

4.4.4 Design against geomembrane puncturing

A major cause of the presence of holes in geomembranes is puncturing by stones present at the surface of the supporting soil. If a geomembrane lining is placed on an old concrete or bituminous concrete lining, the geomembrane may be punctured by sharp edges of cracked concrete or by aggregate protruding from the bituminous concrete. Geomembrane selection is essential to ensure resistance to puncturing. As indicated in Sections 1.5 and 4.1.4, geomembrane selection is not addressed in the present paper. However, it is appropriate to mention here the importance of geomembrane thickness regarding resistance to puncture. The polyethylene and PVC films with a thickness of 0.2 to 0.5 mm, which were used since the 1950s and are still used in some irrigation projects, are not adequate because they always exhibit a large number of holes shortly after installation. Geomembranes with a thickness of 0.5 mm have been successfully used in canals when bonded to an adequate nonwoven geotextile at the manufacturing stage. As a general recommendation, geomembranes that are not fabric-reinforced or bonded to a nonwoven geotextile should have a minimum thickness of 0.75 mm. However, the minimum thickness varies with the type of geomembrane, and, for geomembranes that are not reinforced with a woven or a nonwoven, the thicker geomembrane, the less susceptible it is to puncture. Also, in the case of thermoplastic geomembranes such as PVC and polyethylene geomembranes, welding (i.e. seaming by heat) is more difficult, and possibly less reliable, if the geomembrane thickness is less than 1 mm.

The protection of geomembranes against the risk of puncture by stones present at the surface of the supporting soil is routinely addressed by the cushioning effect of a needle-punched nonwoven geotextile placed under the geomembrane. The simplest and most accurate way to select a geotextile used to protect a geomembrane from being punctured by stones is to conduct laboratory tests where the considered geomembrane is subjected to pressure by a fluid (water or air) while being supported by a geotextile placed on a sample of the actual soil and stones from the field. The same test performed without geotextile can be used to evaluate the resistance to puncturing of the geomembrane itself. The criterion for interpreting these tests is discussed in Badu-Tweneboah *et al*. (1998) [20]. Guidance is also provided in ISO [21].

4.4.5 Design detail above the geomembrane top

Precautions should be taken to prevent runoff water from intruding under the geomembrane on the side slopes. Possible situations include: (1) runoff water flooding the anchor trench at the crest of the side slope and, then, flowing under the geomembrane; and (2) in rare cases where the geomembrane lining does not extend all the way to the top of the side slopes, runoff water flowing downslope from the crest may find its way under the geomembrane, depending on how the geomembrane is secured at its top (e.g. inserted in mid-slope anchor bench, or simply ballasted). Precautions include: preventing runoff water from reaching the crest of the side slope; making the crest watertight by extending the geomembrane horizontally on the crest or by covering the crest with a road pavement; and properly securing the top of the geomembrane if it does not extend all the way to the top of the side slopes.

If the geomembrane lining does not extend all the way to the top of the side slopes, protection of the soil above the top of the geomembrane can be achieved with stones or prefabricated concrete panels placed on a geotextile filter, or by using specialized erosion-control geosynthetics (e.g. geomats, geomattresses, geocells). Geosynthetics that promote the development of vegetation, such as geomats and soil-filled geocells, may also be used on canal side slopes, above the water level, for aesthetic and environmental reasons as well as to control erosion. Alternatively, geosynthetics that promote the development of vegetation can be placed on top of the geomembrane to protect the geomembrane; this was done at the Gardanne Canal, in France (Giroud and Plusquellec 2017) [1].

4.5 Design of geomembrane cover

4.5.1 Types of materials used to cover geomembrane linings

The following materials can be used to cover geomembrane linings:

- Concrete layer, either cast in situ or made of prefabricated panels.
- Granular materials such as gravel, cobbles, stones, and rocks.
- Interlocking concrete blocks.
- Concrete blocks linked by cables or bonded to a geotextile, sometimes called articulated concrete blocks.
- Geomattresses filled with concrete, grout, or sand.
- Geocells filled with concrete, gravel, or soil associated with vegetation.
- Geomats associated with soil and vegetation.

In the following sections (4.5.2.to 4.5.10), several aspects of the design of a cover for a geomembrane lining are discussed. The information and methods presented can be used to select and size the cover system used in any specific case. However, it should be noted that the cost and, perhaps more importantly, constructability are often the critical factors for selecting a certain type of cover. Also, there is an increasingly important environmental factor: a cover material with a ‘natural appearance’ (e. g. vegetation, rocks) is increasingly specified as the cover material to be used above the water level.

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4.5.2 Overview of cover design

Reasons for using geomembrane protection are listed above in Section 4.3.2. From these reasons, it appears that a layer covering a geomembrane can perform the following two functions: protecting and ballasting. The design of the cover layer should address the following two requirements: ensuring that the intended functions (i.e. protecting and/or ballasting) are performed; and making sure that the cover layer has no detrimental impact on the geomembrane lining.

Designing to ensure that the intended functions are performed includes two aspects: the cover should be properly sized to perform its function (this is addressed hereafter in Sections 4.5.3 to 4.5.7); and the cover layer should be able to perform its function during the service life of the canal. For example, in the case of a soil layer, it should be checked that it can resist erosion by the flowing water and that it can remain stable during the service life of the canal. Resistance to erosion is addressed hereafter in Section 4.5.8 and cover stability is addressed hereafter in Section 4.5.9.

Finally, design against any detrimental impact that the cover layer could have on the geomembrane lining is addressed in Section 4.5.10.

4.5.3 Design of cover for geomembrane protection

As indicated above in Section 4.3.2, the protection function of the cover consists in: protection against geomembrane deterioration by animals, humans, boats, floating debris, floating ice, falling objects, and hail; protection against geomembrane abrasion by particles in suspension; protection against geomembrane expansion and contraction due to temperature variations; and protection against geomembrane degradation by temperature and solar radiation.

In some cases, the protection function is practically ensured by the sheer presence of the cover layer and, therefore, no specific design is required. For example, the sheer presence of a cover is sufficient against solar radiation. Also, protection against hail is ensured without further design if the cover has the thickness required for other reasons such as ballasting or constructability.

Geomembrane temperature, which is a major cause of concern for exposed geomembranes, may also affect covered geomembranes: the temperature fluctuations induce repeated stresses in the geomembrane as the geomembrane tends to expand and contract while its movements are restricted by the weight and interface friction of the cover; temperature, high or low, affects the mechanical properties of thermoplastics and bituminous geomembranes (e.g. puncture resistance, interface friction angle); and high temperatures accelerate aging of geomembranes. A certain thickness or mass per unit area of cover material is needed to attenuate the effects of temperature variations on the protected geomembrane. Two aspects should be considered: geomembrane temperature generated by solar radiation and geomembrane temperature due to air temperature. High geomembrane temperature due to solar radiation is significantly reduced by the sheer presence of the cover layer. Regarding the required cover thickness to attenuate the effect of air temperature fluctuation, some indication is provided by the following data from Segrestin and Jailloux (1988) [22]: the required thickness of soil cover on top of a geomembrane to ensure that a temperature variation of 30ºC at the soil cover surface results in a variation of only 1ºC at the geomembrane level is 0.15 m for a one hour exposure and 0.5 m for a one day exposure. Clearly, more information is needed on the effect of a cover layer on geomembrane temperature. It should be noted that the presence of water in a canal ensures a quasi-constant temperature to the submerged geomembrane; this is particularly true in the case of canals equipped with modern water management technology where the canal operates most of the time at full capacity.

To select the type and thickness of the cover to protect the geomembrane from damage by falling objects, boats, floating debris and ice, animals and humans, it is possible to perform full-scale tests. Alternatively, it is possible to rely on lessons learned from past applications. In the case of human activities, such as canal operation and maintenance, a concrete cover may be recommended, because, at the same time, it protects the geomembrane and facilitate operation and maintenance activities.

4.5.4 Design for ballasting against geomembrane uplift by wind

The required mass per unit area of ballast for preventing geomembrane uplift by wind, $\mu_{req\ wind}$, is given by the following equation derived from Equation 40:

$$\mu_{req\,wind} = 0.005\ v_{wind}^2 \ \text{ with } \mu_{req\,wind}\left(\text{kg/m}^2\right) \text{ and } v_{wind}\left(\text{km/h}\right) \tag{41}$$

As indicated in Section 3.5.2, Equation 40, and, therefore, Equation 41, are applicable to the upper part of the side slopes. The required mass per unit area of ballast at the bottom of the canal is lower than calculated with Equation 41.

For example, for a wind velocity of 100 km/h, the required mass per unit area of ballast is 50 kg/m^2, which corresponds to 50 mm of water and, approximately, 30 mm of gravel or 20 mm of concrete. Thus, 30 mm of gravel on a geomembrane is sufficient to prevent uplift by a 100 km/h wind, and 100 mm of concrete is sufficient to prevent uplift by a 220 km/h wind. It should be noted that ballasting with water at the bottom of a canal is effective only if the actual depth of water is significantly greater than the calculated value to account for the fact that a fraction of the water is likely to be displaced by the wind.

Clearly, wind uplift is avoided by practically all types of cover materials discussed in the following sections.

4.5.5 Design for ballasting against geomembrane uplift by dynamic pressure

As explained in Section 3.4.1, a geomembrane can be uplifted by the dynamic pressure due to flowing water. The uplift pressure due to the dynamic pressure expressed by Equation 24, is used here with $\alpha_{dyn} = 1$ for conservativeness. This uplift pressure should be counteracted by the weight per unit area of the cover. The risk of geomembrane uplift by dynamic pressure occurs when there is water in the canal. Therefore, the ballasting effect of the cover material involves the buoyant weight of the cover material. It is important to note that the static pressure applied by the water on the geomembrane does not contribute to the stabilization of the geomembrane, because this static pressure is already accounted for in the calculation of the dynamic pressure, as shown in Section 3.4.1. Therefore, the ballasting contribution of the cover material can be expressed as follows:

$$\sigma_{abv} = \rho_{buoy}\ g\ h_b \tag{42}$$

where ρ_{buoy} is the buoyant density of the ballast material and h_b is the ballast thickness. Note: the thickness and mass of the geomembrane are negligible in this case and are not considered.

Using the classical relationship between buoyant density and porosity of the ballast material, the above equation becomes:

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$$\sigma_{abv} = (1 - n_b)(\rho_s - \rho_w)\, g\, h_b \tag{43}$$

where n_b is the ballast material porosity and ρ_s is the density of the ballast particles.

Balancing the vertical stress exerted by the ballast and the dynamic pressure:

$$(1 - n_b)(\rho_s - \rho_w)\, g\, h_b = \frac{\rho_w\, v^2}{2} \tag{44}$$

Hence the required ballast thickness:

$$h_{b\,req} = \frac{\rho_w\, v^2}{2\, g (1 - n_b)(\rho_s - \rho_w)} \tag{45}$$

For a granular soil material (e.g. gravel, cobbles, stones), ρ_s is typically 2700 kg/m^3, hence:

$$h_{b\,req} = \frac{0.03\ v^2}{1 - n_b} \text{ with } h_{b\,req}(\text{m}) \text{ and } v(\text{m/s}) \tag{46}$$

Hence, for a conservative value of the porosity of 0.4:

$$h_{b\,req} = 0.05\ v^2 \text{ with } h_{b\,req}(\text{m}) \text{ and } v(\text{m/s}) \tag{47}$$

In the case of concrete ballast, the porosity is quasi zero and the density is 2400 kg/m^3, hence from Equation 45:

$$h_{b\,req} = 0.036\ v^2 \text{ with } h_{b\,req}(\text{m}) \text{ and } v(\text{m/s}) \tag{48}$$

For example, for a flow velocity of 1.2 m/s, the required ballast thickness is 72 mm in case of gravel ballast and 52 mm in the case of concrete ballast. No factor of safety may be needed since uplift by dynamic pressure is an extreme situation. It should be noted that a greater thickness of ballast may be required if the geomembrane is subjected to other risks of uplift as discussed in the following sections.

4.5.6 Design for ballasting against groundwater

If the groundwater level is higher than the water level in the canal, the geomembrane lining is uplifted, unless there is a layer of ballast on the geomembrane. The required thickness of ballast can be calculated as follows. The calculation presented below is for the ballast at the bottom of the canal.

The vertical stress on top of the geomembrane can be expressed as follows:

$$\sigma_{abv} = \rho_{buoy}\, g\, h_b + \rho_w\, g\, d_w \tag{49}$$

where ρ_{buoy} is the buoyant density of the ballast material and h_b is the ballast thickness. Note: the thickness and mass of the geomembrane are negligible in this case and are not considered.

Using the classical relationship between buoyant density and porosity of the ballast material, the above equation becomes:

$$\sigma_{abv} = (1-n_b)(\rho_s - \rho_w) g\ h_b + \rho_w\ g\ d_w \tag{50}$$

where n_b is the ballast material porosity and ρ_s is the density of the ballast particles.

The vertical stress below the geomembrane, which is due to groundwater, can be expressed as follows:

$$\sigma_{blw} = \rho_w\ g\ h_{gw} \tag{51}$$

where h_{gw} is the height of groundwater above the geomembrane level.

The geomembrane is not uplifted by groundwater if the vertical stress above the geomembrane is higher than the vertical stress below the geomembrane, hence the required thickness of the ballast:

$$h_{b\,req} = \frac{h_{gw} - d_w}{(1-n_b)\left(\dfrac{\rho_s}{\rho_w} - 1\right)} \tag{52}$$

It is well known that most soil particles have a density approximately 2.7 times the density of water. Assuming conservatively a porosity of 0.4 for the gravel or stone ballast, Equation 52 becomes:

$$h_{b\,req} = \frac{h_{gw} - d_w}{(1-0.4)(2.7-1)} \approx h_{gw} - d_w \tag{53}$$

In the case of concrete ballast, the porosity can be neglected and Equation 52 becomes:

$$h_{b\,req} = \frac{h_{gw} - d_w}{\left(\dfrac{\rho_{concrete}}{\rho_w} - 1\right)} = \frac{h_{gw} - d_w}{1.4} \tag{54}$$

where $\rho_{concrete}$ is the density of concrete, typically 2.4 times the density of water.

For example, if the groundwater level is 0.5 m above the water level in the canal, the required thickness of ballast at the canal bottom is 0.5 m in case of granular ballast (e.g. gravel or stones) or 0.35 m in case of concrete ballast. These ballast thicknesses are not acceptable from a cost standpoint. Furthermore, they would significantly reduce the canal cross section available for flow. Clearly, ballasting is generally not a viable solution against geomembrane uplift by groundwater.

As indicated above in Section 4.2.1, a possible solution against the risk of geomembrane uplift by groundwater is to adopt a different route and/or a different elevation for the canal. Another possibility is to use a pressure-relief valve, a device that allows groundwater to intrude into the canal when the groundwater level is higher than the water level in the canal, thereby preventing geomembrane uplift, and does not allow water from the canal to leak into the ground. Commercially available pressure-relief valves are generally designed to be used in an industrial facility and are too delicate to be used in a canal, in part because of the high risk of clogging by silt particles. It is preferable to use a simple system which consists of a

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hole in the geomembrane covered with a geomembrane flap, with the upstream side of the flap attached to the geomembrane lining. The hole should be circular with a smooth edge to avoid initiating a tear of the geomembrane. It should be noted that there should be no ballast on the pressure-relief valve or the geomembrane flap to allow them to function should be able to function if the groundwater table is rising.

As indicated in Sections 3.3.4 and 4.3.1, if there is no viable solution with a lining, it is possible that the best solution is not to line a canal in an area where high groundwater is expected.

4.5.7 Design for ballasting and protection against turbulence

Considering the risk, confirmed by field observations, of repeated excessive stresses on exposed geomembranes caused by flow turbulence generated by cross-flow obstacles such as gates, weirs and bridge piers, it is recommended to cover geomembrane linings by a ballasting/protective layer over a certain length of canal downstream of these obstacles. The length of canal that requires protection and ballasting should be determined by an evaluation of localized flow velocity and turbulence by a hydraulic analysis performed by a competent specialist of hydraulics engineering.

The selection of the type and mass per unit area of the protective-ballasting cover to be used in this situation is complex. Evaluating the forces involved in the dynamic mechanisms that tend to displace a geomembrane (drag forces, dynamic pressure, flow turbulence) and the cover material is a complex exercise because forces are in different directions and the flow turbulence generates repeated stresses. To the best knowledge of the authors of the present paper, there is no method to evaluate these repeated stresses and to evaluate their consequences. In this case, ballasting has a positive effect that is difficult to quantify. Clearly, this is an area where research is needed. From a practical standpoint, Equations 42 to 45 proposed for the determination of ballast thickness to counteract geomembrane uplift by dynamic pressure could be tentatively used with the maximum localized flow velocity due to turbulence determined by hydraulic analysis.

4.5.8 Design to control erosion by flowing water

It is necessary to ensure that the material used to cover the geomembrane lining is able to resist the erosive action of the flowing water. The mechanism of erosion and the resistance of different materials to erosion have been the subjects of extensive research and numerous publications for decades. In this section, simple equations are proposed, which were obtained theoretically by combining equations for drag stresses, for flow velocity and for gravity. These equations are only intended for approximate evaluation of the risk of erosion at the preliminary design stage.

In the case of a granular cover layer, balancing the driving force exerted by the drag stress on cohesionless particles and the resisting force due to the weight of the particle after several assumptions and simplifications lead the first author of the present paper to propose the following approximate equations:

$$\tau_{eros} \approx 0.6\,D \;\text{ with } \tau_{crit}\left(\text{Pa}\right) \text{ and } D\left(\text{mm}\right) \tag{55}$$

$$v_{eros} \approx \sqrt{\frac{D\ d_w^{1/3}}{10}} \text{ with } \mathrm{v}_{crit}\left(\text{m/s}\right), D\left(\text{mm}\right) \text{ and } d_w\left(\text{m}\right) \tag{56}$$

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where τ_{eros} is the drag stress that causes erosion by displacing particles with a diameter D, v_{eros} is the flow velocity that causes erosion by displacing particles with a diameter D, and d_w is the depth of water. In these equations, the particles are assumed to have the typical density of soil particles of 2700 kg/m^3. Calculated drag stresses and flow velocities calculated using the preceding equations are presented in Table 2.

Table 2. Calculated approximate drag stresses and flow velocities (for a water depth of 1 m) that cause erosion as a function of the particle size for a cohesionless granular cover material.

Particle size	D (mm)	1	2	5	10	20	30	50	100
Erosion drag stress	τ_{eros} (Pa)	0.6	1.2	3.0	6.0	12	18	30	60
Erosion flow velocity	v_{eros} (m/s)	0.3	0.4	0.7	1.0	1.4	1.6	2.2	3.1

While the cover material discussed above is cohesionless, the various materials mentioned below form a cohesive layer because they are intrinsically cohesive, like concrete, or because they consist of cohesionless material mechanically bonded for example by cables, wire mesh or geocells. Data presented below are adapted from several sources, in particular results of full-scale tests published by Escarameia (1998) [23]. Typical acceptable flow velocities for various cover materials are approximately:

- Concrete lining, 6 to 7 m/s, and probably more.
- Concrete-filled geomattresses, 5 to 7 m/s.
- Concrete blocks linked with cables, 5 m/s if heavier than 250 kg/m^2.
- Concrete blocks (interlocking but not linked with cables), 2 m/s (if 150 to 175 kg/m^2) and 1.5 m/s (if 100 to 150 kg/m^2).
- Geocell filled with concrete, 6 to 7 m/s (even 11 m/s, from one source, for a thickness of 75 mm and a mass per unit area of 180 kg/m^2).
- Geocell filled with gravel 2 to 3.5 m/s.
- Geocell filled with grass, 4 m/s, but for only a few hours.
- Gabions, 6 m/s if 1 m thick.
- Gabion mattresses, 5 m/s if 0.5 m thick; 4 m/s if 0.3 m thick; 3 m/s if 0.15 m thick.
- Geomat associated with bitumen-bound gravel and grass 2.5 to 5 m/s;
- Geomat alone 1 m/s.

When designing a cover for a geomembrane lining, the acceptable flow velocity for the cover (from the above list or from other sources) should be greater than the maximum flow velocity expected in the considered canal, taking turbulence into account. The above values are only indicative and should only be used for material selection at the preliminary design stage. Collecting additional information from material suppliers and performing full-scale tests are recommended at the detailed design stage. It is important to note that calculations presented in the following section show very high factors of safety against displacement of most types of cover systems by flowing water. Therefore, the selection of a cover system is generally not governed by its ability to resist flow velocity and is more likely to be governed by other considerations such as cost or constructability.

In this section 4.5.8, the considered effect of flow velocity is progressive deterioration of the cover material by erosion. The following section is also concerned with flow velocity: the risk of displacement of the cover layer by the drag force.

4.5.9 Design for stability of the cover layer against water flow

There are two aspects concerning the stability of the cover layer: the cover layer should not slide in the direction of the flow under the effect of the drag forces; and the cover layer should not slide along the side slopes under the effect of gravity. It is important to verify that the cover layer is stable for two reasons: to ensure that the cover will perform its function and to ensure that the geomembrane will not be damaged by any sliding of the cover layer.

The factor of safety against sliding in the direction of the flow is calculated by dividing the resisting stress by the driving stress. The maximum driving stress is the maximum drag stress, which occurs at the bottom of the canal and at the toe of the side slopes. The resisting stress, which is generated by the weight of the cover material, is lower at the toe of the side slopes than at the canal bottom because, on the slope, only a component of the weight is effective, hence the factor cosβ. Therefore, the resisting force can be expressed as follows:

$$F_{resist} = (1-n)(\rho_s - \rho_w)g\ h_c \cos\alpha\ \cos\beta\ \tan\delta \tag{57}$$

where n is the porosity of the cover material, ρ_s is the density of the solid particles of the cover material, ρ_w is the density of water, h_c is the thickness of the cover material, α is the angle of the longitudinal slope of the canal, β is the side-slope angle, and δ is the interface friction angle of the interface located beneath the cover that has the lowest friction angle.

The factor of safety can then be calculated using the following equation obtained by dividing the resisting stress (Equation 57) by the driving stress (Equation 11) and eliminating the gravitational acceleration:

$$FS_{drag} = \frac{(1-n)(\rho_s - \rho_w) h_c \cos\alpha\ \cos\beta\ \tan\delta}{\rho_w\, d_w\, S} \tag{58}$$

where d_w is the depth of water in the canal and S is the longitudinal slope of the canal (in fact, S = tanα).

If the bottom cover is lighter than the slope cover, a second factor of safety should be calculated using the following equation (similar to the preceding equation but with β = 0):

$$FS_{drag\,bottom} = \frac{(1-n)(\rho_s - \rho_w) h_{c\,bottom} \cos\alpha\ \tan\delta}{\rho_w\, d_w\, S} \tag{59}$$

In the case of a granular cover, ρ_s / ρ_w = 2.7, and, if n = 0.4, the above equations become:

$$FS_{drag} = \frac{h_c \cos\alpha\ \cos\beta\ \tan\delta}{d_w\, S} \tag{60}$$

$$FS_{drag\,bottom} = \frac{h_{c\,bottom} \cos\alpha\ \tan\delta}{d_w\, S} \tag{61}$$

In the case of a concrete cover, ρ_s / ρ_w = 2.4 and n = 0. The above equations become:

$$FS_{drag} = \frac{1.4\ h_c \cos\alpha\ \cos\beta\ \tan\delta}{d_w\, S} \tag{62}$$

$$FS_{drag\,bottom} = \frac{1.4\ h_{c\,bottom} \cos\alpha\ \tan\delta}{d_w\, S} \tag{63}$$

In most canals, the longitudinal slope is very small and, therefore, cosα ≈ 1. More importantly, S = tanα being very small, the factor of safety is very large. For example, Equation 62 gives a factor of safety of 349 for a 0.1 m thick concrete cover on the 1.5H:1V side slope of a canal with a longitudinal slope of 0.1 m per km (i.e. $S = 1 \times 10^{-4}$), under 1 m of water and with tanδ = 0.3. It is only in the case of steep chutes or spillways that the factor of safety is an issue.

4.5.10 Design for stability of the cover layer against gravity

After the stability of the cover against drag stresses, discussed above, stability against gravity of the cover on the side slopes is the second of the two aspects of cover stability indicated at the beginning of Section 4.5.9. In this case, both the driving stress and most of the resisting stress are generated by gravity. Therefore, the mass (and consequently the thickness) of the cover layer do not appear in the following approximate expression of the factor of safety:

$$FS_{gravity} \approx \frac{\tan\delta}{\tan\beta} \tag{64}$$

where δ is the interface friction angle of the interface located beneath the cover that has the lowest friction angle.

A more accurate evaluation of slope stability takes into account the fact that the resisting stresses depend not only on the interface friction angle, δ, but also on the toe buttressing effect (which may not be negligible if the side slopes are relatively short) and on the resistance of the anchorage at the crest of the slope (which is significant in the case of some geosynthetic systems). These two additional contributions to the safety of covers on side slopes are quantified by the following equation provided in Giroud *et al*. (1995b) [24]:

$$\begin{aligned} FS_{gravity} = {} & \frac{\tan\delta}{\tan\beta} + \frac{a}{\rho_c\, g\, h_c \sin\beta} + \left(\frac{h_s}{H}\right)\left[\frac{\tan\phi\left(2\sin\beta\cos^2\beta\right)}{1-\tan\beta\tan\phi}\right] \\ & + \left(\frac{c}{\rho_c\, g\, H}\right)\left[\frac{1/\left(\sin\beta\cos\beta\right)}{1-\tan\beta\tan\phi}\right] + \frac{T_{anchorage}}{\rho_c\, g\, H\, h_c} \end{aligned} \tag{65}$$

where a is the interface adhesion of the interface located beneath the cover that has the lowest adhesion, ϕ is the internal friction angle of the cover material, c is the cohesion of the cover material, H is the height of the slope, and $T_{anchorage}$ is the tensile force resisted by the anchorage at the crest of the slope.

Equation 61 provides a conservative evaluation of the factor of safety (i.e. a lower boundary of the factor of safety) because it does not account for the two other contributors to the resisting stress.

In the case of cast-in-situ concrete, stability during placement is a challenge. The slump index of the concrete should be carefully selected. A thick needle-punched nonwoven geotextile placed on the geomembrane lining and properly anchored at the crest of the slope should be used to enhance the stability of the fresh concrete by draining the small quantity of excess water from the concrete as it is being placed (especially if the concrete is compacted by vibrators).

In the case of soil covers, the critical situation for stability is rapid drawdown of water. The rapid drawdown situation has a high probability of occurrence in canals because it is possible to rapidly interrupt the flow of water (especially in the case of on-off irrigation, defined in Section 3.2.3). Rapid drawdown is a classical situation in dam design, and methods to evaluate the stability of the upstream face of earth dams in case of rapid drawdown have been published. Giroud and Ah-Line (1984) [25] have proposed a demonstration showing that there is a risk of instability due to rapid drawdown if the rate of water level decrease, v_{draw}, exceeds the following value:

$$v_{draw\,max} = k \sin^2\beta \tag{66}$$

where $v_{draw\,max}$ is the rate of water level decrease above which there is a risk of rapid drawdown, and k is the hydraulic conductivity of the cover soil. Numerical values of $\sin^2\beta$ are: 0.1 for 3H:1V slope, 0.2 for 2H:1V slope and 0.3 for 1.5H:1V slope. For example, if the hydraulic conductivity of the soil cover is 1×10^{-5} m/s (silty soil) and the canal side slope is 1.5H:1V, then the rate of drawdown should be lower than 3×10^{-6} m/s, that is 0.26 m/day (water level drop measured vertically). In the case of a less permeable soil, the rate of water level decrease should be lower.

The factor of safety for soil cover stability in case of water drawdown can be calculated using equations published by Giroud *et al*. (1995 c) [26]. If necessary, the soil cover can be reinforced using a geogrid, which was done at Tekapo Canal in New Zealand, as described in: Eldridge *et al*. (2013) [27]; Giroud *et al*. (2013) [28]; Jacka *et al*. (2013) [29]; and Scuero *et al*. (2022) [30].

For the sake of completeness, it should be noted that it may seem appropriate to consider that the cover layer instability may result from the simultaneous action of drag stresses and gravity stresses. Therefore, Equation 58 for stability with respect to drag stress can be combined with the equation by Giroud *et al.* (1995b) [24] for stability with respect to gravity. In the resulting complex equation, the terms related to the effect of drag stress are negligible, as evidenced by the fact that the factor of safety for stability against drag stress is very large as shown in Section 4.5.9. As a result, the complex equation boils down to Equation 65, or even Equation 64.

4.5.11 Design against detrimental impact of the cover layer

It is important to ensure that the cover layer (whether it is used for protection or ballasting) does not cause mechanical damage to the geomembrane. In other words, the lining should be protected from its protective layer. Mechanical damage to the geomembrane by the cover material may occur during construction and when the canal is in service.

Mechanical damage can be caused to the geomembrane during placement of the cover material due to: activities of the installing crew (e.g. walking on the geomembrane with inadequate shoes, dragging hoses or equipment, dropping tools, etc.); construction equipment driven on the geomembrane; backhoe tearing the geomembrane; etc. Instability of the cover layer material during construction (see Section 4.5.10) can cause severe damage to the geomembrane lining. In fact, construction activities associated with the placement of cover material on a geomembrane are known to be the main cause of holes in geomembrane linings.

When the canal is in service, the cover may damage the geomembrane lining by applying normal stresses or tangential stresses.

The normal stresses applied by the cover on the geomembrane are relatively low, because the cover layer is not thick and it is submerged, therefore, acting only with its buoyant weight. However, it is necessary to prevent puncturing of the geomembrane by stones present in the cover material, by corners of precast concrete panels, and by sharp edges of the cracks of a cracked concrete cover. This is ensured by the use of an adequate needle-punched nonwoven geotextile between the geomembrane and the cover layer.

Tangential stresses can cause significant mechanical damage to the geomembrane lining if the cover layer is unstable, i.e. if it moves with respect to the geomembrane. (See Sections 4.5.9 and 4.5.10 for cover layer stability.) As discussed in Section 4.5.9, instability of the cover layer caused by drag stresses applied by flowing water is unlikely. Therefore, the potential cause of cover instability is gravity, i.e. slope instability. It is, therefore, important to check that the cover layer is stable on canal side slopes and, therefore, does not move with respect to the geomembrane. Mechanical damage caused to the geomembrane by a moving cover layer can be severe if stones are protruding from the cover material, Also, a geomembrane lining can be mechanically damaged by edges of cracks if it is covered by a cracked concrete layer that moves back and forth due to cycles of thermal expansion and contraction when the canal is empty. A geotextile placed between the cover layer and the geomembrane lining helps alleviate stresses in the geomembrane.

4.6 Design of exposed geomembrane linings

4.6.1 The challenges associated with the design of exposed geomembranes

By definition, exposed geomembranes are not protected by a cover layer. Therefore, exposed geomembranes are susceptible to mechanical damage, and they are in direct contact with the flow of water. The causes of mechanical damage are presented in Section 3.6 and the protection of a geomembrane lining against mechanical damage is discussed in Section 4.5.3. Most of this Section 4.6 is devoted to design measures against the detrimental effects of flowing water on the geomembrane.

The above-water portion of exposed geomembranes is subjected to weathering and temperature changes. These conditions have an influence on the long-term performance of the geomembrane. This important subject is beyond the scope of the present paper.

A geomembrane lining, if not uplifted, resists drag stresses as discussed above in Section 2.2.5. However, in a canal, there is always a risk of geomembrane lining uplift (e.g. by high groundwater or by dynamic pressure of water from the canal that has leaked through the lining and has accumulated under the lining). An uplifted geomembrane can be displaced downward by drag stresses and, more importantly, by the dynamic pressure exerted by the flowing water on the front of the uplifted geomembrane, as discussed above in Section 2.3.5. Therefore, an exposed geomembrane should be anchored, and the design of anchors is addressed in this Section 4.6. It is important to note that an exposed geomembrane can be uplifted between anchors. It is, therefore, important to check that the strains in a geomembrane uplifted between anchors are acceptable for the type of geomembrane used.

Section 4.6.2 discusses longitudinal anchorage and Section 4.6.3 discusses transverse anchorage. Section 4.6.4 discusses punctual anchorage of geomembranes. A multitude of punctual anchors may be used to replace or supplement longitudinal and transverse anchorage. Section 4.6.5 discusses the complex stress and strain field in geomembrane linings exposed to flowing water.

Exposed geomembranes may cause safety problems, which are discussed in Section 4.6.6.

Exposed geomembranes may exhibit wrinkles and design engineers should be aware of this possibility. Wrinkles are discussed in Section 2.3.2.

4.6.2 Design of longitudinal anchorage

If the geomembrane lining is resting on soil, longitudinal anchorage is typically achieved by anchor trenches or anchor benches located generally at the crest of side slopes or, sometimes, below the crest of the side slopes. Anchorage in anchor trench at the toe of side slopes is often used in addition to the anchorage at the crest of the side slopes.

If the geomembrane lining is resting on concrete in good condition, longitudinal anchorage can be achieved by steel batten strips, placed at the top of the geomembrane lining and, also, at other levels, as needed, in particular at the toe of the side slopes. This was done, for example at Pernegg Canal in Austria. In the case of canals lined with a PVC geomembrane thermally bonded to a nonwoven geotextile installed on an existing concrete lining in good condition, the longitudinal anchorage may be achieved using geomembrane tensioning profiles that keep the lining flat. This was done for example, at Kootenay Canal in Canada.

Longitudinal anchorage of the geomembrane lining can be achieved by ballast both at the bottom of the canal and at the crest of the side slopes, leaving a portion of the side slopes with the geomembrane exposed to ensure high flow velocity. This was done at Tekapo Canal in New Zealand, as described in: Eldridge *et al*. (2013) [27]; Giroud *et al*. (2013) [28]; Jacka *et al*. (2013) [29]; and Scuero *et al*. (2022) [30].

Longitudinal anchorage of the geomembrane lining can be achieved by a combination of ballast at the bottom of the canal and anchorage (batten strips, anchor trenches or benches) at the crest of the side slopes. This was done, for example, at the 50-year old Canal de Provence, in France, a concrete-lined canal (still in excellent condition from a structural standpoint but with cracks), where a section has been rehabilitated using a bituminous geomembrane covered with a 100 mm thick concrete layer at the bottom of the canal and anchored at the crest of the two side slopes using batten strips plus concrete slabs acting as anchor benches. This example shows that the right time to rehabilitate a concrete lining with a geomembrane is when the concrete exhibits cracks, which justify the rehabilitation, and when, at the same time, the concrete is structurally sound, which provides a uniform base to support the geomembrane and facilitates its anchorage.

In all the cases mentioned above, the geomembrane lining is anchored at the crest of the side slopes and at other locations. However, in a number of small canals, the geomembrane lining is only anchored at the crest of the side slopes. It should be noted that anchorage at the crest of the side slopes is generally considered indispensable to ensure the stability of the geomembrane lining during construction. However, there is a notable exception. No anchor trench was used in the main canal of the Toshka Project in Egypt, even though the 2H:1V side slopes are 7.5 m high. This was possible because the geomembrane was exposed during a very short time, as the geomembrane was covered with concrete shortly after seaming the geomembrane panels. This very short time reduced the possibility of a wind gust that would displace the geomembrane and the possibility of a significant temperature increase that would cause the formation of wrinkles in the HDPE geomembrane. As a result of not using an anchor trench, 700,000 square meters of geomembrane were saved, i.e. 3.5% of the total amount of 20 million square meters of geomembrane used in that project (Yazdani 2005, 2012) [31, 32]. What was possible with a geomembrane exposed only temporarily would not have been possible with a geomembrane designed to be permanently exposed. To conclude this discussion, it is clear that: (1) anchorage at crest of side slopes (by anchor trench, anchor bench, batten strips, etc.) is required only in the case of exposed geomembranes linings and in the case of geomembranes that are intended to be covered but where the placement of the cover takes place a long time after the placement of the geomembrane; and (2) in large canals and in canals, or stretches of canals, of any type subjected to high flow velocity, it is recommended to also anchor exposed geomembrane linings longitudinally at the toe of the side slopes or to ballast the entire canal bottom.

After the foregoing discussion on the practice of longitudinal anchorage, equations for the sizing of longitudinal anchors are presented. Considering that the effect of drag stresses is not significant, only uplift by dynamic pressure exerted by the flowing water is considered here. The upward force per unit length of anchor trench located at the toe of side slopes can be evaluated using the following equation:

$$F/L = \frac{1}{2}\left(\frac{1}{2}\rho_w\, v^2\, B_{invert} + \frac{1}{2}\rho_w\, v^2\, L_{slope}\cos\beta\right) \tag{67}$$

where B_{invert} is the width of the canal bottom and L_{slope} is the length of the side slope. The factor ½ is used because it is assumed that there is a total of four anchor trenches, i.e. one at the toe and one at the crest of each side slope. Therefore, an anchorage system (e.g. anchor trench or batten strip) at the slope toe should resist half the uplift forces exerted on the geomembrane at the canal bottom and the adjacent side slope.

Equation 67 can be written as follows:

$$F/L = \frac{\rho_w\, v^2}{4}\left(B_{invert} + L_{slope}\cos\beta\right) \tag{68}$$

For a flow velocity of 2 m/s, a bottom width of 3 m, a slope length of 4 m and a 1.5H:1V slope ($\cos\beta = 0.832$), Equation 68 gives an upward force per unit length of the toe anchor trench of 6,328 N/m. An anchor trench filled with concrete with a 0.36 m^2 cross section (i.e. 0.6 m × 0.6 m) has a weight per unit length of approximately 8,400 N/m, which is higher than the calculated upward force per unit length of 6,328 N/m.

It should be noted that the method expressed by Equation 68 for the slide toe anchorage in only approximate, because it does not account for the inclination of the uplifted geomembrane tension (which was possible because the resultant of the uplift forces is approximately vertical). A more rigorous method can be used for large canals, which was done, for example, for Tekapo Canal. In the case of the crest anchorage, it is necessary to use a design method that takes into account the inclination of the force, which depends on geomembrane strain.

It is important to check that the strain of the geomembrane is acceptable. An approximate value of the geomembrane strain can be calculated using the following equation proposed by Giroud (2009) [13] and adapted to uplift by dynamic pressure:

$$\varepsilon_{approx} \approx \frac{0.3467\left(\dfrac{\rho_w\, v^2\, B_{uplift}}{2E\, t_{GMB}}\right)^{2/3}}{1 - 0.3103\left(\dfrac{\rho_w\, v^2\, B_{uplift}}{2E\, t_{GMB}}\right)^{2/3}} \approx 0.35\left(\frac{\rho_w\, v^2\, B_{uplift}}{2E\, t_{GMB}}\right)^{2/3} \tag{69}$$

where E is the tensile modulus of the geomembrane, and B_{uplift} is the width of geomembrane subjected to uplift.

For a flow velocity of 2 m/s, a width of geomembrane subjected to dynamic pressure of 3 m, a geomembrane modulus of 400 MPa (HDPE geomembrane) and a geomembrane

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thickness of 1.5 mm, the calculated strain is 1.6%, which is acceptable considering a typical allowable strain of 3% for HDPE geomembranes.

4.6.3 Design of transverse anchorage

The purpose of transverse anchorage is to prevent the geomembrane lining, located downstream of the anchorage, from being displaced in the downstream direction by flowing water. The two mechanisms that can displace a geomembrane are: the dynamic pressure applied by the flowing water on the front face of an uplifted geomembrane; and the drag stresses, which are not high but act over the entire length of the geomembrane lining between two transverse anchors.

Transverse anchorage can be achieved by regularly spaced anchor trenches, each anchor trench being excavated in the bottom and the side slopes of the canal. For example, at the Dionysen Canal, a hydroelectric canal in Austria, the geomembrane is secured by concrete-filled transverse anchor trenches every 15 m in addition to longitudinal concrete beams at the crest of the side slopes and insertion beneath the edges of a bituminous concrete lining at the bottom. If a geomembrane lining is placed on an existing concrete lining, transverse anchorage can be achieved by batten strips.

Regularly spaced transverse anchors have an advantage: they limit the area to be repaired in case of geomembrane lining failure. If only a stretch of canal is lined with a geomembrane and the rest of the canal is unlined, there must be a transverse anchor trench at each end of the lined stretch.

The force per unit width applied by the flowing water on a transverse anchor trench or batten strip can be evaluated using the following equation.

$$F/B = \frac{1}{2}\rho_w v^2 H_{uplift} + \frac{\rho_w g\, n_M^2 v^2}{d_w^{1/3}} L_{stretch} \tag{70}$$

where F/B is the force per unit width applied by the flowing water on a transverse anchor trench or batten strip, H_{uplift} is the height above the canal bottom of the uplifted geomembrane, and $L_{stretch}$ is the length of canal stretch between two transverse anchor trenches.

The first term on the right part of the equation corresponds to the action of the dynamic pressure applied by the flowing water on the front of the uplifted geomembrane (based on Equation 24), and the second term corresponds to the action of the drag stresses on the geomembrane (based on Equation 16). It should be noted that the dynamic pressure acts on the front of the uplifted geomembrane immediately downstream of the anchor trench whereas the drag stresses act over the entire length of the stretch of canal between two transverse anchor trenches. Equation 70 can be written as follows:

$$F/B = \rho_w v^2 \left(\frac{1}{2} H_{uplift} + \frac{g\, n_M^2}{d_w^{1/3}} L_{stretch} \right) \tag{71}$$

A numerical application for a flow velocity of 2 m/s, an uplift height of 1 m, a water depth above the geomembrane of 1 m, a stretch length of 50 m, and a Manning's coefficient of 0.013 for the geomembrane, gives:

$$F/B = 1000 \times 2^2 \left(\frac{1}{2} \times 1 + \frac{9.81 \times 0{,}013^2}{1} \times 50 \right) = 4000\left(0.5 + 0.083\right) = 2{,}332\ \text{N/m} \tag{72}$$

In this case, as in many cases, the action of drag stresses is small compared to the action of the dynamic pressure.

The force per unit width thus obtained should be resisted by the anchor trench. A concrete-filled anchor trench with a 0.25 m cross section (i.e. 0.5 m × 0.5 m) has a weight per unit length of approximately 5800 N/m. With an interface friction angle of 22 º at the base of this anchor trench, a horizontal force of 2340 N/m would be generated. This does not take into account the passive pressure of soil on the downstream side of the anchor trench, which would add significantly to the resisting force. Even though a more sophisticated geotechnical design of the anchor trench would be required, the simple calculation presented above shows that a simple concrete-filled anchor trench every 50 m is able to resist the stresses generated by a flow velocity of 2 m/s, if the geomembrane is uplifted by one meter. It should be noted that the geomembrane is also likely to be anchored longitudinally, as discussed in Section 4.6.3, which reduces the stresses transmitted to the transverse anchor trenches. This numerical example indicates that, in some cases, a 50 m distance between transverse anchor trenches is adequate and leads to a lower construction cost than a shorter distance such as 15 m, as in the canal mentioned at the beginning of this Section 4.6.3. It should also be noted that, in the case of geomembrane failure, the cost difference between repairing 50 m or 15 m of geomembrane lining is not excessive.

It is important that the connection between the geomembrane lining and transverse anchor trenches be done with great care. A defective connection at this location could allow water to enter under the geomembrane and would uplift it in accordance with the mechanism of dynamic pressure buildup described in Section 3.4. This is particularly important if the geomembrane lining is underlain by a low-permeability material, such as an old concrete lining. Indeed, in this case, dynamic pressure buildup under the geomembrane lining is facilitated because the water that accumulates under the geomembrane lining is not drained, as indicated in Section 3.4.3.

The design of transverse anchor trenches should also include a verification of the ability of the geomembrane to resist the tension applied to the anchor trenches. For example a 1.5 mm thick HDPE geomembrane has a tensile strength at yield of 22 kN/m and a 2.5 mm thick PVC geomembrane bonded to a needle-punched nonwoven geotextile with a mass per unit area of 500 g/m^2 has a tensile strength at break of approximately 25 kN/m. Clearly, these geomembranes, which are typically used in canals, have a tensile strength significantly higher than the 2,332 N/m tension calculated above. However, in case of geomembrane uplift, the stresses are not uniformly distributed and large stress concentration may exist, which could tear a geomembrane.

4.6.4 Design of punctual anchorage

As pointed out in Section 4.6.1, a multitude of punctual anchors may be used to replace or supplement longitudinal and transverse anchorage. The design of punctual anchors is addressed below.

Punctual anchorage can be achieved by driving nails through the geomembrane and into the underlying concrete, or driving ground anchors through the geomembrane and into the underlying soil. Hereafter, the terms ‘punctual anchor’ or ‘anchor’ are generically used for both nails and ground anchors. At each anchor, a rigid cap with a sufficient diameter is used to minimize stress/strain concentration at the geomembrane/anchor connection. The cap must be sealed at its periphery to ensure a watertight connection with the geomembrane.

In the field, the pullout resistance of each anchor can be evaluated using a special pullout tester prior to attaching the geomembrane to it. At the design stage, it is necessary to have data on the pullout resistance of the anchors related to the type of material underlying the geomembrane. These data (which are often provided by the supplier of anchors) can be obtained from experience gained on past projects or from full scale tests.

The required pullout resistance depends on the expected uplift pressure divided by the number of anchors per unit area. The expected uplift pressure can be, for example, the wind suction or the dynamic pressure of water. The available pullout resistance (which should be greater than the required pullout resistance) is the minimum of the anchor pullout resistance from the ground or concrete and its pullout resistance from the geomembrane after sealing the cap to the geomembrane. The required number of anchors is given by the following equation:

$$N = \frac{p_{uplift}}{F_{pullout}} \tag{73}$$

where N is the number of anchors per unit area, p_{uplift} is the uplift pressure, and $F_{pullout}$ is the available pullout resistance of the anchor.

In the case of a square pattern of anchors, the required distance between anchors, d_a, can be calculated as follows:

$$d_a = \sqrt{\frac{F_{pullout}}{p_{uplift}}} = \frac{1}{\sqrt{N}} \tag{74}$$

A numerical application of these equations shows that, for an uplift pressure of 908 Pa and a ground anchor pullout resistance of 8000 N, the required number of anchors is 0.1135 per m^2 and the required distance between anchors is 3 m.

It is important to note that the required number of anchors per unit area and the distance between anchors are independent of the geomembrane type and properties (provided that the cap diameter is sufficiently large to ensure that the geomembrane will not fail due to stress/strain concentration at the perimeter of the cap). Therefore, it is easier to design the anchorage of geomembranes by punctual anchors than by anchor trenches (where the sizing of anchor trenches depends in general on the tensile properties of the geomembrane).

In addition to the determination of the required number of anchors presented above, it is necessary to check that the geomembrane strain is acceptable for the considered type of geomembrane. The shape of an uplifted geomembrane between anchors is complex, and numerical modeling would be necessary to determine the strain field in the geomembrane. An average strain can be calculated using the following equation proposed by Giroud (2009) [13] adapted to the case of punctual anchors:

$$\varepsilon_{approx} \approx \frac{0.3467\left(\dfrac{p_{uplift}\, d_a}{E\ t_{GMB}}\right)^{2/3}}{1-0.3103\left(\dfrac{p_{uplift}\, d_a}{E\ t_{GMB}}\right)^{2/3}} \approx 0.35\left(\frac{p_{uplift}\, d_a}{E\ t_{GMB}}\right)^{2/3} \tag{75}$$

where E is the tensile modulus of the geomembrane, and t_{GMB} is the thickness of the geomembrane.

The strain thus calculated must be lower than the acceptable strain for the considered geomembrane.

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The maximum stress and strain occur at the perimeter of the cap. The diameter of the cap should be sufficiently large to ensure that the geomembrane will not fail due to stress/strain concentration at the perimeter of the cap. The required cap diameter to prevent geomembrane rupture by excessive stress or strain at the edge of the cap should be determined by full scale tests. An approximate value of the required cap diameter can be calculated using the following equation:

$$D_{req} = \frac{F_{pullout}}{\pi\, T_{allow} \sin\theta} = \frac{F_{pullout}}{\pi\, \sigma_{allow}\; t_{GMB} \sin\theta} \tag{76}$$

where T_{allow} is the allowable tension in the geomembrane, σ_{allow} is the allowable stress in the geomembrane in the case of a non-reinforced geomembrane, t_{GMB} is the geomembrane thickness, and θ is the angle between the uplifted the geomembrane and the plane of the cap.

In the case of an HDPE geomembrane, with a strain at yield of 12-15%, θ is approximately 50º (as tabulated in Giroud *et al.* 1995a [10]), hence $\sin\theta \approx 0.75$. Using as an allowable stress equal to the yield stress of 16 MPa, Equation 70 gives a required cap diameter of 0.14 m for a typical ground anchor pullout resistance of 8000 N. More conservatively, an allowable stress obtained by dividing the yield stress by 1.4 can be used. Considering the non-linear stress-strain curve of HDPE geomembranes, the 1.4 factor on stress corresponds to a 4.0 factor on strain, hence an allowable strain of 3-4%; θ is then approximately 25º (as tabulated in Giroud *et al.* 1995a [10]), hence $\sin\theta \approx 0.42$. Equation 70 gives a required cap diameter of 0.35 m with this conservative calculation.

4.6.5 Stresses and strains in a geomembrane exposed to flowing water

This section presents discussions that reflect the complexity of stresses and strains in geomembrane linings exposed to flowing water.

In the analyses presented in Sections 4.6.2 and 4.6.3, the cases of longitudinal and transverse anchorage were treated separately, and relatively simple geomembrane deformation scenarios were considered. In reality, when the geomembrane lining is displaced in the downstream direction, longitudinal and transverse anchors are subjected to stresses simultaneously. For the sake of simplicity, the discussion is limited to the case of a canal bottom where the exposed geomembrane lining is a rectangle between two transverse anchor trenches and two longitudinal anchor trenches at the toe of the side slopes. When the geomembrane is displaced in the downstream direction, the upstream transverse anchor trench and both longitudinal anchor trenches are subjected to tensile stresses (while the downstream anchor trench is not subjected to stresses). Away from the upstream transverse anchor trench, the maximum geomembrane displacement is at the center line of the canal since the geomembrane cannot move at its connection with the longitudinal anchors. As a result, the geomembrane lining is distorted. Therefore, it is not surprising to observe geomembrane tensile failure or tears at an angle (with respect to the longitudinal direction of the canal) near the sides of the canal.

This discussion provides guidance to explain the pattern of tears and tensile rupture observed in the case of geomembrane lining failure due to the uplift-dragging combination. However, to the best knowledge of the authors of the present paper, there is no available design method that takes into account the distortion of the geomembrane to design longitudinal anchorage. Numerical modeling of the deformed geomembrane lining could be used to determine the location, magnitude and orientation of the maximum stresses and strains.

Another aspect of the complexity of stresses and strains in exposed geomembrane linings in canals is the action of flow turbulence on the geomembrane. To the best knowledge of the authors of the present paper, there is no method to evaluate the repeated stresses induced in an exposed geomembrane lining by flow turbulence. It is reasonable to assume that these repeated stresses may, after some time, cause rupture of the geomembrane by fatigue or progressive cracking. Therefore, it is appropriate to avoid these repeated stresses by covering the geomembrane with a soil or concrete layer in areas where turbulence is likely to develop, for example downstream of regulating structures such as gates or weirs, over a distance determined by a hydraulic analysis, as indicated in Sections 4.2.4 and 4.5.7.

4.6.6 Design of exposed geomembranes for safety

It is important to address a safety concern with exposed geomembranes in canals. Many geomembranes are slippery. Therefore, measures should be taken to allow egress of humans and animals from a canal, such as ladders or steps, every 50 m for example. Alternatively, a geomembrane with a rough surface should be selected.

5 Conclusion

Despite its title, the present paper is not a complete treatise of the design of canals lined with geomembranes. Thus, geomembrane selection and leakage/seepage control are not addressed in the present paper, but will be addressed in the book on Geomembranes for Lining Canals (Giroud and Plusquellec 2023b) [14]. However, many aspects of the design of geomembrane linings are addressed herein, and a number of original methods and equations are presented. Some of these equations were developed for actual canal design and some were developed during the preparation of the present paper. By providing design methods, it is hoped that the present paper will contribute to the expected growing use of geomembrane linings in canals. Indeed, because of their watertightness, geomembrane linings can play an essential role in canals, in the decades to come, considering the increasing demand for water.

As shown in the present paper, geomembrane linings need to be protected in many instances, and lining systems that associate geomembrane watertightness and concrete robustness are often the best solution to achieve effective leakage/seepage control and long-term performance. The best illustration of this geomembrane-concrete association is the Toshka Canal, a landmark project used several times as an example in the present paper.

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