The Vienna Terzaghi Lecture in Yokohama

Geosynthetics engineering: successes, failures and lessons learned

by

J.P. Giroud

Consulting Engineer

JP GIROUD, INC.

Chairman Emeritus of GeoSyntec Consultants

Past President of the International Geosynthetics Society

18 September 2006

Under the auspices of the Japanese Chapter of the International Geosynthetics Society

The Vienna Terzaghi Lecture in Yokohama

First presentation in Japan of the lecture
originally presented by Dr. Giroud in Vienna, Austria, in February 2005

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Presentation of the Vienna Terzaghi Lecture
by Professor Heinz Brandl
Technical University of Vienna
on the next page,

followed by
biographical note, abstract, lecture notes and copies of all slides,

and followed by a technical paper

The 2005 Vienna Terzaghi Lecture

By Professor Heinz Brandl of the Technical University of Vienna

On 21 and 22 February 2005, the 5th Austrian Geotechnical Conference took place in Vienna, Austria, organized by the ÖIAV (Austrian Society of Engineers and Architects) and the Austrian Member Society of ISSMGE (International Society for Soil Mechanics and Geotechnical Engineering) – in cooperation with Members of the IGS (International Geosynthetics Society). This conference takes place in two year intervals and its highlight has always been the "Vienna Terzaghi Lecture," presented immediately after the opening ceremony.

Dr. J.P. Giroud delivered this prestigious lecture choosing the title "Geosynthetics Engineering: Successes, Failures and Lessons Learned," and he did this in a historical place, in the Festivity Hall of the Palais Eschenbach. The ÖIAV has existed since 1848, and has therefore a very old tradition, being the "umbrella" society of Austrian engineers and architects (until 1918 also for the whole Austrian-Hungarian Monarchy). Engineering history has been written there, not only regarding civil engineering, but also several other branches of engineering. Among numerous outstanding members, Karl Terzaghi (soil mechanics), Ferdinand Porsche (automotive industry), Leopold Müller (rock mechanics) and Nikola Teska (electro-engineering) may be mentioned.

Dr. Giroud delivered his Vienna Terzaghi Lecture from the podium where engineering history has been written since the midst of the 19th century - for instance, the discussions about the design of the Suez Canal by Alois Negrelli. Here, Karl Terzaghi, Professor at the Technical University of Vienna, presented his revolutionary "Theory of Clay Settlement" for the first time. The very critical audience was split in pros and cons, and intensive, sometimes aggressive debates followed. These days it is a little different. The Vienna Terzaghi Lecturer, J.P. Giroud, was certainly not in such danger – he did not stir a scientific war but was instead met with unanimous applause!

In his introduction, the Conference Chairman, Professor Brandl, changed Jean Pierre Giroud to "Jean Paul" Giroud underlining that Dr. Giroud can be considered the "Pope of Geosynthetics". He mentioned the "Giroud Lecture" as the prestigious opening lecture at the International Conferences of the IGS and his Honorary Membership of the IGS, which he was awarded with the citation "Dr. Giroud is truly the father of the IGS and the geosynthetics industry". Prof. Brandl's introduction ended by comparing the three fathers of their discipline: Prof. K. Terzaghi for soil mechanics, Prof. L. Müller for rock mechanics and Dr. J.P. Giroud for geosynthetics.

In his Vienna Terzaghi Lecture, Dr. Giroud combined theory and practice in an optimal way, covering practically the entire field of versatile geosynthetics applications. Some contradictions between "common sense" and rational analyses illustrated the danger of ignoring theory and overestimating mere practice. The first part of the lecture was devoted to failures as an excellent means of learning. The second part then focused on lessons learned from successes. Theories and numerous case histories referred to liquid impoundments, landfill slopes, dam rehabilitation, and filter design.

J.P. Giroud's Vienna Terzaghi Lecture was an outstanding firework-presentation from a rhetorical and professional point of view. The audience thanked with long-lasting enthusiastic applause, and the feedback of the conference participants has continued to be enthusiastic. The oral version of J.P. Giroud's Vienna Terzaghi Lecture has been stored on a CD in the "Vienna Terzaghi Museum" that is being created under the auspices of the ISSMGE at the Technical University of Vienna.

The 2005 Vienna Terzaghi Lecturer

Dr. Giroud, a pioneer in the field of geosynthetics since 1970, is recognized throughout the world as a geosynthetics leading expert. A former professor of geotechnical engineering, he is a consulting engineer under JP GIROUD, INC., and chairman emeritus and founder of GeoSyntec Consultants. Dr. Giroud is past president of the International Geosynthetics Society (the IGS), chairman of the editorial board of *Geosynthetics International*, and was Chairman of the Editorial Board of *Geotextiles and Geomembranes* (1984-1994). Dr. Giroud was chairman of the 2nd International Conference on Geotextiles (1982) and the International Conference on Geomembranes (1984). He served two terms as chairman of the Technical Committee on Geosynthetics of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE).

Dr. Giroud coined the terms "geotextile" and "geomembrane" in 1977, thus starting the geo-terminology used in geosynthetics engineering. He has authored over 350 publications, including a monumental Geosynthetics Bibliography (1721 pages, more than 10,000 references); and he recently wrote the chapter on filter criteria in the prestigious book commemorating the 75th anniversary of Karl Terzaghi's book "*Erdbaumechanik*".

Dr. Giroud has developed many of the design methods used in geosynthetics engineering. For example, he developed methods for the evaluation of leakage through liners, for the design of drainage layers (including leachate collection layers and leakage detection layers), for soil cover stability, for the reinforcement of liners and soil layers overlying voids, for the resistance of geomembranes exposed to wind uplift, for the design of unpaved roads, and for the design of geotextile and granular filters. Also, he played a key role in the development of landfill construction quality assurance (1983-1984).

Dr. Giroud has extensive field experience and has originated a number of geosynthetics applications such as: first nonwoven geotextile filter (1970), first geotextile filter in a dam (1970), first geotextile cushion for geomembrane (1971), first double liner with two geomembranes (1974), first entirely geosynthetic double liner system with two geomembranes and a geonet leakage detection system (1981). He has been instrumental in the development of the technique of exposed geomembrane landfill covers (1995-1998).

Dr. Giroud has received awards from the French Society of Engineers and Scientists, the Industrial Fabrics Association International, and the IGS (in 1994 for liner leakage prediction and in 2004 for filter design). In 1994, the IGS named its highest award "The Giroud Lecture", "in recognition of the invaluable contributions of Dr. J.P. Giroud to the technical advancement of the geosynthetics discipline". In 2002, Dr. Giroud became Honorary Member of the IGS with the citation "Dr. Giroud is truly the father of the International Geosynthetics Society and the geosynthetics industry". In 2005, Dr. Giroud has been awarded the status of "hero" of the Geo-Institute of the American Society of Civil Engineers (ASCE). It was the first time this new award was granted.

Dr. Giroud has delivered keynote lectures at numerous international conferences. In 2005, he presented the prestigious Vienna Terzaghi Lecture, and, in 2005-2006, the prestigious Mercer Lecture series.

Dr. Giroud can be contacted at jpg@jpgiroud.com

The Vienna Terzaghi Lecture in Yokohama

Geosynthetics engineering: successes, failures and lessons learned

by

J.P. Giroud

ABSTRACT

The lecture presents in detail two cases of failures and two cases of successes related to structures incorporating geosynthetics. Analyses of these cases are presented and lessons learned are discussed. An important lesson learned is that engineering problems, whether they are related to failures or successful applications, can always be solved by following a rational approach, generally including theoretical analyses. In contrast, common sense or "engineering judgment", used without the support of a rational approach, can be misleading, as illustrated by examples. In other words, this lecture is consistent with the "theory and practice" approach promoted by Terzaghi. Another lesson learned is that geosynthetics engineering is an integral part of both geotechnical and civil engineering, which results in fruitful technology transfer. Also, failures and successes are put into perspective: it is shown that failures represent a very small fraction of the structures incorporating geosynthetics. To illustrate this point, the lecture includes a survey of the most important applications of geosynthetics using spectacular photographs of structures incorporating geosynthetics constructed in various countries. Even though innovative methods are presented, the lecture is presented in a simple and entertaining way and is accessible to all civil engineers.

A lively report of the lecture by Dr. Giroud in Vienna, with photographs of the podium from where Karl Terzaghi was lecturing in the 1920s, can be found by following the link:
http://www.geosynthetica.net/calendar/5thAustrianConferenceReport.asp

See also IGS News, Vol. 20, No. 3, November 2004, pp. 9-10,
and Vol. 21, No. 1, March 2005, pp. 10-11.
IGS News can be obtained on www.geosyntheticssociety.org

The Vienna Terzaghi Lecture in Yokohama

by J.P. Giroud

NOTES

Most slides are self explanatory. A few notes follow.

SLIDE 1
In 2005, I was very honored to present the Terzaghi Lecture in Vienna, Austria.

SLIDE 2 (2nd slide of page 1)
The setting was impressive as you see on this photo where I am seated between President Cazzuffi of the International Geosynthetics Society and Professor Brandl from the Technical University of Vienna.

SLIDE 3 (3rd slide of page 1)
I was using the very lectern from where Terzaghi had presented the consolidation theory.

SLIDE 11 (5th slide of page 2)
The observations are summarized on this slide.
There was a wide open central crack.
On both sides of the central crack the geomembrane was shattered.
The weather was very cold.
And the geomembrane was under tension away from this area.

SLIDE 17 (5th slide of page 3)
The observations that I just described are summarized here.
The central crack was next to a seam, and it was open at mid-slope and closed at crest and toe.
The shattering cracks were approximately symmetrical with respect to the central crack.
Also, there was no crack along the xx' axis.

SLIDE 28 (4th slide of page 5)
I found that the strain in the geomembrane at the location of maximum bending was 1.8% when it was 1% in the geomembrane away from the seams. In other words, the strain at the location of maximum bending was 80% greater than the strain in the geomembrane away from the seams.

SLIDE 30 (6th slide of page 5)
The upper face of the geomembrane was exposed to the cold weather, whereas the lower face of the geomembrane was in contact with the relatively warm soil. As a result, the geomembrane tended to contract in the vicinity of its upper face, which caused an additional strain in the upper face.

SLIDE 31 (1st slide of page 6)
Therefore, the location of maximum strain is in the upper face of the lower geomembrane panel, as shown here. This explains why cracking occurs in the lower panel, not in the upper panel.

SLIDE 35 (5th slide of page 6)
Here is the model I used for the demonstration. It is based on the assumption that the opening of the central crack occurred first.
The opening of the central crack resulted in a distortion of the geomembrane as shown by these parallelograms.
This distortion decreases with increasing values of the abscissa x, and there is no distortion at a distance d from the central crack.
This distortion increases with increasing values of the ordinate y, and there is no distortion along the Ox axis.

SLIDE 36 (6th slide of page 6)
The distortion depends on x and y, and the model was used to develop the Mohr's circle for strains. Therefore, the Mohr's circle depends on x and y.

SLIDE 37 (1st slide of page 7)
It is a well-known property of the Mohr's circle that the red line is perpendicular to the direction of maximum strain. Therefore, the red line is the crack direction.

SLIDE 39 (3rd slide of page 7)
This is the theoretical pattern of cracks derived from the Mohr's circle. This theoretical pattern of cracks is very similar to the observed pattern of cracks, including the fact that there is no crack along the horizontal axis.

SLIDE 41 (5th slide of page 7)
To reduce the geomembrane tension, compensation panels were used.
These compensation panels consisted of additional strips of geomembrane forming a fold, inserted at regular spacing, to allow geomembrane contraction without tension.

SLIDE 50 (2nd slide of page 9)
The slope stability equations that take into account seepage forces and all the stability mechanisms are shown here.
It is important to note that there are two equations: the first equation for the case where the slip surface is above the geomembrane, and the second equation for the case where the slip surface is below the geomembrane.
The most important term in these two equations is the first term, the term related to interface friction angle.

SLIDE 51 (3rd slide of page 9)
Here we consider only the first term of the equations.
As we can see here, when there is water flow, the first term for a slip surface located below the geomembrane is the same as the first term when there is no water. In other words, stability for a potential slip surface located below the geomembrane is not affected by water flowing above the geomembrane. If fact, I should say "not significantly affected" instead of "not affected" because, here, I consider only the first term of the equations.
Now, we see that, when there is water flow, the first term for a slip surface located above the geomembrane is different from the first term when there is no water. The difference is due to this fraction circled in red.
And the value of this fraction, as all geotechnical engineers know, is approximately 0.5 due to the densities of soil and water.

Therefore, the factor of safety with respect to stability for a potential slip surface located above the geomembrane is significantly affected by water flowing above the geomembrane.

SLIDE 66 (6th slide of page 11)
But, first, I want to put failures in perspective. The rate of significant failures in geosynthetic applications has been estimated as 0.1% of the applications, which is a small number. In contrast, to date, 20 billion square meters of geosynthetics have been used successfully in several million projects. And a number of these projects are significant and spectacular.

SLIDE 118 (4th slide of page 20)
When concrete is exposed to water, it can be deteriorated by frost or alkali-aggregate reaction. Here, we see the face of a concrete dam after the reservoir has been emptied. The concrete had deteriorated in less than 40 years.
And, in the case of concrete deterioration, the rate of leakage through a dam can increase by a factor up to 10.

SLIDE 120 (6th slide of page 20)
This slide shows the various steps of rehabilitation.
First, the concrete is repaired locally as needed.
Then, a geonet or thick geotextile drainage layer is placed on top of the concrete.
Then, a geomembrane is placed on the geonet or thick geotextile.

SLIDE 125 (5th slide of page 21)
Samples of geomembrane are taken periodically from the rehabilitated dams, and are sent to a laboratory for testing.
Based on 20 years of testing, a durability of 50 years is predicted.

SLIDE 131 (5th slide of page 22)
Here we see a drainage trench where a geotextile filter is located between the gravel drainage material and the soil.

SLIDE 132 (6th slide of page 22)
Here we see a geotextile filter placed between a geonet drain (in fact a leachate collection layer in a landfill) and a soil layer.

SLIDE 133 (1st slide of page 23)
To design a geotextile filter, we need to use a retention criterion, in other words we must answer the following question:
How should we select the maximum allowable opening size of a geotextile filter to retain a soil?
A simple answer consists of adapting Terzaghi's criterion for granular filters, which is expressed by the first equation here:
d15 of the filter must be less than 5 times the d85 of the soil.
It is known that the opening size of a granular filter is about one fifth of the d15 of the filter, as shown by the second equation.
Combining the first two equations, gives the last equation, which is shown in red.

SLIDE 134 (2nd slide of page 23)
According to this equation, the opening size of the geotextile filter should be less than the d85 of the soil.
This equation means that a filter should only retain large soil particles.

Incidentally, this is against common sense. Indeed, common sense dictates that a filter should retain not only the large soil particles, but also the smallest soil particles.
Retaining large soil particles works if the large particles retain smaller particles, in other words if the sol is internally stable.

SLIDE 141 (3rd slide of page 24)
Here we have a graph that gives the ratio between the opening size of the filter and the d85 of the soil, as a function of the coefficient of uniformity of the soil.
I will use this graph to represent retention criteria.

SLIDE 142 (4th slide of page 24)
The red horizontal dashed line represents the traditional Terzaghi's criterion adapted for geotextile filters as I explained earlier. This criterion means that the opening size of the geotextile filter should be equal to or less than the d85 of the soil, regardless of the coefficient of uniformity of the soil.

SLIDE 143 (5th slide of page 24)
The black curve represents a retention criterion for geotextile filters that takes into account the internal stability of the soil. This retention criterion shows that the ratio between the filter opening size and the d85 of the soil can be greater than 1 for soils having a small coefficient of uniformity, but must be smaller than 1 for soils having a large coefficient of uniformity.

SLIDE 148 (4th slide of page 25)
Finally, the retention criterion developed for geotextile filters has been extended for granular filters.
This retention criterion shows that the ratio between the d15 of the filter and the d85 of the soil can be greater than 5 for soils having a small coefficient of uniformity. This is consistent with classical experiments by Bertram and Sherard. This shows that for small coefficients of uniformity, the traditional Terzaghi's criterion is conservative, which is not a problem.
But, far more importantly, we see that for soils with a large coefficient of uniformity, the traditional Terzaghi's criterion is unconservative, and that is a serious problem.

* * * * *

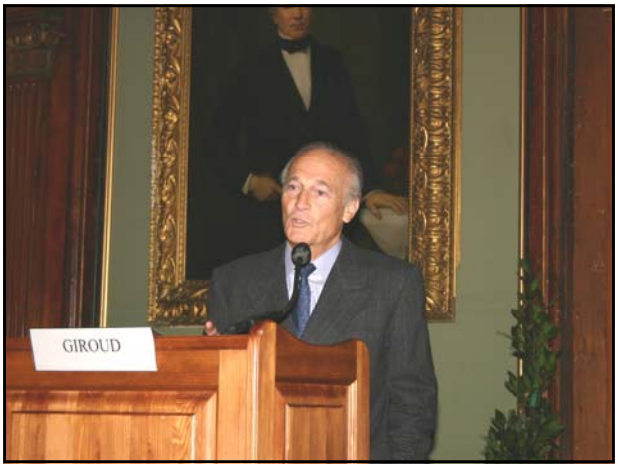
A copy of all slides follows (pages 1 to 26)

VIENNA, 2005

The Terzaghi Lecture

**GEOSYNTHETICS ENGINEERING:
SUCCESSES, FAILURES
AND LESSONS LEARNED**

J.P. GIROUD



YOKOHAMA, 2006

The Terzaghi Lecture

**GEOSYNTHETICS ENGINEERING:
SUCCESSES, FAILURES
AND LESSONS LEARNED**

J.P. GIROUD

**Karl Terzaghi
and Mission Dam
(now Terzaghi Dam)
1960**

There were localized failures, but essentially it was a success.

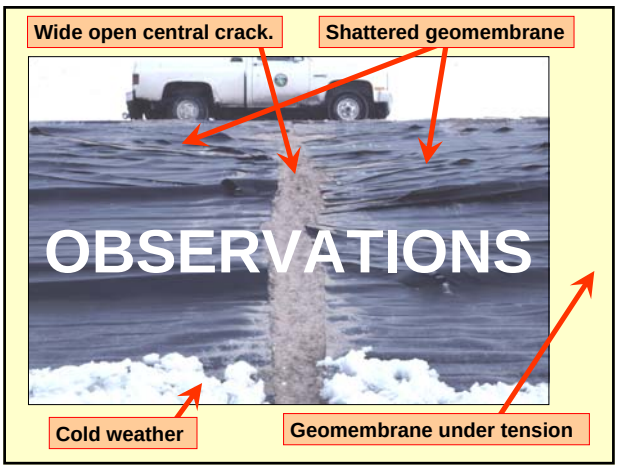
Following Terzaghi's example, let's learn from failure and success.

FIRST EXAMPLE

INVESTIGATION OF GEOMEMBRANE CRACKING FAILURE

The problem happened in large water reservoirs lined with geomembrane.

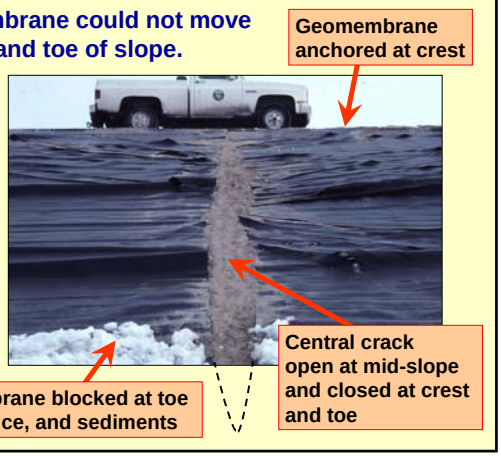
In the winter, the reservoirs were empty and cracking of the geomembrane liner occurred at several locations.



Just before cracking occurred, the cold weather tended to contract the geomembrane, but the contraction was restrained, which resulted in geomembrane tension.

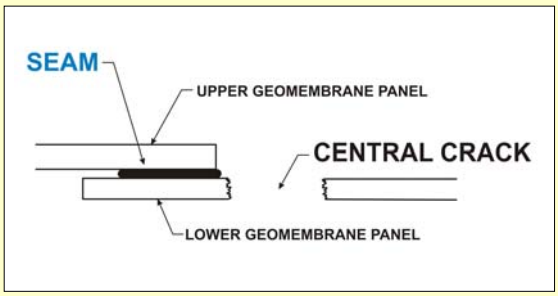
Thermal contraction was restrained because the geomembrane could not move at crest and toe.

Geomembrane could not move at crest and toe of slope.



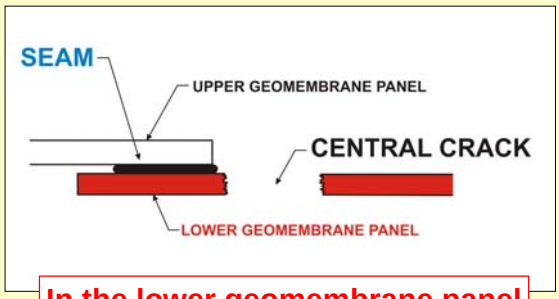
Shattering cracks go up in upper half and go down in lower half

The CENTRAL CRACK was next to a SEAM.



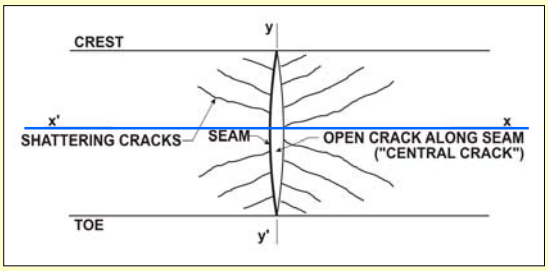
CROSS SECTION OF GEOMEMBRANE

The CENTRAL CRACK was next to a SEAM.



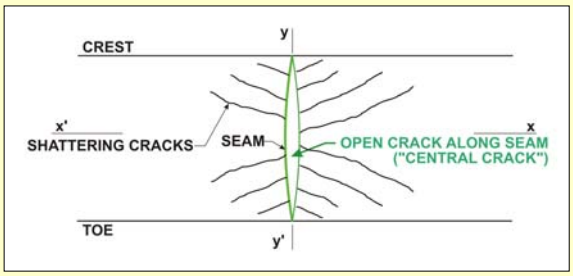
In the lower geomembrane panel

SUMMARY OF OBSERVATIONS



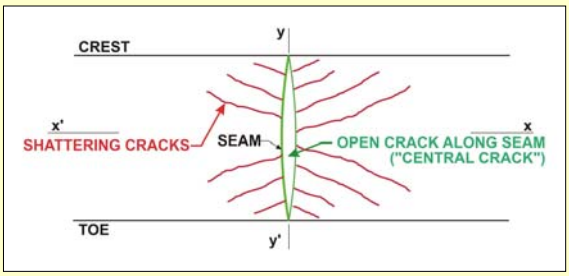
No crack along the x'x axis.

Did the crack along the seam occur first?
(which could have caused the shattering cracks)



Did the crack along the seam occur first?
(which could have caused the shattering cracks)

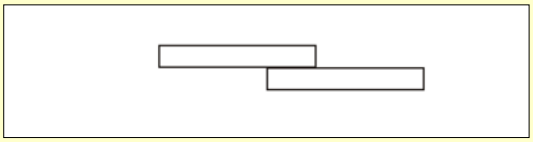
Did the shattering cracks occur first?



It was important to find the mechanism of failure.

- **If the opening of the central crack triggered the shattering cracks, then the geomembrane tension played a role, and reducing the tension could be the solution.**
- **If the shattering cracks were not linked to the central crack, the geomembrane was defective and had to be replaced.**

To explain the mechanism,
I looked for something special
next to the seam,
I noted that a geomembrane under tension
has to bend next to a seam
to ensure that the tensile forces are aligned.



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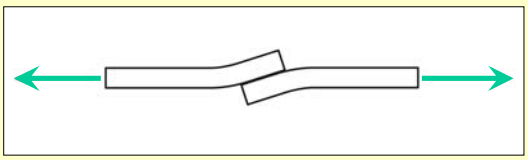

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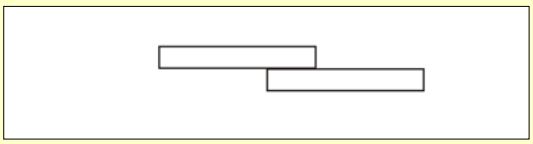

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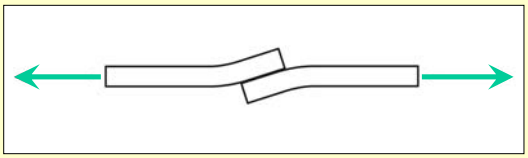

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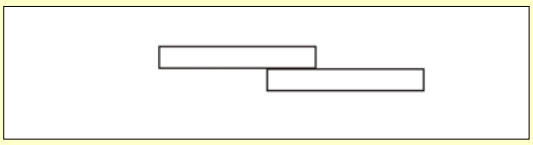
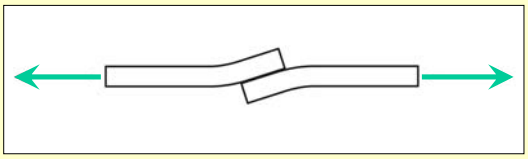
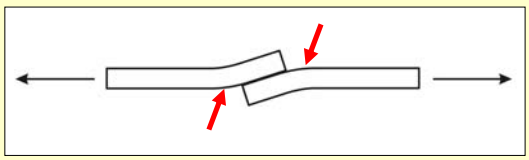


Looking for something special next to the seam, I noted that a geomembrane under tension has to bend next to a seam to ensure that the tensile forces are aligned.

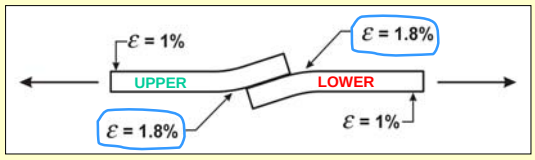


I calculated the geomembrane strain at the locations of maximum bending.



Results on the next slide →

CALCULATED GEOMEMBRANE STRAINS

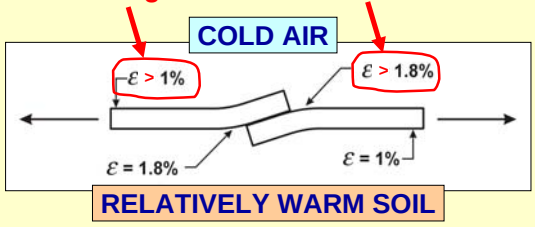


This 80% explains why cracking is more likely to occur next to seams than away from seams,

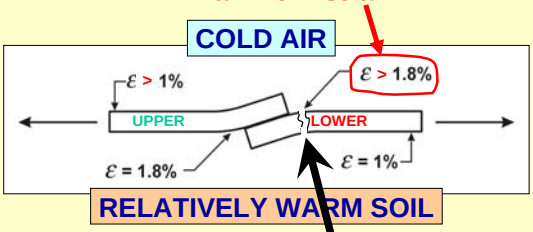
but it does not explain why cracking occurs in the lower geomembrane panel and not in the upper geomembrane panel.

However, there was a difference between the situation of the upper panel and the situation of the lower panel.

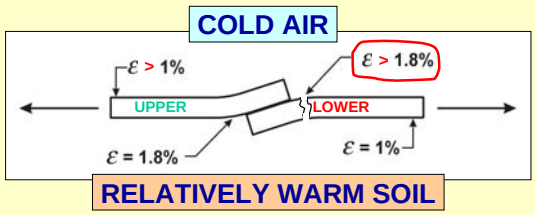
Additional geomembrane strain



Maximum strain

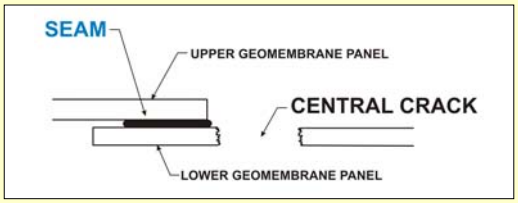


This explains why cracking occurs in the lower panel and not in the upper panel.



This also shows that the problem is not the quality of the seam, but the presence of the seam.

At this point, we had an explanation for the development of the central crack next to the seam, in the lower panel.

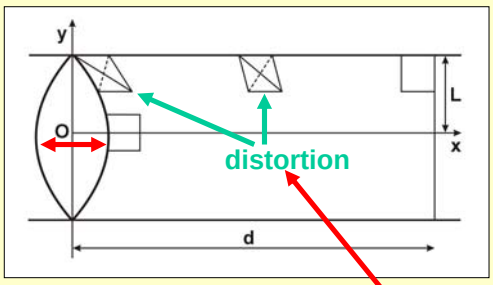


But . . . we had not demonstrated that the central crack had triggered the shattering cracks.

To be convincing, the demonstration had to be based on engineering principles.

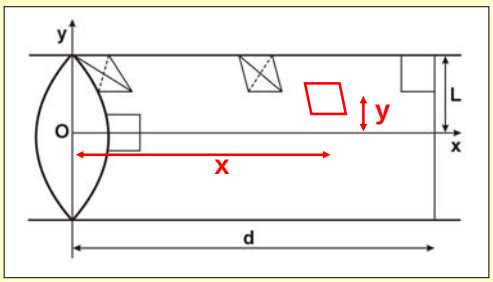
[ALL SORTS OF "COMMON SENSE" EXPLANATIONS HAD BEEN PROPOSED.]

MODEL USED FOR THE DEMONSTRATION

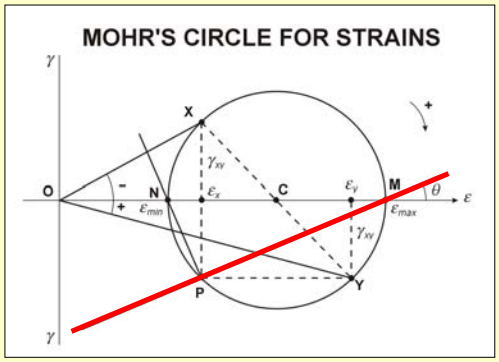


ASSUMPTION: Opening of the central crack occurs first

MODEL USED FOR THE DEMONSTRATION



Using this model: Mohr's circle for strains in the geomembrane as a function of x and y.

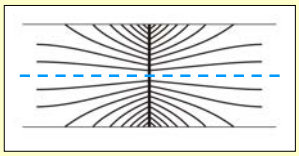


PM is perpendicular to the maximum strain. Therefore, it is the crack direction.

The Mohr's circle depends on x and y. Therefore, the direction of cracks depends on x and y.

The entire process is analytical. Therefore, I obtained the equation for a family of curves.

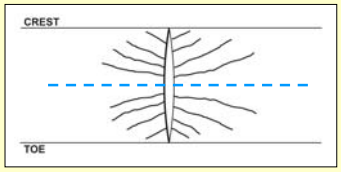
This family of curves is the theoretical pattern of cracks.



THEORETICAL PATTERN OF CRACKS

very similar to

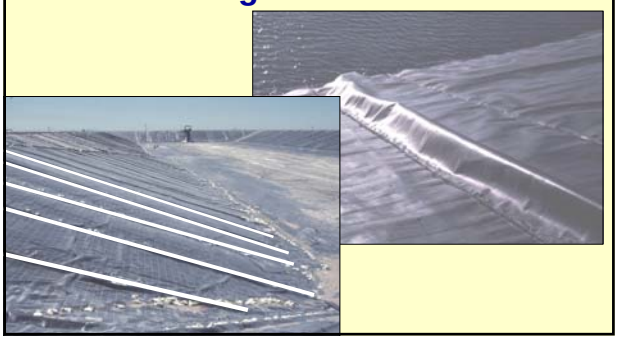
OBSERVED PATTERN OF CRACKS



CONCLUSION OF THE ANALYSIS

- **The analysis was based on the assumption that the central crack occurred first.**
- **The analysis explained the observations.**
- **Therefore, the assumption was correct.**
- **The opening of the central crack was caused by geomembrane tension in cold weather.**
- **Therefore, the geomembrane tension in cold weather had to be reduced.**

"Compensation panels" were used to reduce tension in the geomembrane.



LESSON LEARNED

from this failure investigation

Complex mechanisms associated with geosynthetics can be rationally analyzed using methods typically used in engineering disciplines.

Clearly, geosynthetics engineering is one of the engineering disciplines.

SECOND EXAMPLE

INSTABILITY OF GEOMEMBRANE/SOIL LAYERED SYSTEM ON SLOPE

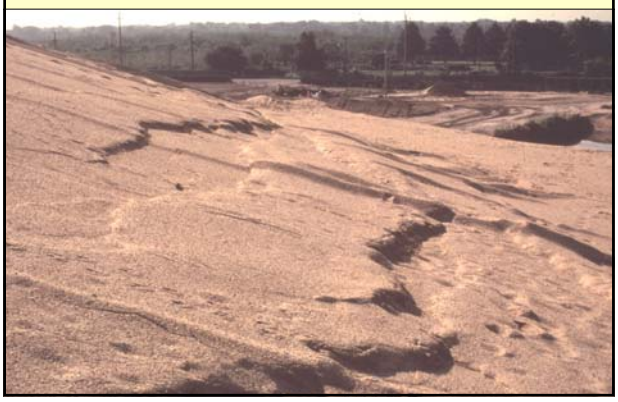
In landfills, reservoirs, dams, etc. we use layered systems composed of:

- **Soil layers**
  (sometimes reinforced with geogrid)
- **Geotextiles**
- **Geonets**
- **Geocomposites**
- **Geomembranes**

A slip surface may develop at one of the interfaces between these layers, which results in instability.

INSTABILITY OF LINER SYSTEM AT END OF CONSTRUCTION

INSTABILITY OF TEMPORARY COVER

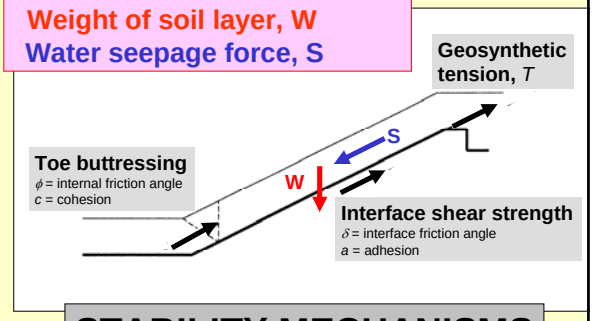


INSTABILITY OF LANDFILL COVER

CAUSES OF INSTABILITY:

Weight of soil layer, W

Water seepage force, S



STABILITY MECHANISMS

SLOPE STABILITY EQUATIONS

(full water depth)

$$FS_A = \frac{\gamma_b}{\gamma_{sat}}\frac{\tan\delta_A}{\tan\beta} + \frac{a_A}{\gamma_{sat}\, t \sin\beta} + \frac{\gamma_b}{\gamma_{sat}}\frac{t}{h}\frac{\tan\phi/(2\sin\beta\cos^2\beta)}{1-\tan\beta\tan\phi} + \frac{c}{\gamma_{sat}\, h}\frac{1/(\sin\beta\cos\beta)}{1-\tan\beta\tan\phi} + \frac{T}{\gamma_{sat}\, t\, h}$$

$$FS_B = \frac{\tan\delta_B}{\tan\beta} + \frac{a_B}{\gamma_{sat}\, t \sin\beta} + \frac{\gamma_b}{\gamma_{sat}}\frac{t}{h}\frac{\tan\phi/(2\sin\beta\cos^2\beta)}{1-\tan\beta\tan\phi} + \frac{c}{\gamma_{sat}\, h}\frac{1/(\sin\beta\cos\beta)}{1-\tan\beta\tan\phi} + \frac{T}{\gamma_{sat}\, t\, h}$$

A = above geomembrane **B = below geomembrane**

SLOPE STABILITY EQUATIONS

FIRST TERM WITHOUT WATER

$$FS = \frac{\tan\delta}{\tan\beta}$$

FIRST TERM WITH WATER

ABOVE

$$FS_A = \frac{\gamma_b}{\gamma_{sat}}\frac{\tan\delta}{\tan\beta}$$

$$\frac{\gamma_b}{\gamma_{sat}} = 0.50 \text{ to } 0.55 \approx 0.5$$

BELOW

$$FS_B = \frac{\tan\delta}{\tan\beta}$$

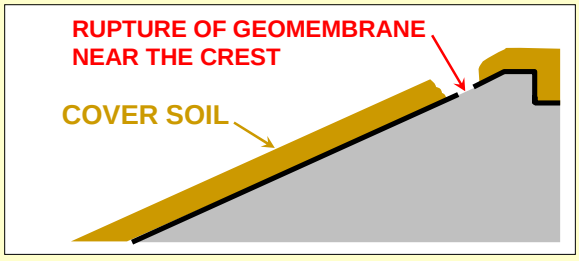
VERY SIGNIFICANT

Important lesson from theoretical analysis

Water above the geomembrane has little influence on slope stability if the slip surface is below the geomembrane, but has significant influence on the stability if the slip surface is above the geomembrane.

This lesson was used for a forensic analysis.

A landfill cover failed with geomembrane rupture near the crest of the slope, and large downward displacement.



The facts were simple:

- **Instability occurred after a thaw (at the end of a cold winter).**
- **The geomembrane ruptured near the crest of the slope.**

The explanation offered by all observers was simple:

- Instability occurred after a thaw.
- The thaw caused water to flow along the slope.
- It is known that water flowing along a slope causes instability.
- Therefore, the observed instability was caused by water flowing along the slope .

This simple explanation was:

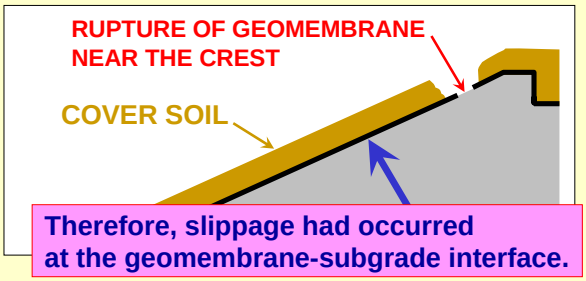
- consistent with experience, [failures are often caused by water]
- consistent with common sense, [water is not good for soil]
- easily understood and accepted, and
- incorrect !

The real explanation was:

- not provided by experience,
- not provided by common sense,
- not provided by engineering judgment.

The real explanation was provided by theoretical analysis.

Remember: the geomembrane rupture occurred near the crest of the slope, with large downward displacement of both cover soil and geomembrane.



The real explanation was:

- Slippage had occurred at the geomembrane-subgrade interface (i.e. below the geomembrane).
- Water flowing along a slope does not significantly affect the factor of safety for slippage below the geomembrane.
- Therefore, the failure was probably not caused by water flowing along the slope.

Based on this rational analysis, I could convince other participants that further investigation was necessary.

As shown by further investigation, there was a two-step mechanism.

STEP 1, WINTER

- In the winter, due to frost, there was migration of water vapor in the subgrade soil toward the geomembrane; and formation of ice beneath the geomembrane.
- This ice was sticking to the geomembrane, which ensured stability of the slope during the winter.

As shown by further investigation, there was a two-step mechanism.

STEP 2, SPRING

- **In the spring, due to a thaw, the ice melted under the geomembrane.**
- **The resulting water created a very low interface shear strength beneath the geomembrane, which caused instability of the slope along the interface between the geomembrane and the underlying soil.**

LESSON LEARNED from this failure investigation

- **Common sense is often wrong and should not be used as a basis for engineering decisions.**
- **Good observations and theoretical analyses lead to rational explanations.**

I have presented two examples of failure investigation, and lessons were learned.

LESSONS LEARNED FROM FAILURES

- **Complex mechanisms associated with geosynthetics can be rationally analyzed using engineering principles.**
- **Common sense is not an engineering principle.**

We just learned lessons from failures and, now, we will learn lessons from successes.

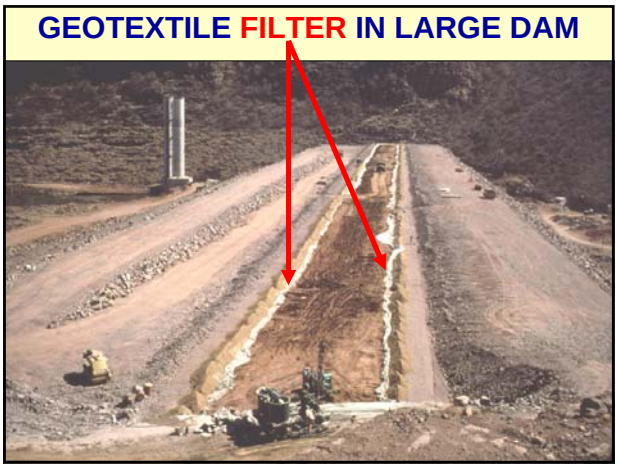
FAILURES AND SUCCESSES IN PERSPECTIVE

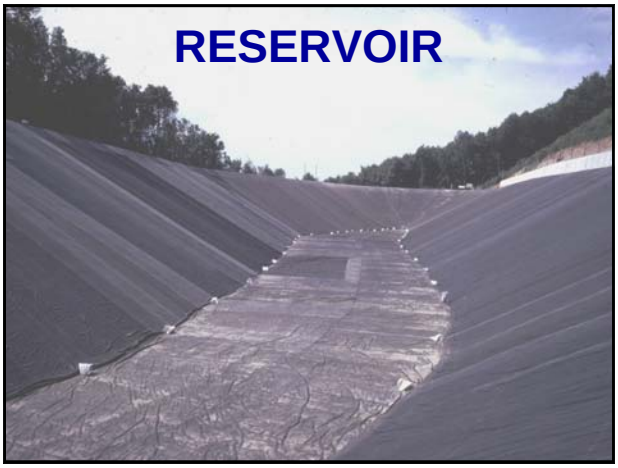
The rate of significant failures in geosynthetics applications has been estimated as 0.1 %.

Whereas, to date, 20 billion m² of geosynthetics have been used successfully in several million projects.

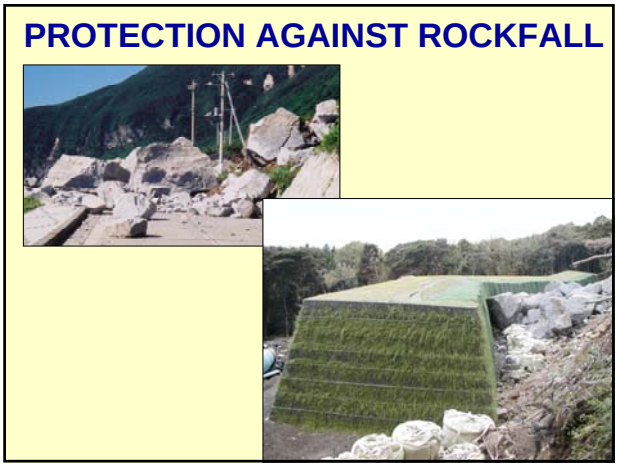
a number of them significant and spectacular

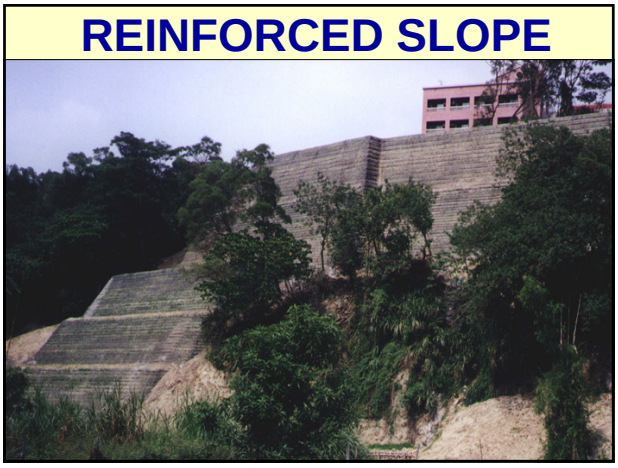

UPSTREAM FACE OF LARGE DAM


GEOTEXTILE FILTER IN LARGE DAM


RESERVOIR

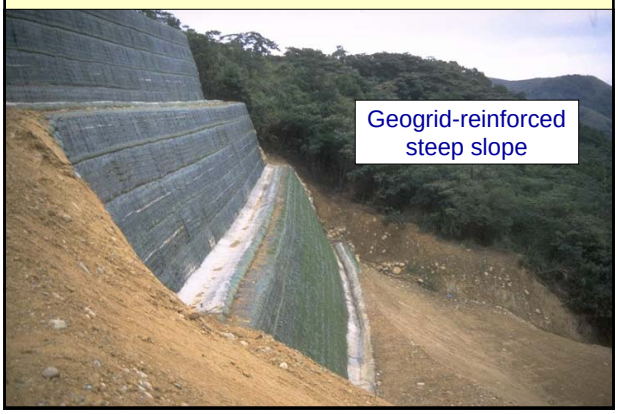

REINFORCED SOIL WALL
Light concrete blocks used as facing
Geogrid reinforcement


PROTECTION AGAINST ROCKFALL


REINFORCED SLOPE

REINFORCED SLOPE

LANDSLIDE REPAIR



REINFORCED LANDFILL

HAZARDOUS WASTE LANDFILL

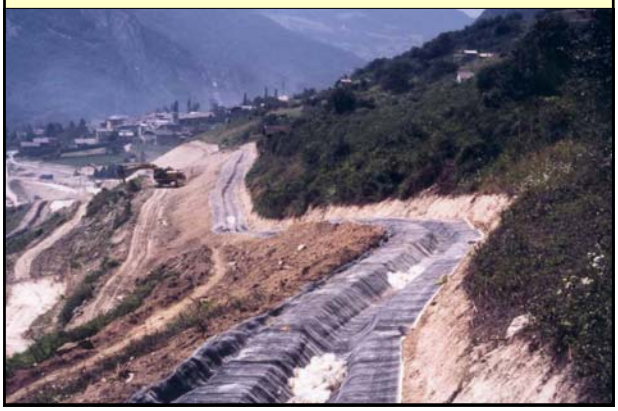
LANDFILL IN CANYON

EXPOSED GEOMEMBRANE AS LANDFILL COVER

MINING

LEACH PAD

SURFACE DRAINAGE



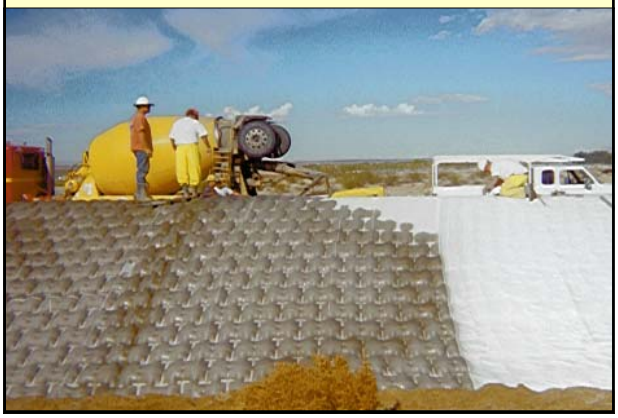
CANAL LINING

CANAL BANK PROTECTION

RIVER BANK PROTECTION

BANK PROTECTION

CONCRETE FORMING

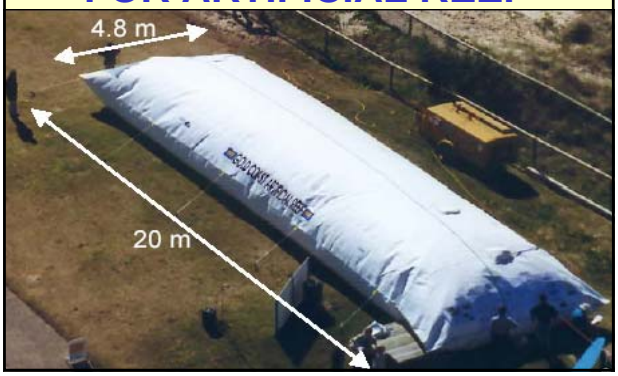


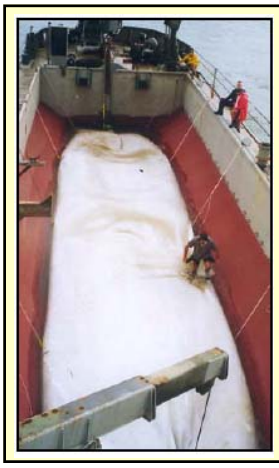
CONTAINMENT DIKES FOR ARTIFICIAL ISLAND

COASTAL WORKS

UNDERWATER INSTALLATION

LARGE SAND CONTAINERS FOR ARTIFICIAL REEF





DELIVERED USING A SPECIAL BARGE

GOLF COURSE

EROSION CONTROL

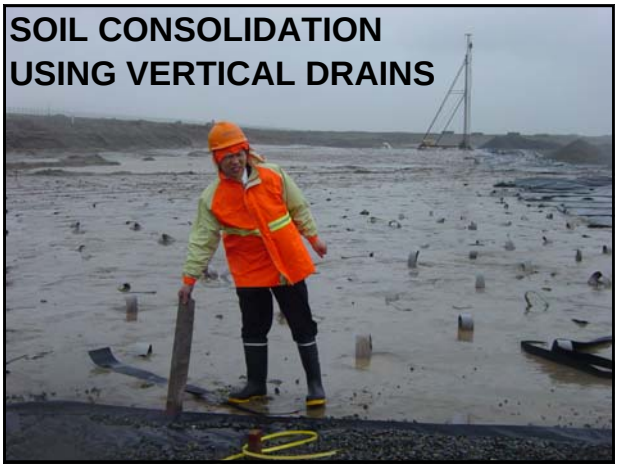
WITH GEOGRIDS

EROSION CONTROL WITH GEOMATS

EROSION CONTROL WITH GEOCELLS

To be filled with soil

SOIL CONSOLIDATION USING VERTICAL DRAINS



CONSTRUCTION ON SOFT SOIL

CONSTRUCTION ON SOFT SOIL

HIGHWAY EMBANKMENT

LOG YARD

AREA STABILIZATION

REINFORCED UNPAVED ROAD

UNPAVED ROAD CONSTRUCTED USING GEOCELL FILLED WITH AGGREGATE

ROAD CONSTRUCTION

ROAD BASE

ROAD PAVEMENT

ASPHALT OVERLAY

RAILWAY TRACK REPAIR

RAILWAY TRACK CONSTRUCTION

TUNNEL LINING

PILE FOUNDATION

These examples demonstrate that geosynthetics have successfully pervaded all branches of geotechnical engineering.

Now, I will discuss in more detail two examples of successes with geosynthetics.

FIRST EXAMPLE

USE OF GEOSYNTHETICS TO REHABILITATE OLD CONCRETE DAMS

Concrete exposed to water can be deteriorated by frost or aggregate-alkali reaction.

DAM FACE REHABILITATED USING GEOSYNTHETICS

REHABILITATION IN PROGRESS

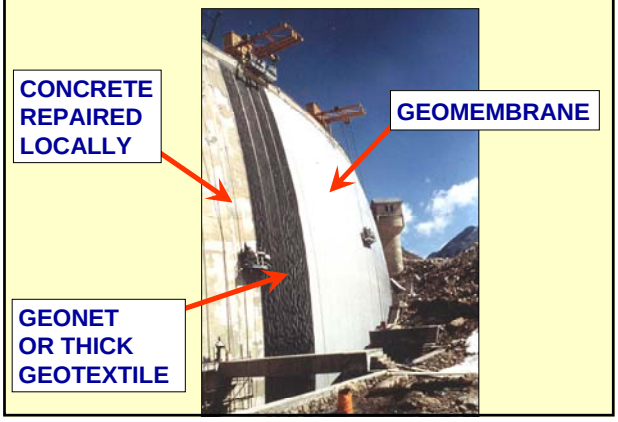
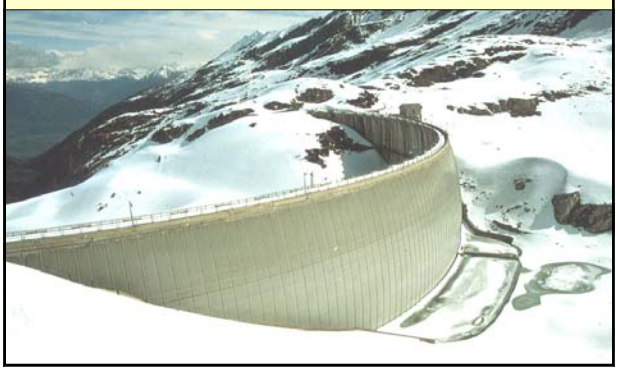


Photo taken 10 years after rehabilitation



REHABILITATION CONCEPT

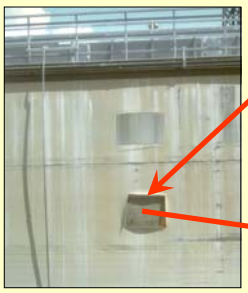
- The geomembrane provides impermeability.
- A geonet or a thick geotextile placed between the geomembrane and the concrete is acting as a drain.
- The main purpose of the system is to allow the concrete to progressively dry.
- Removing water from concrete decreases to a negligible level frost action and alkali-aggregate reaction.
- The geomembrane also decreases to a negligible level the leakage associated with concrete deterioration.

DURABILITY

- Durability is a major consideration in this application.
- In the rehabilitated dams, the concrete had deteriorated to a critical level in 40-60 years.

GEOSYNTHETIC DURABILITY

- In this application, the geosynthetics are exposed to harsh conditions (sunlight, weather, floating debris).
- To ensure durability, the geosynthetics have been carefully selected.
- To check durability, the geosynthetics are tested periodically.



Sample

Testing

Based on 20 years of testing, a durability of 50 years is predicted.

SYSTEM DURABILITY

- The geosynthetics on the dam face can be easily replaced at the end of their service life.
- This increases the durability of the dam indefinitely.
  (since the concrete does not deteriorate behind the geosynthetics)

A good example of complementarity between geosynthetics and traditional construction materials

LESSON LEARNED
from this successful application

The durability of geosynthetics
is not a problem
(if properly selected and properly used).

In some specific cases,
the durability of geosynthetics
can be similar to the durability
of traditional construction materials
such as concrete.

GEOMEMBRANES IN LARGE DAMS

- Geomembranes have been used as the only waterproofing barrier in more that 200 large dams according to the ICOLD.
- The first large dam with a geomembrane was constructed 46 years ago and is still in service.
- The highest dam with a geomembrane is 174 m high.

SECOND EXAMPLE
OF
LEARNING FROM SUCCESS

DEVELOPMENT
OF A
RETENTION CRITERION
FOR
GEOTEXTILE FILTERS

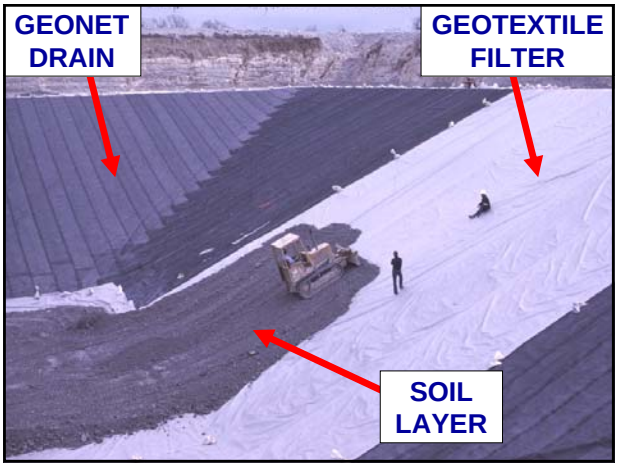
Filters are used

in geotechnical engineering

to separate drainage materials
(such as gravel or geosynthetic drains)

from soils that could clog them.

Example of geotextile filter between gravel drain and soil



RETENTION CRITERION

How should we select the maximum allowable opening size of a geotextile filter to retain a soil?

A simple answer consists of adapting Terzaghi's criterion for granular filters

$d_{15\ FILTER} < 5\ d_{85\ SOIL}$

$O_{FILTER} \approx d_{15FILTER} / 5$

hence $O_{FILTER} < d_{85\ SOIL}$

$O_{FILTER} < d_{85\ SOIL}$

- **This equation means that a filter should only retain large soil particles.**
- **(This is against common sense.)**
- **Retaining only large soil particles works if the large particles retain smaller particles.**

In other words, if the soil is internally stable.

Therefore,
an ideal retention criterion should take into account not only the opening size of the filter,
but also the internal stability of the soil.

To a certain degree, granular filters may work even if the soil is not internally stable because they are thick.

The mechanism is:
Particles that are not retained may accumulate in the filter, thereby decreasing the filter opening size, until the filter works.

In other words, a granular filter adapts itself to the soil (to a certain degree).

As a result, a granular filter can be designed (to a certain degree) using a retention criterion (i.e. Terzaghi's retention criterion) that does not take into account the internal stability of the soil.

Essentially, granular filters, because they are thick, can be designed using a simple retention criterion (Terzaghi's retention criterion).

However, this is true only to a certain degree, which limits the applicability of Terzaghi's retention criterion to soils with maximum particle size 4.75 mm.

hence, the practice of truncation

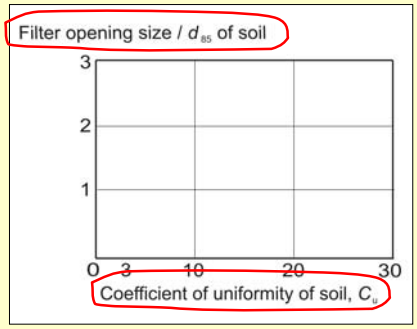
While granular filters benefit (to a certain degree) from their thickness,

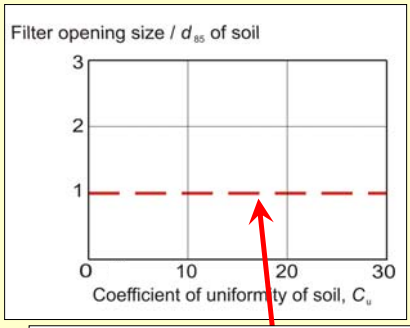
geotextile filters are thin, which has created an incentive for developing a more accurate retention criterion.

A criterion that takes into account the internal stability of the soil.

Internal stability depends on the particle size distribution of the soil, which is characterized by the coefficient of uniformity.

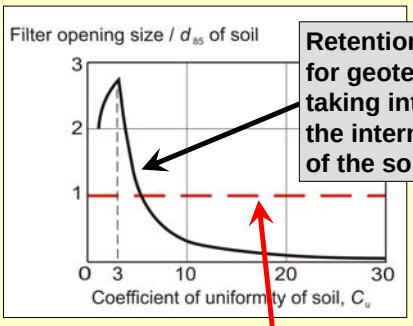
Therefore, an accurate retention criterion should take into account the coefficient of uniformity of the soil.





THE HORIZONTAL DASHED LINE REPRESENTS THE TRADITIONAL TERZAGHI'S CRITERION, DIRECTLY ADAPTED FOR GEOTEXTILE FILTERS:

$O_{FILTER} = d_{85\ SOIL}$

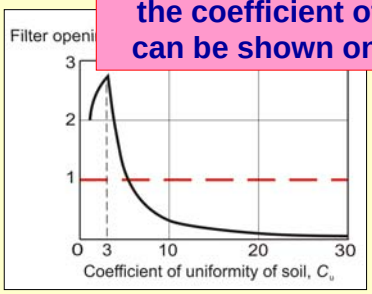


Retention criterion for geotextile filters taking into account the internal stability of the soil.

THE DASHED HORIZONTAL LINE REPRESENTS THE TRADITIONAL TERZAGHI'S CRITERION, DIRECTLY ADAPTED FOR GEOTEXTILE FILTERS:

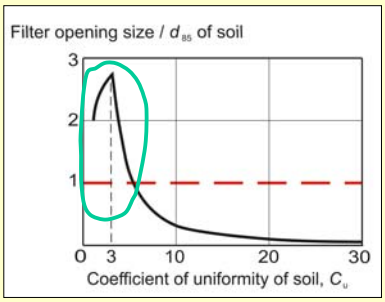
$O_{FILTER} = d_{85\ SOIL}$

The consequences of a retention criterion that does not take into account the coefficient of uniformity can be shown on this graph.



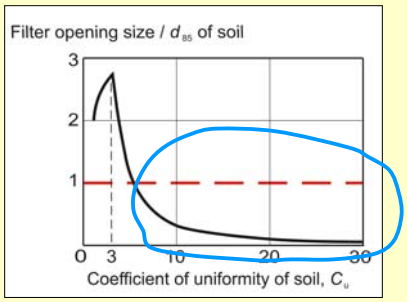
If the coefficient of uniformity is small, the criterion represented by the red line allows filter openings that are too small.

Risk of filter clogging



If the coefficient of uniformity is large, the criterion represented by the red line allows filter openings that are too large.

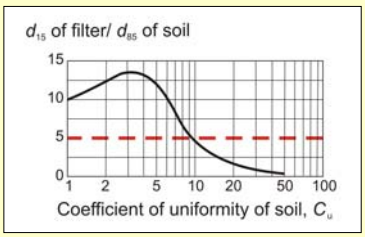
Risk of soil piping



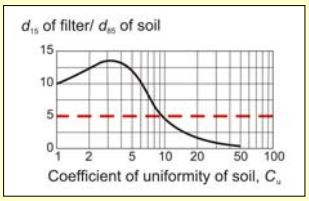
Therefore,
a geotextile filter is safer if it is designed with the retention criterion that takes into account the internal stability of the soil.

The same can be done with granular filters.

The retention criterion developed for geotextile filters has been extended for granular filters.



Here, the vertical axis is d_{15} / d_{85} to be consistent with the practice for granular filters.



RETENTION CRITERION FOR GRANULAR FILTERS

- **This retention criterion is applicable regardless of maximum particle size.**
- **The limitation of Terzaghi's retention criterion to 4.75 mm is eliminated.**
- **The tedious operation of truncating particle size distribution curves at 4.75 mm is eliminated.**

Therefore,
by extending to granular filters the retention criterion developed for geotextile filters, we have obtained a tool for designing granular filters that is simpler and safer than the traditional criterion in the case of soils having a large coefficient of uniformity.

LESSON LEARNED
from this successful method

**What started as technology transfer
from geotechnical engineering to geosynthetics engineering
ended as technology transfer
from geosynthetics engineering to geotechnical engineering.**

**Geosynthetics engineering
is a new discipline
with innovative research that can
benefit a mature discipline
such as geotechnical engineering.**

**Just imitating the great masters
is not the best approach
to solving modern problems.**

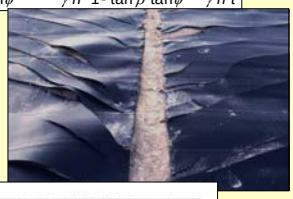
**We do not have to do today
what Terzaghi would have
done 50 years ago.**

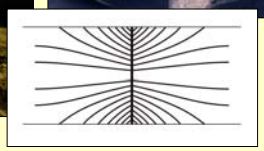
**We need to do today
what Terzaghi would do today.**

CONCLUSION

$$FS = \frac{\tan\delta}{\tan\beta} + \frac{a}{\gamma t \sin\beta} + \frac{t}{h}\frac{\tan\phi/(2\sin\beta\cos^2\beta)}{1-\tan\beta\tan\phi} + \frac{c}{\gamma h}\frac{1/(\sin\beta\cos\beta)}{1-\tan\beta\tan\phi} + \frac{T}{\gamma h t}$$

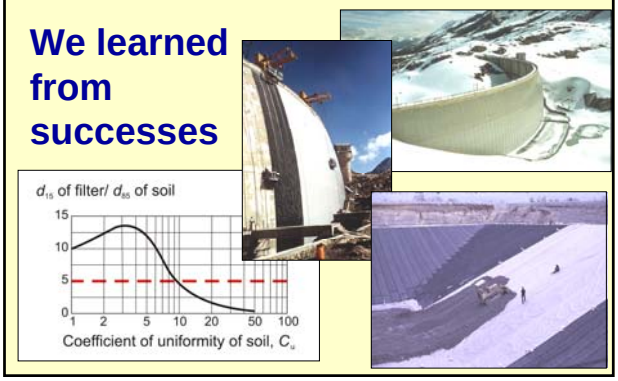
**We learned
from failures**





CONCLUSION

**We learned
from
successes**



CONCLUSION

**Essentially, we learned that
engineering problems
(with geosynthetics or not)
are solved by rational analyses
based on engineering principles,
and good observations,
not by common sense.**

**This is consistent with
the theory and practice approach
advocated by Terzaghi.**

Thank you

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Geosynthetics engineering:

successes, failures and lessons learned

by

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Geosynthetics engineering: successes, failures and lessons learned

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ABSTRACT

This paper presents two cases of failures and two cases of successes related to structures incorporating geosynthetics. Analyses of these cases are presented and lessons learned are discussed. An important lesson learned is that engineering problems, whether they are related to failures or successful applications, can always be solved by following a rational approach, generally including theoretical analyses. In contrast, common sense or "engineering judgment", used without the support of a rational approach, can be misleading, as illustrated by examples. Another lesson learned is that geosynthetics engineering is an integral part of both geotechnical and civil engineering, which results in fruitful technology transfer.

1. INTRODUCTION

1.1. Learning lessons from successes and failures

Terzaghi used a geosynthetic in his last project, the Mission Dam (now called the Terzaghi Dam) in Canada. This application of a geosynthetic was a success in spite of a localized failure, and it provided an opportunity to learn from success and failure.

Geotechnical engineers who do not learn from successes achieved by others will miss opportunities. Geotechnical engineers who do not learn from mistakes made by others will learn from their own mistakes. This should encourage geotechnical engineers to learn lessons from both successes and failures.

The remainder of this introduction is an updated version of comments presented by the author in a preceding paper (Giroud 2000).

1.2. A science, not an art

Geotechnical engineering is an art as much as a science, as many like to say. This statement is incorrect and it may mislead those who are learning about geotechnical engineering. Geotechnical engineering is a science; it is not an art. It is a science because all phenomena of geotechnical engineering can be explained rationally. It is not an art because, in geotechnical engineering, there is no room for personal emotions and abstract imagination. One may object that the word “art”, being used in expressions such as “the art of building”, does apply to geotechnical engineering. In these expressions, the word “art” designates *methods and skills*, generally *derived from practice*. Geotechnical engineering, being an applied science, certainly includes activities that require *methods and skills derived from practice* (e.g. the art of conducting field investigations or, even, the art of writing papers). However, geotechnical engineering itself is a science, an applied science, not an art.

1.3. Rational analyses, not common sense

As a science, geotechnical engineering requires reliable tools. Common sense, which is often invoked by geotechnical engineers, is not a reliable tool, because it is a random collection of beliefs, many of them being bad habits, only justified by tradition. As the origin of the beliefs packaged under the label “common sense” is usually unknown, there is no way to distinguish between the good and the bad. Therefore, common sense cannot be used as a basis for rational decisions in a scientific discipline. The same applies to “engineering judgment”. Design engineers should be particularly circumspect with methods justified solely by common sense or engineering judgment: these two phrases are generally used as a screen to hide the laziness inspired by the difficulty inherent to rational analyses. This is a strategy often used by those who use their “experience” as an excuse for not doing the hard work required by rational analyses.

1.4. Learning from experience

The fact that, in a scientific discipline, all phenomena can be explained rationally on the basis of first principles does not mean that all knowledge must result from logical deduction. In fact, a large proportion of the present scientific knowledge — and this applies to all

disciplines — was generated from experience, often by chance. Rational explanations of the phenomena were eventually developed. This is particularly true for geotechnical engineering, a discipline where the complexity of materials, mechanisms and boundary conditions makes it difficult to predict phenomena only by pure logical deduction. This is also true for geosynthetics engineering, because, in addition to the constraints inherent to geotechnical engineering, there is the fact that the use of geosynthetics is relatively new. As a result, the body of rational knowledge is still under development, while the variety of uses and users creates a wealth of experience from which additional knowledge can be tapped.

It is clear from the above discussion that no opportunity should be missed to learn from experience. However, there is a great difference between "*experience*" and "*learning from experience*". The only way to learn from experience is to analyze available data and incorporate the results of the analyses into an organized body of knowledge. This is particularly true for learning from failures, which constitute the ultimate level of experience. In this paper, case histories are used to show how lessons can be learned from a rational analysis of failures.

Based on the above discussion, it is important to know what a failure is.

1.5. Definition of failure

Asking the question, "What is a failure?", regarding geotechnical structures, often attracts a confusing answer based on "common sense", the magic phrase used every time it appears difficult to develop a rational approach. Indeed, it is not easy to rationally define what a failure is, as seen below.

A failure in a geotechnical structure can affect the entire structure (e.g. a road embankment), a system (e.g. a cover system on a landfill), or a component (e.g. a geosynthetic). Several definitions can be considered for a failure.

A first tentative definition, which is often mentioned, would be:

A structure, system, or component fails if it does not perform its intended function.

Certainly, a structure, system, or component must perform its intended function, but a definition that only contains this requirement is not complete. This definition may be too lax or may be excessive depending on the interpretation of the word “function”. For example, according to the above definition, a retaining structure that exhibits a very large deformation, but still retains the soil, is not considered a failure, regardless of the consequences of the large deformation, if the function of the retaining structure is only understood to be “to retain the soil”. Also, according to the above definition, a geomembrane liner with a very small defect causing an inconsequential leak would be considered a failure if it is understood that the intended function of a liner is to act as an absolute fluid barrier.

At this point, it is important to note that there is a difference between the function of a structure and the function of the geosynthetic in the structure. Thus, while the function of the geomembrane liner in a pond is to act as a fluid barrier, the function of the pond is to contain a liquid. This distinction is illustrated by the case of a small pond where the geomembrane liner is entirely uplifted by gas (a real case). In this case, the pond fails to perform its function of containing liquid while the geomembrane liner does perform its function of acting as a fluid barrier. The function of the structure should not only be clearly defined, it should also be quantified; for example, in the case of a geomembrane-lined pond, the volume of liquid to be contained should be specified. However, this may still not be sufficient, because a single geomembrane bubble in the case of a large pond may not significantly affect the volume of liquid contained, but may affect the long-term performance of the geomembrane (by exposing the geomembrane to sunlight, wind, etc.) and may hamper the operation of the pond. Clearly, a definition of failure based on the function of the structure is too vague to be adequate. Based on a comment made above, the definition of failure should include a number of quantified requirements, i.e. performance criteria. This leads to the second tentative definition:

A structure, system, or component fails if it does not meet its performance criteria.

This definition is better than the first definition because it includes performance criteria, but it is flawed because it implies that performance criteria were set for the considered structure, system, or component, which is not always the case. (In other words, this definition opens the door to the absurd situation where there cannot be a failure because no criteria were set.) Also, this definition implies that the performance criteria, if any, are complete and adequate. For example, simplistic performance criteria such as “no settlement” or “zero leakage” are not

adequate because they cannot be met and, therefore, any behavior is a failure with respect to such criteria.

Clearly, a better definition is needed. Simply combining the two above tentative definitions in a phrase such as "*if it does not perform its intended function and/or does not meet its performance criteria*" does not solve the problems illustrated by the examples presented above.

With this in mind, finally, the proposed definition could be:

> *A structure, system, or component fails if it does not meet complete and adequate performance criteria.*

This definition is technically correct because "*complete and adequate performance criteria*" can be expected to define and quantify, completely and adequately, the intended function of the structure, system or component. A potential drawback of the above definition is that the adjectives "*complete and adequate*" may be subject to interpretation and debate. However, it should be possible to develop guidance regarding the definition and content of "*complete and adequate performance criteria*". Tentatively, the following guidance is proposed.

1.6. Types of failure

To be complete, the criteria should address three potential types of failure: failure to perform the function of the structure, system, or component; disruption of, or nuisance to, operation or use of the structure, system, or component; and threat to the future performance of the structure, system, or component. These three potential types of failure are discussed below.

1.6.1. Failure to perform the function of the structure, system, or component

As stated after the first tentative definition, it is clear that a structure, system, or component must perform its intended function. Therefore, to be *complete and adequate*, performance criteria should include qualitative and quantitative requirements describing the ability of the structure, system, or component to perform its intended function. In the case of a pond, examples of such requirements are: the volume of liquid that the pond must contain and the maximum allowable leakage rate. Examples of cases where these requirements are not met would be: a geomembrane liner uplifted by gas to the extent that the required volume of liquid

cannot be contained; a leak that exceeds the maximum allowable leakage rate; and a very large leak that both exceeds the maximum allowable leakage rate and prevents the pond from containing the required volume of liquid. Such failures can be referred to as "*functional failures*".

1.6.2. Disruption of, or nuisance to, operation or use of the structure, system, or component

Every structure, system, or component is operated or used. Therefore, there are disruptions of, or nuisances to, the operation or use of the structure, system, or component that cannot be tolerated by the operator or user. This kind of failure is often referred to as "*serviceability failure*". Therefore, to be *complete and adequate*, performance criteria should include qualitative and quantitative requirements describing the disruptions of, or nuisances to, the operation or use of the structure, system, or component that cannot be tolerated. In the case of a pond, an example of such requirements is that boats can navigate in all parts of the pond, which means that the localized uplift of the geomembrane liner by gas cannot be tolerated even if it does not affect the ability of the pond to perform its function, which is to contain a certain volume of liquid.

1.6.3. Threat to the future performance of the structure, system, or component

A structure, system, or component must perform its function and be operated or used for a certain period of time. Therefore, to be *complete and adequate*, performance criteria should include qualitative and quantitative requirements describing the ability of the structure, system, or component to perform its function and be operated or used during a certain period of time usually referred to as the design life. Criteria can even include *trends* (such as change in some geomembrane characteristic that indicates degradation, or monitoring of the inclination of a reinforced-soil wall facing) or *symptoms* (such as water seeping through the downstream face of a dam) that may help predict future failure, or even *imminent failure* (if the trends and/or symptoms indicate rapid or even accelerating material and/or structure degradation).

Failures that result from not meeting the requirements related to the future performance of the structure, system, or component can be referred to as "*durability failures*". In the case of a pond, a localized uplift of the geomembrane liner forming a "bubble" may not prevent the

pond from performing its function (which is to contain a certain volume of liquid) and may not hamper the operation and use of the pond. Therefore, the first two types of criteria are met. However, the bubble exposes the geomembrane to sunlight and vandals, which may decrease the ability of the geomembrane liner to perform its function during the entire design life of the pond. Therefore, the performance criteria should include some language treating the development of a geomembrane bubble as a symptom that is not acceptable and requires immediate action because it indicates the beginning of a mechanism that could lead to failure.

1.7. Discussion of the types of failure

The boundaries between the three types of failure are not totally rigid and some criteria may happen to be at the boundary between two types. For example, the deformation of a reinforced soil wall may be only a nuisance to the user if it affects the appearance of the wall face or it may be a failure to perform the function if, due to the deformation of the wall, a foundation that was to be built on the retained soil cannot be built. (The limit may even evolve with time: a wall with a face tilting forward may only be a "nuisance to the use of the structure", however, as the tilting continues to increase, it may become a warning of imminent collapse.) However, the above guidance makes it possible to establish a list of criteria that is *complete*, which is essential. In addition to being *complete*, the criteria should also be *adequate*. Adequate criteria are criteria that are rationally quantified in a way that reflects the performance of the structure and the needs of its operators and users.

It should be noted that the three types of failure mentioned above are different from the two types often mentioned, structural failure and serviceability failure. The terminology "*structural failure and serviceability failure*" is applicable to structures that may collapse when poorly designed and/or constructed (e.g. reinforced-soil structures with a vertical face). This terminology is not applicable to the many types of structures that do not collapse. Clearly, instead of referring to "*structural failure and serviceability failure*", it is more general and more correct to refer to "*functional failure, serviceability failure, and durability failure*", as explained in Sections 1.6.1, 1.6.2 and 1.6.3, respectively. An advantage of the proposed definition of failure (see the end of Section 1.5) is that it makes it possible: (1) to rationally evaluate designs and specifications; and (2) to identify those that are based on incomplete or inadequate performance criteria.

1.8. Failures and successes

It is important to put failures in perspective. In a preceding paper (Giroud 2000), the author estimated that the number of significant failures is less than 0.1% of the number of structures constructed using geosynthetics. Clearly, the geosynthetic discipline has been characterized by success far more than by failure.

Lessons can be learned from both failures and successes. A comprehensive survey of lessons learned from failures and successes was presented in an earlier paper (Giroud 2000). The present paper will focus on two cases of lessons learned from failures (Section 2) and two cases of lessons learned from successes (Section 3).

2. LESSONS LEARNED FROM FAILURES

2.1. Overview

Two case histories of failure associated with geosynthetics are presented. The first case is that of cracking of a geomembrane liner on the side slopes of empty reservoirs. The analysis, which was performed shortly after the failure in 1989, explained the observed pattern of cracks even though the failure mechanism appeared to be complex. The second case is that of the instability of the cover system of a landfill. The analysis, which was performed as part of the investigation of the failure that occurred in 1993, showed that the failure mechanism was different from the mechanism that appeared to be obvious to some of the first observers. In both cases, common sense was misleading and the solution was found by following a rational approach, including the performance of theoretical analyses.

2.2. Cracking of a geomembrane liner

2.2.1. Description of the case

At various locations on the side slopes of several geomembrane-lined reservoirs that were empty, the geomembrane liner underwent severe cracking during very cold weather (–30°C) (Figure 1). A “central crack” (Figure 2), which was wide open due to geomembrane contraction resulting from low temperature, was located next to a seam of the geomembrane

liner (Figure 3). On both sides of the central crack, the geomembrane was shattered (Figures 1 and 2).

Three important observations were made: (1) the central crack was wide open (approximately 30 cm) at mid slope but could not get open at the crest and the toe of the slope because the geomembrane was restrained by an anchor trench at the crest and by ice at the toe; (2) the shattering cracks were oriented upward in the upper half of the slope and downward in the lower half of the slope (Figure 2); and (3) there was no crack along the horizontal axis at mid slope.

Fig. 1: Cracking of the geomembrane liner observed on the side slope of an empty geomembrane-lined reservoir after an extremely cold night. [Photo J.P. Giroud]

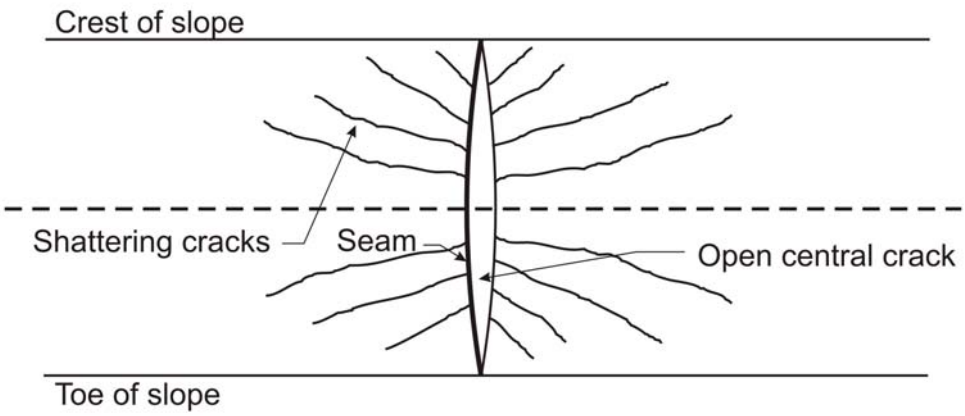

Fig. 2: Summary of the main observations on the side slope of the geomembrane-lined reservoir at the location where cracking of the geomembrane was observed. (Note: No crack was observed along the axis at mid slope, represented by a dashed line.)

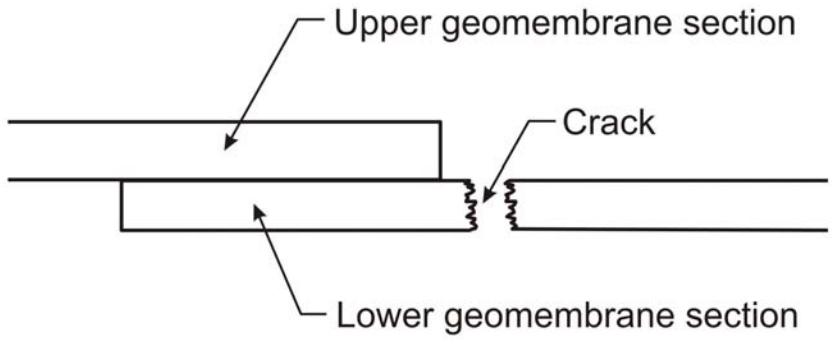

Fig. 3: Cross section of the geomembrane showing the crack located next to a seam, in the lower geomembrane section. (Note: Here, the crack is shown before it got wide open due to thermal contraction of the geomembrane.)

2.2.2. A dilemma and common sense

This failure posed a dilemma: (1) did the central crack occur first and triggered the shattering cracks? or (2) did the shattering cracks develop independently of the central crack? It was important to find the answer for the following reasons: (1) if the opening of the central crack triggered the shattering cracks, then the geomembrane tension played a role, and alleviating the tension could be the solution; (2) in contrast, if the shattering cracks were not linked to the central crack, the geomembrane was defective and had to be replaced. The spectacular nature of the shattering cracks impressed all observers to the point that some of them felt that common sense dictated that the shattering cracks were the main mechanism of failure. Therefore, they recommended replacing the geomembrane by a new geomembrane. The author's position was that only a rational analysis, not common sense, could solve the dilemma.

2.2.3. Summary of the analysis

First, it should be noted that, all thermoplastic geomembranes (e.g. PVC and HDPE geomembranes) become more brittle as temperature decreases. As a result, they become more susceptible to cracking.

The first step of the analysis consisted of explaining why the central crack occurred along a seam. The key to the explanation is the fact that, when a geomembrane is subjected to tensile forces on both sides of a seam, the geomembrane must bend to allow the tensile forces to be aligned and, therefore, balanced (Figure 4). The resulting bending stress is added to the tensile

stress directly induced in the geomembrane by the tensile forces. As a result, there is a concentration of tensile stresses on both sides of the seam (Figure 4). This mechanism has been quantified by Giroud *et al.* (1995a) using the theory of elasticity. They showed that, for the type of seam used in this case, the total stress in the vicinity of the seam is 1.8 times the tensile stress in the geomembrane away from the seam. Clearly, stress concentration due to relatively small bending is significant.

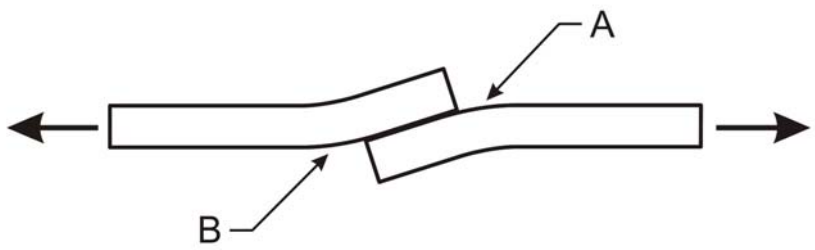

Fig. 4: Cross section of a geomembrane subjected to tensile forces, showing the bending of the geomembrane next to a seam, which ensures that the tensile forces are aligned and, therefore, balanced. Points A and B are the location of maximum tensile stress in the geomembrane.

The phenomenon illustrated in Figure 4 is symmetrical: the same maximum tensile stress occurs on both sides of the seam. Figure 4 explains why cracking may develop next to seams, but does not explain why the central crack developed on only one side of the seam. Regardless of on which side of the seam the crack actually develops, it is noteworthy that a crack may develop next to a seam, even though the seam is not defective.

In the case considered herein, the tensile forces exerted on the geomembrane are due to thermal contraction of the geomembrane resulting from low temperature. It should be noted that thermal contraction *per se* does not generate stresses. However, in the field, thermal contraction always results in tensile stresses in the geomembrane because movements of the geomembrane are restrained by friction between the geomembrane and the underlying soil, anchor trenches, the impounded liquid (especially if it is frozen), etc.

As the air temperature decreases rapidly during the night, it may happen that, during a certain period of time, the temperature is lower in the air above the geomembrane than in the soil beneath the geomembrane. As a result, there is more thermal contraction (hence more tensile stress) on the upper face of the geomembrane than on the lower face. Therefore, in the case considered herein, the tensile stress is not the same at the two potential locations of maximum

tensile stress (A and B in Figure 4). The stress is greater at location A, i.e. on the upper face of the lower geomembrane panel next to the seam (Figure 5). This may explain why a crack could develop in the lower geomembrane panel (Figure 3), next to the seam. There may be other explanations: (1) the upper surface of the geomembrane may have been systematically scratched along the seam during seaming operations; (2) heat induced in the geomembrane during seaming may have caused deterioration of the geomembrane polymer on one side of the seam; and (3) the seam may not be as symmetrical as shown in Figure 4 and an extra flap of the lower geomembrane section may have somehow protected the area of point B (Figure 4). (It should be noted that deterioration of the geomembrane surface due to outdoor exposure was unlikely in the considered case, because the geomembrane failure occurred only a few months after the installation of the geomembrane.)

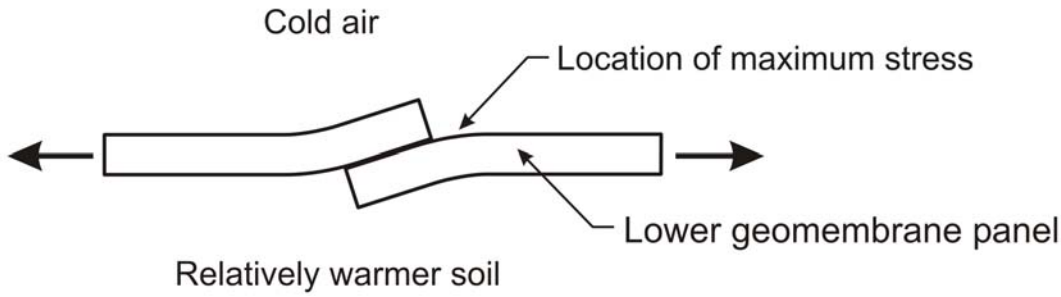

Fig. 5: Cross section of the geomembrane subjected to tensile forces and exposed to cold air on its upper face, showing the location of maximum tensile stress, which occurs in the lower geomembrane section.

The second step of the analysis consisted in demonstrating that the opening of the central crack had triggered the development of the shattering cracks. The model used for the demonstration is shown in Figure 6. This model is based on the assumption that the central crack opened first and shows the distortion of the geomembrane that resulted from the opening of the central crack.

The Mohr's circle for strains at the location *x*, *y* of the model is shown in Figure 7. A well-known property of the Mohr's circle is that the line PM is perpendicular to the maximum strain. Therefore, PM is the crack direction. Using geometric properties of the Mohr's circle, the direction of PM can be expressed analytically. Since it varies as a function of *x* and *y*, a family of curves was obtained (Figure 8). This family of curves is remarkably similar to the

pattern of shattering cracks observed (Figures 1 and 2). Details of the calculations have been presented in earlier papers (Giroud 1994a, 1994b).

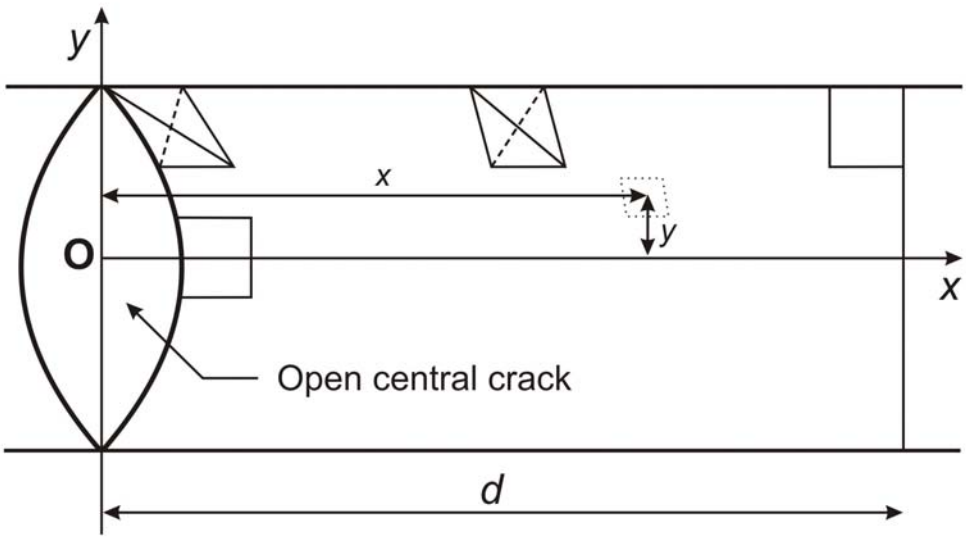

Fig. 6: Model used for the demonstration (Giroud 1994b). (Note: *d* is the distance beyond which there is no distortion of the geomembrane.)

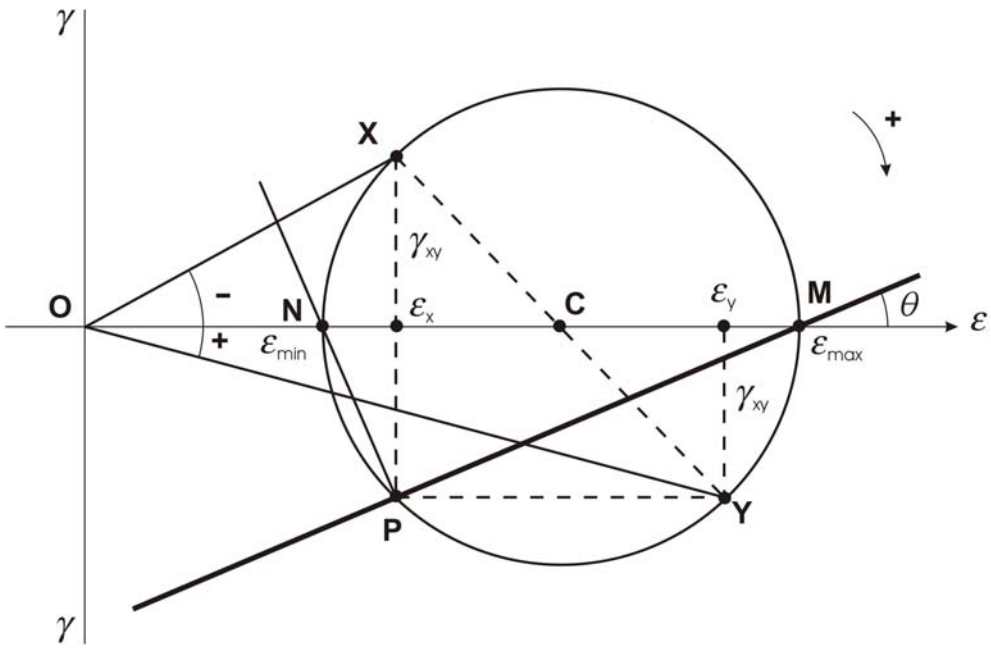

Fig. 7: Mohr's circle for the strains in the geomembrane at the point of abscissa *x* and ordinate *y* in Figure 6 (Giroud 1994b).

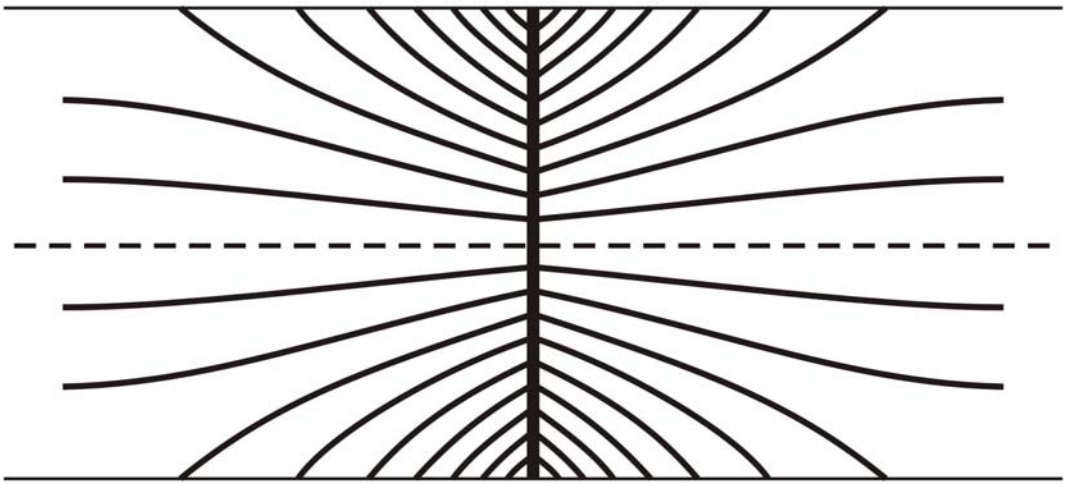

Fig. 8: Pattern of cracks obtained analytically (Giroud 1994b). (Note: The theoretical analysis predicted no crack along the horizontal axis at mid slope (dashed line), which is consistent with the observations.)

2.2.4. Conclusion of the analysis

The result of the analysis summarized above is consistent with the observations; therefore, the analysis is adequate to explain the failure mechanism. This analysis was based on the assumption that the central crack occurred first. Therefore, the assumption was correct and the mechanism of failure can be summarized as follows: due to geomembrane tension (resulting from geomembrane contraction induced by low temperature) and stress concentration next to the seam, a crack developed in the geomembrane next to the seam; due to geomembrane contraction, the central crack opened widely at mid slope; in contrast, the central crack could not open at the crest and the toe of the slope where the geomembrane was restrained; the differential opening of the central crack between mid slope, on one hand, and crest and toe, on the other hand, caused a distortion in the geomembrane, which triggered the formation of shattering cracks, as the geomembrane was made brittle by the low temperature.

One may rightfully argue that a geomembrane should be flexible and be able to withstand distortion without permanent damage. In fact, the geosynthetic industry learned a lesson from this case history as well as similar failures in the late 1980s and early 1990s. As a result, newly formulated geomembranes, less susceptible to cracking, have been developed. However, when the analysis described above was performed, its goal was to solve the dilemma presented in Section 2.2.2. This goal was achieved as discussed below.

2.2.5. Remedial measures

Based on the above analysis, it was concluded that the geomembrane did not need to be replaced provided that the tensile stresses in the geomembrane liner in case of low temperature were alleviated. This was achieved by cutting the geomembrane parallel to the seams (i.e. along the slope) at regular intervals (approximately 20 m) to add a strip of geomembrane forming a fold (Figure 9), thereby increasing the length of the geomembrane. These added strips were called “compensation panels”.

Fig. 9: Compensation panel. [Photo J.P. Giroud]

2.2.6. Lesson learned

The problem posed by the complex mode of failure of the geomembrane was solved by a rational analysis using methods from other engineering disciplines, such as: the theory of elasticity for calculating the tensile stress due to geomembrane bending, and the Mohr’s circle to determine the direction of maximum stresses and strains. Several lessons can be learned from this case history:

– Geosynthetics are an integral part of geotechnical engineering, therefore an integral part of civil engineering. The same methods and approaches are used in all branches of civil engineering, including geosynthetics engineering.

– Geosynthetics engineering is a science, not an art. Even when phenomena (such as geomembrane cracking) exhibit shapes that could be seen in modern art museums, engineers should not be impressed and should use a rational approach, which generally includes theoretical analyses. Theoretical analyses are very powerful and can be used in all engineering circumstances.
– Engineering problems can be solved by rational analyses, not by common sense. In fact, common sense is often misleading because people are accustomed to believe in common sense without questioning its validity.

2.3. Stability of liner or cover systems on slopes

2.3.1. Overview

The liner and cover systems used in landfills, reservoirs and dams are generally layered systems comprising several layers of soil and geosynthetics, such as:

– Soil layer (sometimes reinforced with geogrids or high-strength geotextiles);
– Geotextile filter;
– Geonet drain;
– Geotextile protection; and
– Geomembrane.

More layers are used in the case of double liners.

A slip surface may develop between some of these layers, which results in instability of the system. The driving forces are: gravity (essentially the weight of the soil layer(s) located above the slip surface); and seepage forces caused by water that tend to flow parallel to the slope owing to the presence of an impervious component, the geomembrane. The resisting forces are: the toe buttressing effect (i.e. the strength of the soil at the toe of the slope); the interface shear strength along the slip surface; and the tension in the geosynthetics located above the slip surface, which can be accounted for if these geosynthetics are properly anchored at the crest of the slope (Figure 10).

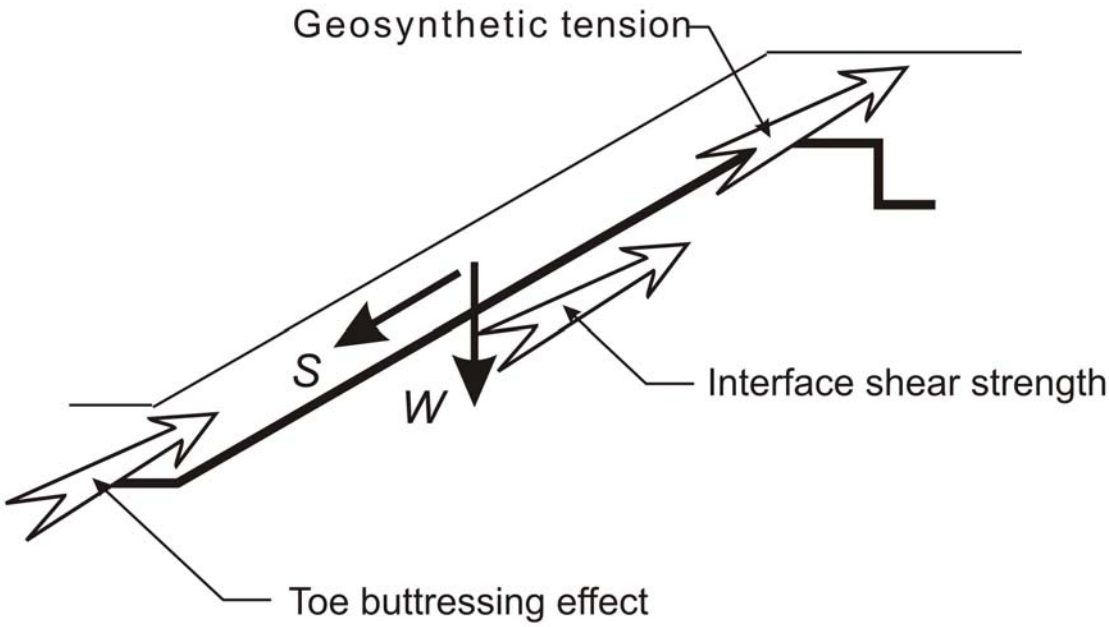

Fig. 10: Liner system on slope: the two causes of instability (the weight of the soil layer, *W*, and the seepage force, *S*) and the three mechanisms that contribute to stability.

2.3.2. Equations for the stability of liner systems on slopes

Stability is an important consideration in liner system design and several methods have been developed to evaluate the stability of liner systems on slopes. Using simplifying (but reasonable) assumptions, it was possible to develop the following equation that makes it possible to calculate a factor of safety that characterizes the stability of a liner system on a slope (Giroud *et al.* 1995b):

$$FS = \frac{\tan\delta}{\tan\beta} + \frac{a}{\gamma t \sin\beta} + \frac{t}{h}\frac{\tan\phi/(2\sin\beta\cos^2\beta)}{1-\tan\beta\tan\phi} + \frac{c}{\gamma h}\frac{1/(\sin\beta\cos\beta)}{1-\tan\beta\tan\phi} + \frac{T}{\gamma h t} \qquad (1)$$

where FS is the factor of safety, δ is the interface friction angle, β is the slope angle, a is the interface adhesion, γ is the unit weight of the soil, t is the thickness of the soil layer, h is the height of the slope, ϕ is it internal friction angle of the soil, c is the cohesion of the soil, and T is the tension in the geosynthetics located above the slip surface and properly anchored at the crest (Figures 10 and 11).

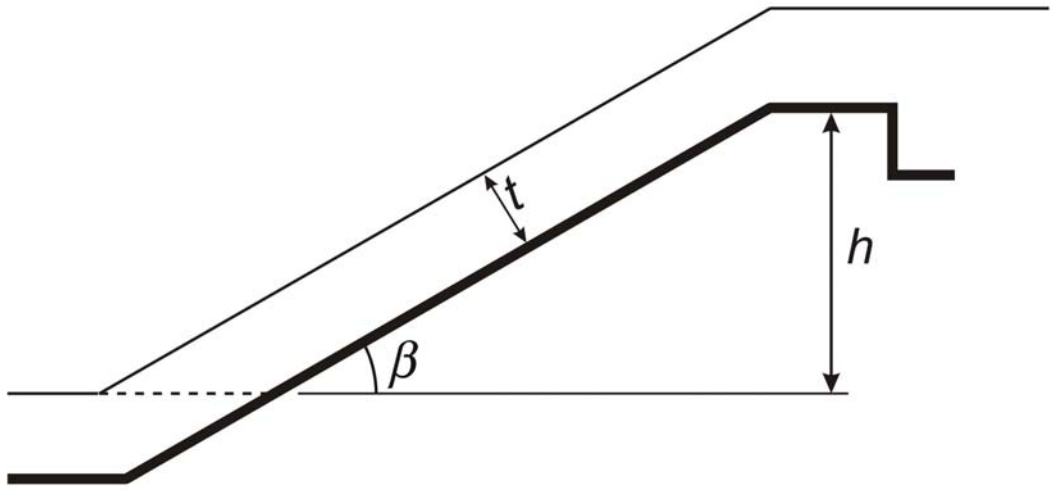

Fig. 11: Liner system on slope: definition of the geometrical parameters. (Note: The liner system is schematically represented: the bold line represents all the geosynthetics located above the slip surface and *t* is the total thickness of soil above the geosynthetics.)

Equation 1 has the advantage of making it possible to quantify the influence of each stability parameter separately:

- the first term quantifies the contribution of the *interface friction angle* to the factor of safety;
- the second term quantifies the contribution of the *interface adhesion* to the factor of safety;
- the third term quantifies the contribution of the *internal friction angle of the soil* to the factor of safety, through the toe buttressing effect;
- the fourth term quantifies the contribution of the *cohesion of the soil* to the factor of safety, through the toe buttressing effect; and
- the fifth term quantifies the contribution to the factor of safety of the *tension in the geosynthetics* located above the slip surface and properly anchored at the crest.

In other words:

- the first and second terms quantify the contribution of the *interface shear strength* along the slip surface to the factor of safety;
- the third and fourth terms quantify the contribution of the *toe buttressing effect* to the factor of safety; and
- the fifth term quantifies the contribution of the *geosynthetic tension* to the factor of safety.

It is important for the design engineer to be able to quantify each term separately because some of the parameters (e.g. interface friction angle) are more reliable than others (e.g. interface adhesion). Also, it is important to quantify the effect of the geosynthetic tension independently from the other stability mechanisms for the following reasons:

- in liner system design, the tension in non-structural geosynthetics such as geomembranes and geonets should in general be neglected;
- in contrast, in forensic analyses, all contributions to the factor of safety, even small, should be quantified; and,
- when structural geosynthetics (e.g. geogrids and high-strength geotextiles) are used to reinforce the soil layer(s) in a liner system on a slope, it is useful to quickly compare the effectiveness of candidate geosynthetics.

In most typical cases, the magnitude of the contribution of the interface shear strength to the factor of safety is much greater than the magnitude of the contribution of the toe buttressing effect.

2.3.3. Effect of water on the stability of liner systems on slopes

Equation 1 is valid for the case where there is no seepage force. A seepage force (Figure 10) develops when the soil layer is saturated. As indicated in Section 2.3.1, the seepage force is parallel to the geomembrane, hence parallel to the slope. It is important to note that the magnitude of the seepage force is independent of the velocity of the water flowing along the slope, whether the water flows slowly in a low-permeability soil or rapidly in a high-permeability soil. The magnitude of the seepage force depends only on the thickness of the soil that is saturated.

A drainage layer in a liner system on a slope contributes to stability by changing the direction of the seepage force in the overlying soil layer from parallel to the slope (which is detrimental to stability) to vertical (which has no more impact on the factor of safety than gravity). However, a seepage force parallel to the slope develops in the drainage layer. A properly designed drainage layer should therefore meet the following two conditions:

- The water thickness in the drainage layer must be less than the thickness of the drainage layer. This condition is essential, because, if the drainage layer is full, it cannot drain water from the overlying soil. As a result, the soil will saturate and a seepage force parallel to the slope will develop.

– The water thickness in the drainage layer must be small to minimize the seepage force in the drainage layer. This is the case when high-transmissivity geosynthetic drains are used.

The seepage force has its maximum value when the soil layer is saturated over its entire thickness, *t*. The factor of safety can then be calculated using the following equations (Giroud *et al.* 1995c):

$$FS_{\mathrm{A}} = \left(\frac{\gamma_{\mathrm{b}}}{\gamma_{\mathrm{sat}}}\right)\frac{\tan\delta_{\mathrm{A}}}{\tan\beta} + \frac{a_{\mathrm{A}}}{\gamma_{\mathrm{sat}}\, t \sin\beta} + \left(\frac{\gamma_{\mathrm{b}}}{\gamma_{\mathrm{sat}}}\right)\frac{t}{h}\frac{\tan\phi/(2\,\sin\beta\,\cos^2\beta)}{1-\tan\beta\tan\phi} + \frac{c}{\gamma_{\mathrm{sat}}\, h}\frac{1/(\sin\beta\cos\beta)}{1-\tan\beta\tan\phi} + \frac{T}{\gamma_{\mathrm{sat}}\, h\, t} \tag{2}$$

$$FS_{\mathrm{B}} = \frac{\tan\delta_{\mathrm{B}}}{\tan\beta} + \frac{a_{\mathrm{B}}}{\gamma_{\mathrm{sat}}\, t\, \sin\beta} + \left(\frac{\gamma_{\mathrm{b}}}{\gamma_{\mathrm{sat}}}\right)\frac{t}{h}\frac{\tan\phi/(2\sin\beta\,\cos^2\beta)}{1-\tan\beta\tan\phi} + \frac{c}{\gamma_{\mathrm{sat}}\, h}\frac{1/(\sin\beta\cos\beta)}{1-\tan\beta\tan\phi} + \frac{T}{\gamma_{\mathrm{sat}}\, h\, t} \tag{3}$$

Where FS_{A} is the factor of safety for a slip surface located above the geomembrane, FS_{B} is the factor of safety for a slip surface located below the geomembrane, δ_{A} is the interface friction angle for a slip surface located above the geomembrane, δ_{B} is the interface friction angle for a slip surface located below the geomembrane, a_{A} is the interface adhesion for a slip surface located above the geomembrane, a_{B} is the interface adhesion for a slip surface located below the geomembrane, γ_{b} is the buoyant unit weight of the soil, and γ_{sat} is the saturated unit weight of the soil.

It is important to note that there are two different equations for calculating the factor of safety: one for the case where the slip surface is located above the geomembrane; and one for the case where the slip surface is located below the geomembrane. The reason for these two different equations is the following (Giroud *et al*. 1995c):

– In the case where there is no seepage force (Equation 1), the normal stress on the slip surface is the same whether the slip surface is above or below the geomembrane. Therefore, the only difference in interface shear strength between the two cases of slip surface would result from a difference in interface friction angle or in interface adhesion,

between above and below the geomembrane. The case where there is no seepage force will serve as a reference for the discussion of the cases where there is a seepage force.

- When there is a seepage force (Equation 2 or 3), there is an increase in shear stress that affects equally a slip surface above the geomembrane and a slip surface below the geomembrane.
- In the case where there is a seepage force and the slip surface is above the geomembrane, there is a decrease in effective normal stress due to the buoyant weight of the soil, which results in a decrease in interface shear strength. This decrease in interface shear strength and the increase in shear stress (mentioned above) result in a decrease in the factor of safety, which is expressed by Equation 2.
- In the case where there is a seepage force and the slip surface is below the geomembrane, there is an increase in normal stress due to the saturated weight of the soil, which results in an increase in interface shear strength. This increase in interface shear strength approximately (even exactly, in some cases) cancels out the increase in shear stress mentioned above. As a result, the factor of safety is not much (or not at all) affected. This is expressed by Equation 3.

Clearly, the situation above the geomembrane is different from the situation below the geomembrane.

More complete equations have been established for the case where the saturated soil thickness is smaller than the thickness of the soil layer (Giroud *et al.* 1995c).

The following two comments can be made:

- Inspection of Equation 2 (i.e. the equation that gives the factor of safety for a slip surface located above the geomembrane) and Equation 3 (i.e. the equation that gives the factor of safety for a slip surface located below the geomembrane) shows that the difference between the two equations occurs only in the first two terms of the equations.
- Inspection of Equation 1 (i.e. the equation that gives the factor of safety when there is no seepage force) and Equation 2 shows that the difference between the two equations occurs only in the first and third terms.
- In most cases of practical interest, the magnitudes of the first two terms of Equations 1 to 3 are far greater than the magnitudes of the other three terms. Furthermore, the second term

is often neglected in slope stability calculations because interface adhesion is often either small or unreliable. Therefore, the impact that water may have on the factor of safety is essentially through the first term of the equation.

Based on the above discussion, the essential difference between Equation 2, on one hand, and Equations 1 and 3, on the other hand, is the γ_b/γ_{sat} ratio in the first term. For most soils, the γ_b/γ_{sat} ratio is between 0.50 and 0.55. Therefore, when the soil layer above the geomembrane is completely saturated:

- the factor of safety for slippage above the geomembrane is decreased by a factor of approximately two, compared to the case where the soil is not saturated; and
- in contrast, the factor of safety for slippage below the geomembrane is virtually unchanged compared to the case where the soil is not saturated.

In other words, water in the soil layer above the geomembrane significantly reduces the factor of safety for a slip surface located above the geomembrane, but has a relatively small effect on the factor of safety for a slip surface located below the geomembrane.

2.3.4. Forensic analysis case history

The cover system for a landfill was constructed in the fall of 1992 with the following cross section from top to bottom:

- 0.1 m of topsoil underlain by 0.2 m of fill, this 0.3 m thick layer being referred to as the cover soil in subsequent discussions;
- a nonwoven geotextile filter;
- a 0.3 m thick sand drainage layer;
- a 0.75 mm thick PVC geomembrane;
- a clay layer with a minimum thickness of 0.45 m; and
- a fill layer of variable thickness over waste.

The slope varied between 1V:3H and 1V:4H depending on the location.

In the spring of 2003, during the first thaw after the winter and a period of rainfall, a slope failure took place on a 1V:4H slope. The geomembrane ruptured at the crest of the slope, near the edge of the anchor trench (Figure 12). The geomembrane portion located on the

downslope side of the rupture had moved downward. It was also noted that snow and ice had accumulated in the swale at the toe of the drainage layer, which could have impaired the functioning of the drainage layer.

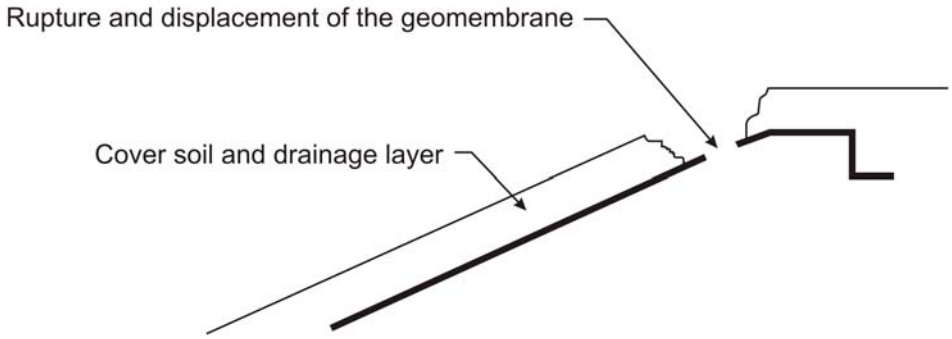

Fig. 12: Landfill cover system failure.

2.3.5. A first explanation and common sense

For the first observers, the explanation was simple:

- Instability occurred after a thaw, which melted the ice previously contained in the cover soil, and after a period of rainfall, which added water in the cover soil.
- The sand drainage layer did not drain water from the cover soil because the toe of the drainage layer was obstructed by snow and ice, and, possibly, because water was still frozen in the drainage layer.
- It is known that water that saturates a soil layer on a slope and tends to flow along the slope (if it is not drained from underneath) causes instability.
- Therefore water present in the soil cover was the cause of the observed instability.

Based on the above explanation, the landfill operator could be partly held liable for the instability for not properly ensuring drainage of the excess water at the end of the winter (in particular, because the toe of the drainage layer was obstructed by snow and ice).

The simple explanation of the instability mentioned above was consistent with experience, consistent with common sense, and, therefore, easily understood and accepted. However, it was incorrect. Furthermore, the “simple explanation” failed to address the fact that the slide did not occur on the steepest slope of the landfill cover.

2.3.6. Explanation based on rational analysis

The real explanation was provided neither by common sense nor by engineering judgment. The real explanation was derived from the theoretical analysis of the effect of water on the stability of liner systems on slopes presented in Section 2.3.3:

- Since the geomembrane had ruptured near the top of the slope and the geomembrane had moved downward, slippage had occurred at the interface between the geomembrane and the underlying soil (i.e. below the geomembrane).
- Water flowing along a slope does not significantly affect the factor of safety for slippage below the geomembrane.
- Therefore, the failure was probably not caused by water flowing along the slope.

Additional investigation, including a review of the construction certification report, showed that (due to heavy rainfalls during construction) the water content of the clay underlying the geomembrane was excessively high (i.e. 26 to 28 %) prior to installing the geomembrane in the area affected by the slide (where the slope was 1V:4H, as mentioned in Section 2.3.4). In contrast, the clay water content was normal (i.e. 21 to 24 %) in other areas; in these areas, there was no slide, regardless of the slope (1V:4H to 1V:3H).

As confirmed by further analysis, including shear box testing with freeze-thaw simulation, the mechanism was:

- Due to frost in the winter, water vapor migrated toward the cold geomembrane in the soil underlying the geomembrane. As a result, ice formed beneath the geomembrane. This is similar to the classical mechanism of ice formation beneath road pavements.
- The ice, sticking to the geomembrane, ensured stability of the liner system during the winter.
- During the thaw, the ice melted beneath the geomembrane, which resulted in very low interface shear strength between the geomembrane and the underlying clay-water mixture, hence the instability.

This explains why the instability occurred and why it occurred where it occurred.

2.3.7. Lessons learned from liner stability on slopes

The following lessons were learned from the theoretical analysis of liner system stability on slopes and the forensic analysis of the observed failure:

- Common sense is often wrong and can be misleading. Common sense should not be used as a basis for engineering decisions.
- Water, even though it is often involved in slope instability, is not always the culprit. Whereas common sense would always consider water as the culprit if a slope is unstable, a theoretical analysis makes it possible to determine under which circumstances water causes instability and under which circumstances it does not.
- Engineering problems can always be addressed using theoretical analyses. These analyses, if properly conducted, give reliable answers. The solution of engineering problems generally does not require original theoretical analyses. The considerable body of knowledge accumulated in geotechnical engineering and geosynthetics engineering makes it possible to use theoretical results to solve practical problems.
- Complete construction records provide information that is essential for forensic analyses.

3. LESSONS LEARNED FROM SUCCESSES

3.1. Overview

Geosynthetics have now pervaded all branches of geotechnical engineering. They have been used in more than one million projects and, today, one thousandth of the surface of Europe is covered with geosynthetics. Geosynthetics are now routinely used in retaining structures, slope stabilization, landfills, dams, reservoirs, mining applications, canals, bank protection, coastal works, embankments, roads, railway tracks, tunnels, underwater construction, erosion control, drainage, filtration, soil reinforcement, soil improvement, pile foundations, etc. Two examples of successful uses of geosynthetics are presented and lessons learned from these examples are discussed. The first example is the rehabilitation of concrete dams using geosynthetics. It will be shown that, in some circumstances and with properly selected materials, the durability of some geosynthetics can be at least equal to that of traditional construction materials. The second example is the design method developed for the selection of geotextile filters. It will be shown that technology transfer can be beneficial, not only from

geotechnical engineering to geosynthetics engineering, but also from geosynthetics engineering to geotechnical engineering.

3.2. Rehabilitation of concrete dams

3.2.1. Description of the application

A number of concrete dams constructed in the first half of the 20th century have been rehabilitated because they suffered from concrete deterioration. The main cause of concrete deterioration is frost action, in the case of dams located at high altitude or in cold regions. Another cause of concrete deterioration is alkali-aggregate reaction, where alkali from the cement reacts with certain types of aggregate. Water promotes concrete deterioration: concrete saturated with water deteriorates as water expands under freezing temperatures; and water facilitates migration of alkali from the cement to the aggregate. Today, low-alkali-content cement is available for use with alkali-sensitive aggregate, but this was not the case in the first half of the 20th century.

Concrete deterioration may be associated with an increase in the rate of leakage through the dam. In some cases, the rate of leakage detected in monitoring galleries has increased by a factor of the order of 10 as a result of concrete deterioration.

3.2.2. The geosynthetic solution

The geosynthetic solution that is used for the rehabilitation of concrete dams consists in waterproofing the face of the dam and draining out of the dam the water that has saturated the concrete over the years. Typically, the face of the dam (which is generally quasi-vertical in concrete dams) is lined using a geomembrane-geotextile composite. The geomembrane component is in contact with the impounded water and the geotextile component is in contact with the face of the dam. The geomembrane and the geotextile components of the composite are heat-bonded together in the manufacturing plant. The geomembrane typically used in this application is a PVC geomembrane with a thickness of 2 to 3 mm, most generally 2.5 mm. This thickness does not include the thickness of the geotextile heat-bonded to the geomembrane. The geotextile is typically a polyester needle-punched nonwoven geotextile with a mass per unit area of 500 g/m^2. More details are provided by Cazzuffi *et al.* (1993) and Cancelli & Cazzuffi (1994).

The geomembrane and the geotextile components of the geomembrane-geotextile composite perform complementary functions. The geomembrane performs the barrier function and the geotextile performs the drainage function. To that end, the geotextile is connected to internal galleries used to monitor leakage through the dam. Furthermore, the geotextile component protects the geomembrane from mechanical damage by irregularities of the dam face and reinforces the geomembrane, thereby reducing the creeping or sagging of the geomembrane along the quasi-vertical dam face.

In some cases, additional drainage capacity is provided by placing a geonet or a thick needle-punched nonwoven geotextile between the geomembrane-geotextile composite and the dam face. The case where a geonet is used is illustrated in Figure 13. The additional geonet or geotextile is connected to the internal galleries used to monitor leakage. The addition of a geonet or a thick needle-punched nonwoven geotextile between the geomembrane and the face of the dam further protects the geomembrane from mechanical damage by irregularities of the dam face.

The geosynthetic rehabilitation technique for concrete dams may be summarized as follows:

- The concrete at the face of the dam is locally repaired, as necessary.
- The face of the dam is lined with a geomembrane-geotextile composite.
- The geomembrane is acting as a barrier between the water and concrete, and the geotextile is acting as a drainage layer while also reinforcing the geomembrane. Additionally, a geonet or a thick geotextile may be placed between the dam face and the composite to augment the drainage capacity.
- The geomembrane (acting as a water barrier) and the drainage layer (by collecting water from the concrete) allow the concrete to progressively dry (which may take a long time).
- Removal of water from concrete eliminates the causes of concrete deterioration: frost action, and alkali-aggregate reaction (in cases where the aggregate is alkali-sensitive).
- The geomembrane also decreases the rate of leakage associated with deterioration (e.g. by a factor of the order of 10 or more in cases where leakage was significant).

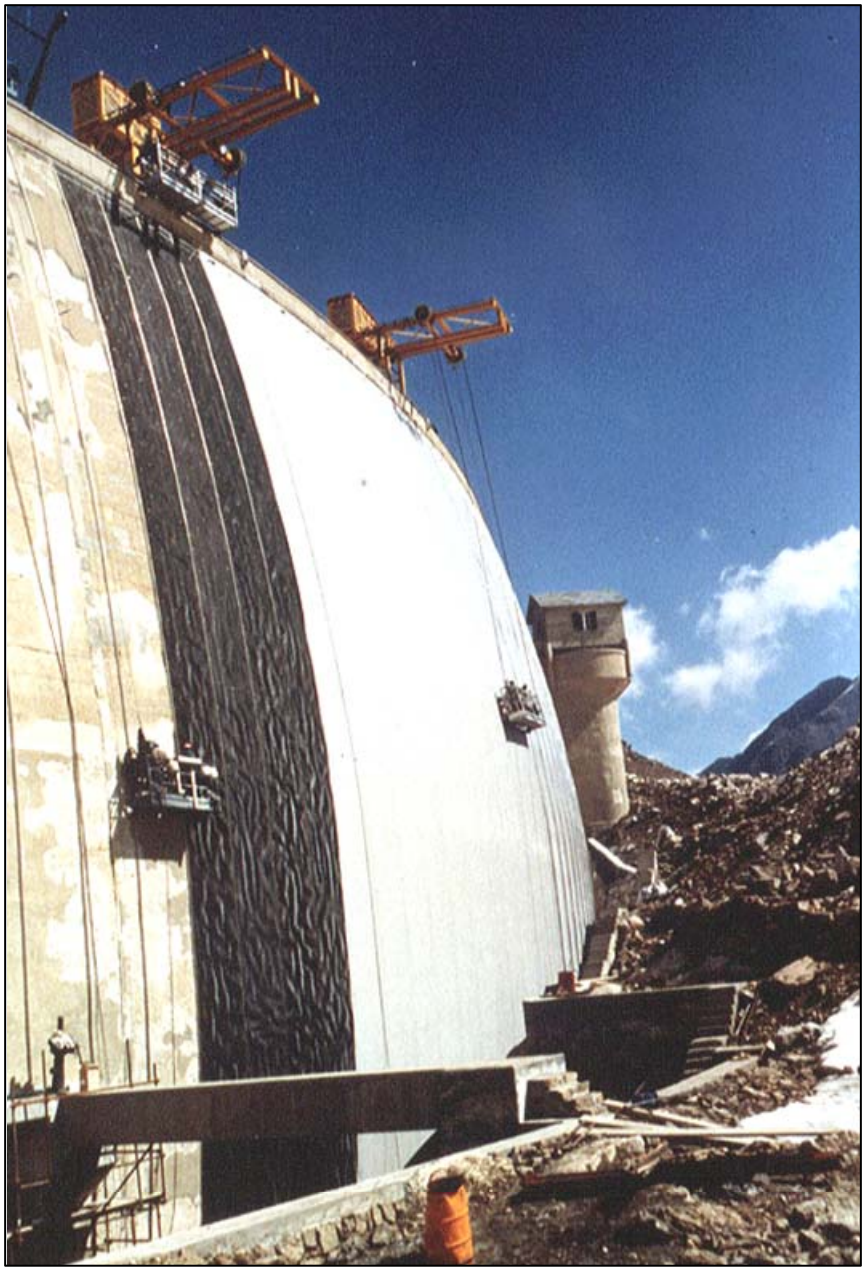

Fig. 13: Rehabilitation of a concrete dam, from left to right: concrete face repaired locally to smoothen surface irregularities, geonet drainage layer (black color), and geomembrane-geotextile composite with only the geomembrane component visible (light grey color). [Courtesy of D. Cazzuffi and A. Scuero]

3.2.3. Experience and durability

The dam rehabilitation technique described in Section 3.2.2 was developed in Italy and has been used in a number of countries. Concrete dams up to 174 m high have been rehabilitated using this technique. Significant experience has been gained since the first application of this technique in 1970 and since the first use, with this technique, of the geomembrane-geotextile composite in 1979.

The dams that were rehabilitated using the geosynthetic rehabilitation technique were typically constructed 40 to 60 years prior to rehabilitation. In a few cases, this interval was shorter (down to 15 years in one case). In other words, the time period during which concrete deterioration occurred is generally of the order of 50 years, and is sometimes less. It is interesting to compare this period with the expected durability of the geosynthetics used.

The durability of the geomembrane-geotextile composite has been a major consideration in the development of the geosynthetic dam-rehabilitation technique. A great amount of care and expertise has been involved in the selection of the plasticizer used in the composition of the PVC geomembrane to ensure the durability of the geomembrane. Indeed, migration of plasticizer out of the geomembrane is the main mechanism of PVC geomembrane deterioration. Durability is critical in the geosynthetic dam-rehabilitation technique considering the harsh exposure conditions, in particular at the water surface and above: the geomembrane is not protected from sunlight and cold weather, and there is a risk of mechanical damage by wind, hail, floating ice, floating debris, etc.

Tests are periodically conducted on samples of the geomembrane-geotextile composite from several rehabilitated dams (Cazzuffi 1998). In particular, plasticizer migration is periodically measured. Based on these tests, a service life of at least 50 years can be conservatively predicted for the geomembrane-geotextile composite. It is important to note that the durability of the exposed composite geomembrane is at least equivalent to the durability of concrete exposed to the same conditions. It is also important to note that, during its entire service life, the composite geomembrane performs its barrier function to the full extent, thereby reducing leakage to a very small amount during its entire service life. In contrast, in the absence of a geomembrane, leakage may increase progressively as concrete deteriorates. Furthermore, the geosynthetics lining the face of a dam can easily be replaced at the end of their service life, whereas the concrete cannot. Therefore, the use of the geosynthetic lining system indefinitely increases the durability of the dam. Also, performance monitoring is facilitated by the use of the geosynthetic lining system on the face of the dam. Considering the typical situation of a high and quasi-vertical dam face, it is easier to take geosynthetic samples than concrete samples. Therefore, it is easier to evaluate the condition of a geosynthetic than to evaluate the condition of concrete from the face of a large dam.

More generally, it is interesting to mention the experience gained in the use of geosynthetics in large dams (whether new or rehabilitated). The geomembrane experience in large dams as of 2005 is the following, for the two types of geomembrane that the most often used in dams: 32 years with PVC geomembranes, and 27 years with bituminous geomembranes. The experience for geotextiles performing critical functions (e.g. internal filter) in large dams is 35 years. For both geomembranes and geotextiles the oldest applications are currently still in service. Clearly, there is significant experience in the use of geosynthetics in large dams.

3.2.4. Lesson learned from this successful application

The geosynthetic technique for concrete dam rehabilitation provides an excellent example of complementarity between traditional and innovative construction materials. Concrete provides the strength and the geomembrane provides the impermeability. Together, they provide the durability.

It is interesting to note that, in this application, the durability of a synthetic material is at least equivalent to that of a traditional construction material, whereas common sense dictates the opposite. In fact, a serious problem posed by the aging of a traditional construction material has been solved using a geosynthetic. The rehabilitation technique discussed above shows that the durability of geosynthetics is not a problem when the geosynthetics are properly selected and properly used.

As a final note, it should be mentioned that the technology developed for dam rehabilitation has been so successful that it has also been used for the construction of new dams using roller-compacted concrete (RCC).

3.3. Criteria for the design of filters

3.3.1. Filtration and common sense

A filter used in geotechnical engineering must have openings small enough to retain the soil and, at the same time, must be permeable enough to allow water to pass as freely as possible. In other words, the filter must meet both a retention criterion and a permeability criterion. These two criteria are, to some extent, contradictory because the permeability of a filter increases with increasing opening sizes, whereas retention decreases with increasing opening sizes. However, in the majority of cases, it is possible to find a filter material, whether

granular or geotextile, that has openings small enough to retain the soil and yet, at the same time, large enough to ensure that the filter permeability is sufficiently high for the considered case. In other words, it is generally possible to find a filter that meets both the retention criterion and the permeability criterion. In this section, only the retention criterion is discussed.

Common sense dictates that, to retain the soil, the filter must prevent the migration of soil particles and, therefore, the largest opening of the filter must be smaller than the smallest soil particle. However, common sense, as is often the case, is wrong. Soil retention does not require that the migration of all soil particles be prevented. Soil retention simply requires that the soil behind the filter remain stable; in other words, some small particles may migrate into and/or through the filter, provided that this migration does not affect the soil structure (i.e. does not cause any movement of the soil mass). The soil structure is then said to be "internally stable". (Of course, the filter and the drainage medium located downstream of the filter should be such that they can accommodate the migrating particles without significant clogging.) It should be noted that the "smallest" soil particle is potentially so small that the common sense requirement mentioned above would lead to selecting a quasi-impermeable membrane as a filter, which is absurd because it could not meet the permeability criterion. The common sense requirement that openings should be smaller than the smallest particles only applies to sieving, where particles are constantly agitated, until they pass if they can.

3.3.2. The geotechnical approach to filtration

The above discussion is not unknown to geotechnical engineers. They are accustomed to designing granular filters or selecting the openings of perforated drainage pipes using not the size of the "smallest" soil particle, but the size of a particle that is almost the largest soil particle: they use d_{85}, the size of the soil particle that is larger than 85% by mass of the soil particles. (d_{85} is used rather than d_{100}, the size of the largest soil particle, because the measurement of d_{100} is likely to fluctuate significantly from one sample to another if there are only a few large particles that may be present in one sample and not in another, whereas the value of d_{85} is less likely to fluctuate due to the large number of particles that have that size in a typical soil sample.)

Clearly, the traditional approach in geotechnical engineering is to consider that, if the quasi-coarsest soil particles (defined by d_{85}) are retained, the entire soil is retained; in other words,

the coarsest soil particles form a matrix that entraps other soil particles and prevents them from moving. Furthermore, this mechanism must work at every particle size level for the soil structure to be internally stable; in other words, particles at any size level must be entrapped in the matrix formed by particles of a larger size. Therefore, it is implicitly assumed that the soil contains a fair share of particles of each size; in other words, it is assumed that the particle size distribution of the soil is continuous (i.e. it is assumed that the soil is not gap-graded). A gap-graded soil is a soil that has a particle size distribution gap between two groups of particles: a group of coarser particles and a group of finer particles. In the case of a gap-graded soil that contains a large proportion of coarser particles and a proportion of finer particles too small to fill the voids between coarser particles, it is clear that the finer particles can migrate between the coarser ones and, therefore, are not entrapped.

Traditionally, the retention criterion for granular filters is written as follows:

$$d_{15\text{filter}} < 5\, d_{85\text{S}} \tag{4}$$

where $d_{15\text{filter}}$ is the size of the filter particle that is larger than 15% by mass of the filter particles, and $d_{85\text{S}}$ is the size of the soil particle that is larger than 85% by mass of the soil particles.

In a granular medium, such as a granular filter, the size of the openings is approximately one fifth of d_{15}. Therefore, Equation 4 can be written as follows:

$$O_{\text{GRA}} < d_{85\text{S}} \tag{5}$$

where O_{GRA} is the opening size of the granular filter.

Equation 5 is consistent with the discussion presented above, i.e. Equation 5 shows that the retention criterion for granular filters (traditionally expressed using Equation 4) means that soil retention is ensured if the filter opening size is less than the d_{85} of the soil. In fact, it is likely that, during the research work that led to the development of the retention criterion for granular filters, Equation 5 was developed before Equation 4. However, Equation 4 is the only one known, because it is elegant, and the only one used, because it is practical. Terzaghi played a key role in the development of the criteria for granular filters, criteria often referred to as Terzaghi's criteria.

3.3.3. From geotechnical engineering to geosynthetics engineering

As many of the design methods used in geosynthetics engineering were developed from methods used in geotechnical engineering, it appeared natural in the early days of geosynthetics engineering to propose, for geotextile filters, the following retention criterion derived from Equation 5, i.e. derived from the retention criterion for granular filters:

$$O_{95} < d_{85S} \tag{6}$$

where O_{95} is the geotextile apparent opening size (AOS). (It should be noted that O_{95} is traditionally used rather than O_{100} because the measurement of O_{95} is more reliable than the measurement of O_{100}, just as d_{85} is used rather than d_{100}, as discussed above.)

It seems legitimate to use the same type of retention criterion for geotextile filters and granular filters. But, is it safe?

Based on the discussions presented in Sections 3.3.1 and 3.3.2, the internal stability of the soil is an essential consideration in filter design. Nevertheless, the only soil parameter present in Equation 4 or 5 is d_{85S}, which characterizes the particle size distribution. Thus, Equations 4 and 5 ignore the internal stability of the soil.

Two soils may have the same d_{85S} but different degrees of internal stability. If the same granular filter is used for these two soils (based on Equation 4), there is a possibility that the filter openings will be too large for one of the soils. If the filter openings are larger than they should be, some soil particles migrate. These particles are more likely to be entrapped in a granular filter than in a geotextile filter, because granular filters are much thicker than geotextile filters and their porosity (typically 30%) is less than that of geotextile filters (typically 90% for needle-punched nonwovens). As a result, the opening size of the granular filter decreases as soil particles become entrapped. Equilibrium is reached when the opening size of the partially clogged granular filter has reached the appropriate value required to retain the soil.

In contrast, a geotextile filter being thin and having a large porosity is less likely than a granular filter to entrap particles or to be significantly modified by those entrapped. Therefore, it is possible that granular filters are more “forgiving” than geotextile filters, i.e. a granular

filter is more likely to function than a geotextile filter if its openings are too large. This difference between geotextile filters and granular filters has created an incentive for developing, for geotextile filters, a retention criterion that is more accurate than the criterion traditionally used for granular filters. Based on the above discussion, this more accurate criterion must take into account the internal stability of the soil.

3.3.4. Development of a retention criterion for geotextile filters

As indicated in Section 3.3.1, a geotechnical filter (i.e. a granular filter or a geotextile filter) can work only if the retained soil is internally stable. The internal stability of a soil depends on many parameters. A soil can be internally stable if there is sufficient cohesion between its particles. Cohesive soils are not discussed herein; only cohesionless soils are considered. Also, large external forces (especially repeated forces such as those resulting from wave action) may disorganize any soil, even a very stable one. Only filters subjected to steady flow in porous media are considered herein, such as filters used in drainage systems.

The problem with Equation 6, derived from geotechnical engineering practice, is that it implicitly assumes, as mentioned above, that the soil is internally stable, provided that its particle size distribution curve is continuous. This assumption is not necessarily correct. Continuity of the particle size distribution curve is a necessary, but not sufficient, condition to ensure the internal stability of a cohesionless soil. For a cohesionless soil to be internally stable, particles of any given size must be entrapped in the matrix formed by particles of a larger size, as mentioned in Section 3.3.1. As indicated by Giroud (1982), this is only possible if the particle size distribution of the soil has a coefficient of uniformity of 3 or less ($C_u \leq 3$). If a soil has a coefficient of uniformity greater than 3, there are not enough of the largest particles to form a matrix wherein smaller particles are interlocked. In that case, it is only at a lower level of particle size that there exists a matrix of particles wherein the smaller particles are interlocked. The above considerations form the basis for the development of the retention criterion for geotextile filters. They can be summarized as follows:

- If the coefficient of uniformity of the soil is 3 or less ($C_u \leq 3$), a geotextile filter that just retains the largest soil particles is adequate. It retains all the soil because, in the case where $C_u \leq 3$, all soil particles are entrapped in the matrix formed by the largest soil particles.
- If the coefficient of uniformity of the soil is greater than 3 ($C_u > 3$), a geotextile filter that just retains the largest soil particles does not retain all the soil. In this case, smaller particles are not entrapped within a matrix formed by the largest particles and they pass

through the geotextile filter if they are dragged by flowing water. Clearly, if the coefficient of uniformity of the soil is greater than 3, the geotextile filter must be designed to retain not the largest soil particles, but the particles that form a matrix where the smaller particles are entrapped. In other words, if the coefficient of uniformity of the soil is greater than 3, the design of the filter should ignore the largest particles and only consider the particles that, if they were alone, would form an internally stable soil (i.e. a soil with a coefficient of uniformity of 3). As shown in Figure 14, this leads to selecting a filter that just retains soil particles with a certain size d_{max}. (Figure 14b is derived from Figure 14a by truncating the particle size distribution of the soil. This is achieved by ignoring particles greater than d_{max}, where d_{max} is such that the particles smaller than d_{max} form a soil with a coefficient of uniformity of 3.)

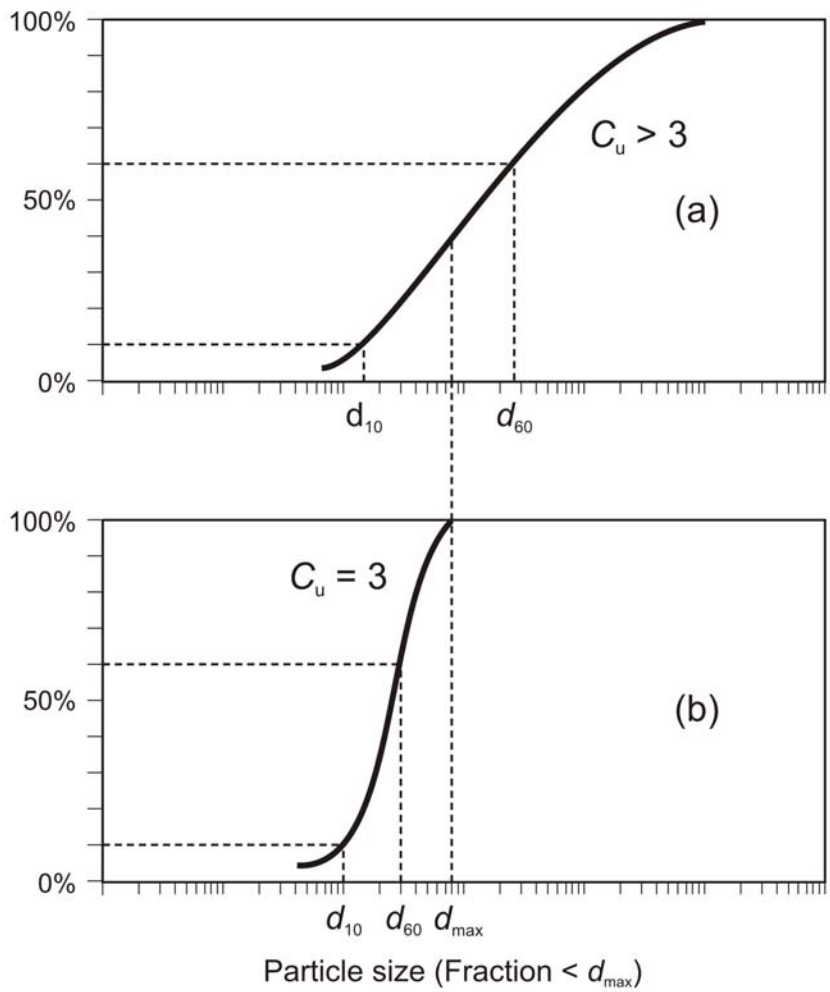

Fig. 14: Determination of the particle size that a geotextile filter should retain if the coefficient of uniformity of the soil is greater than 3: (a) particle size distribution curve of the soil; (b) particle size distribution curve of the fraction of the soil that is internally stable. (Note: The coefficient of uniformity is defined as d_{60} / d_{10} .)

In the above demonstration, the following expression was used several times: “selecting a filter that just retains” certain soil particles. This expression seems to imply, but intentionally does not exactly state, that such a filter should have its maximum openings equal to the considered soil particle size. In fact, this would be the case only for a loose soil. In the case of a dense soil, particles tend to interlock and the filter openings need to be significantly larger than the particle sizes to allow the particles to pass. As shown by Giroud (1982), larger openings could be used for a dense soil. Essentially, in the case of a loose soil, the filter openings must be just smaller than the size of the particles that the filter is intended to retain, whereas, in the case of the same soil in a dense state, the filter openings can be twice as large as the particles the filter is intended to retain.

Two essential conditions were established in the above demonstrations: (1) the filter must retain the largest soil particle of the fine fraction of the soil that has a coefficient of uniformity of 3 (i.e. d_{max} in Figure 14); and (2) to retain particles of a certain size, the filter openings can be as large as the particle size if the soil is in a loose state and as large as twice the particle size if the soil is in a dense state. Giroud (1982) has mathematically expressed these two conditions. The retention criterion thus obtained is presented in Figure 15 for the case of dense soils.

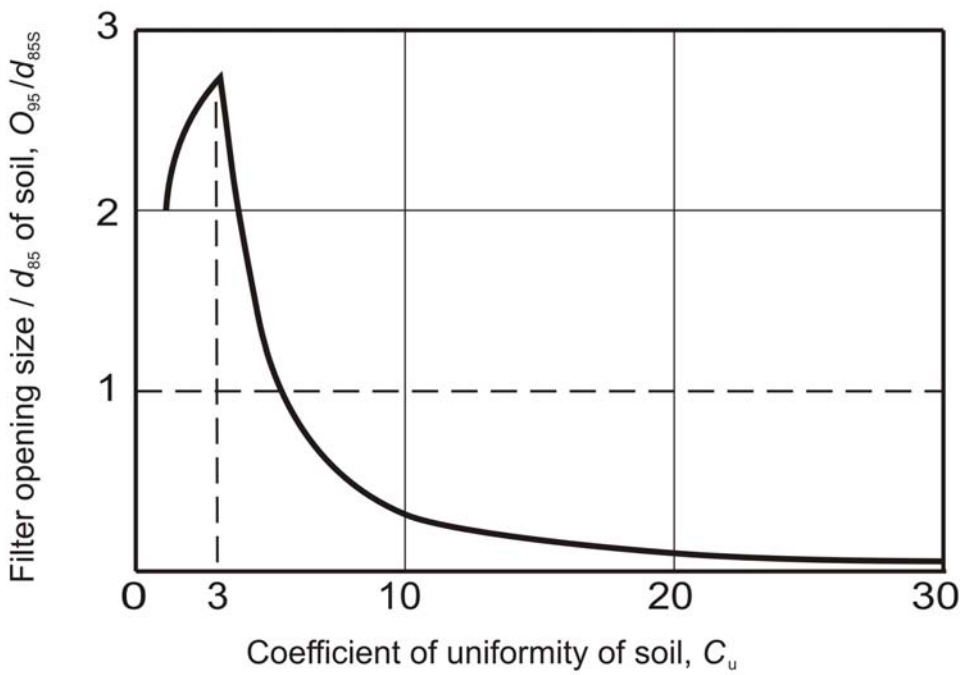

Fig. 15: Retention criterion for geotextile filters (Giroud 1982) for the case of dense soils (solid curve). (The retention criterion for geotextile filters (Equation 6) adapted from the classical Terzaghi retention criterion for granular filters is represented by the dashed line).

Figure 15 shows a very important result. For values of the coefficient of uniformity of the soil, C_u, greater than approximately 5, the required value of the geotextile opening size, O_{95}, is less than d_{85S}. In these cases, it would be dangerous to design a filter using Equation 6: the soil would not be retained (unless it had high cohesion). Figure 15 also shows that soils with a small coefficient of uniformity (i.e. soils that are naturally rather stable) are retained by filters that have openings larger than the largest soil particles.

3.3.5. Arbitrary truncation and automatic truncation

The analysis presented above shows that adapting a criterion used in geotechnical engineering for granular filters may be dangerous for geotextile filters, because this criterion may lead to using geotextile filters with openings larger than they should be. This point deserves a comment. How could a criterion that can be unsafe for geotextile filters be safe for granular filters? A first answer to this question was given at the end of Section 3.3.3 where it was indicated that a granular filter is likely to be more “forgiving” than a geotextile filter when openings are larger than they should be, i.e. a granular filter is more likely to function than a geotextile filter if its openings are too large. However, is a granular filter always “forgiving”? To answer this question, one may assume that a granular filter may function even though its openings are too large if the discrepancy between the retention criterion that takes the internal soil stability into account (solid curve in Figure 15) and the simple retention criterion that does not (dashed line in Figure 15) is relatively small. Figure 15 shows that this discrepancy increases as the soil coefficient of uniformity increases. Therefore, it may be inferred that the greater the coefficient of uniformity of the soil, the less likely is the granular filter to be forgiving, if it is designed using the classical retention criterion expressed by Equation 4.

In fact, it is to address this problem that geotechnical engineers use the retention criterion expressed by Equation 4, on a particle size distribution curve truncated at 4.75 mm, in the case of soils that contain particles larger than 4.75 mm. In other words, geotechnical engineers ignore soil particles larger than 4.75 mm when they design granular filters. By doing so, they artificially reduce the coefficient of uniformity of the soil they use in filter design. This approach is similar to what was done in Figure 14, as part of the establishment of the geotextile filter retention criterion. However, there is a major difference between the development of the retention criterion for geotextile filters and the practice of truncation in geotechnical engineering: (1) in the establishment of the geotextile filter criterion, the separation between the coarse fraction (which is ignored in design) and the fine fraction

(which is considered in design) depends on the particle size distribution curve of the considered soil, based on a rational consideration (i.e. the internal stability of the fine fraction); whereas (2) the separation traditionally practiced in geotechnical engineering is done for a fixed value (4.75 mm), which is arbitrary. Furthermore, in the retention criterion for geotextiles, the separation of the soil particle size distribution in two parts (i.e. the truncation shown in Figure 14) is automatically included in the retention criterion presented in Figure 15. As a result, the user of the retention criterion for geotextiles does not have to actually truncate the particle size distribution curve of the soil (contrary to what geotechnical engineers do at 4.75 mm when they design granular filters). In fact, it is remarkable that, even if the user of the geotextile retention criterion truncates the particle size distribution curve of the soil, the calculated value of the filter opening size is unchanged, as shown by Giroud (2003), because the filter criterion for geotextiles adjusts itself automatically to the particle size distribution, whether it is truncated or not.

At this point, one may wonder why the arbitrary 4.75 mm truncation procedure has been successful with granular filters. The following explanation has been proposed by Giroud (1996, 2003). After truncation at 4.75 mm, any soil with a continuous particle size distribution curve and without fines has a coefficient of uniformity of the order of 5. According to Figure 15, the ratio O_{95} /d_{85S} should then be less than one. In other words, the filter opening size should then be less than the d_{85} of the soil to be retained. This is equivalent to the classical retention criterion for granular filters (Equation 4) as indicated at the end of Section 3.3.2. This explains why the truncation at 4.75 mm coupled with the classical retention criterion for granular filters (Equation 5) works for soils that do not contain fines (i.e. no particle smaller than 0.075 mm).

3.3.6. From geosynthetics engineering to geotechnical engineering

The demonstration presented above should not be construed as a justification for the truncation at 4.75 mm. As shown by Giroud (2003), in the case of soils containing more than 10% fines, the truncation method leads to excessively large values of the filter opening size, hence a high risk of soil piping. Therefore, it is recommended to eliminate the practice of truncation at 4.75 mm from the design of granular filters. To that end, a method similar to the method used for geotextile filters is recommended for the design of granular filters, i.e. it is recommended to use a retention criterion that includes automatic truncation at the required level (not at the arbitrary level of 4.75 mm). Such a criterion for granular filters has been

developed by Giroud (2003) and is presented in Figure 16. This retention criterion for granular filters, inspired from the now classical retention criterion for geotextile filters, is a remarkable example of technology transfer from geosynthetics engineering to geotechnical engineering.

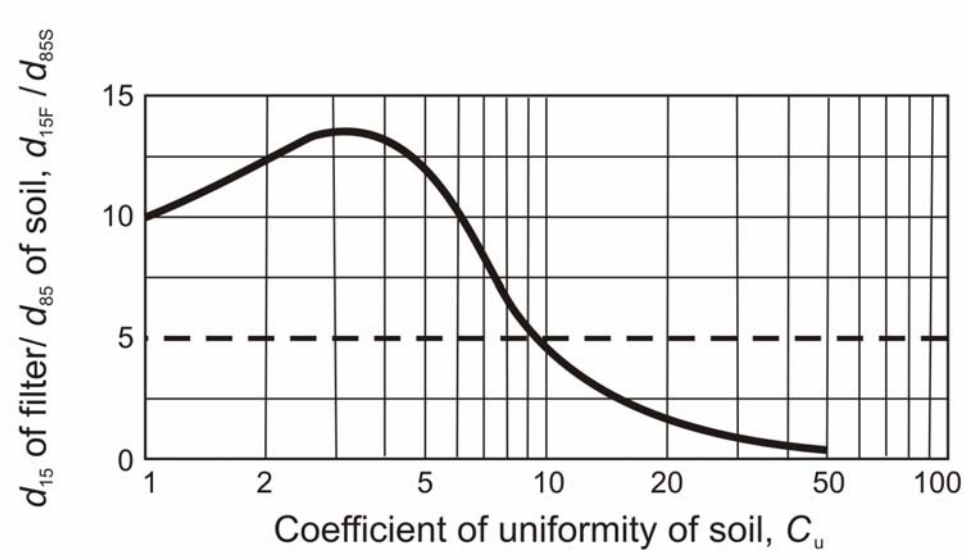

Fig. 16: Retention criterion for granular filters in the case of dense soils derived from the retention criterion for geotextile filters (the classical Terzaghi retention criterion is represented by the dashed line).

The proposed retention criterion for granular filters (Figure 16) is applicable regardless of the maximum particle size of the soil to be retained. In other words, the limitation of Terzaghi's retention criterion to 4.75 mm does not apply to the retention criterion presented in Figure 16. In other words, the need for truncation of the particle size distribution curve is eliminated when the retention criterion for granular filters presented in Figure 16 is used.

Inspection of Figure 16 leads to the same comments as Figure 15: (1) for small coefficients of uniformity, Terzaghi's retention criterion (represented by the dashed line in Figure 16) leads to selecting a filter opening size that is too small, hence a risk of clogging; and (2) for large coefficients of uniformity, Terzaghi's retention criterion leads to selecting a filter opening size that is too large, hence a risk of piping.

3.3.7. Summary and historical perspective

Terzaghi worked with granular (e.g. sand) filters. New challenges came with geotextile filters. Particularly challenging was the fact that geotextile filters are very thin compared to granular filters. Thanks to its thickness, a granular filter has many opportunities to stop a moving soil

particle. In contrast, a geotextile filter has limited opportunities, a situation that demands rigorous design criteria. Clearly, with the advent of geotextile filters, more work was needed on design criteria for filters.

The model to follow was obviously Terzaghi's criteria for granular filters. Terzaghi's criteria for granular filters are remarkable because they were developed on the basis of a rational approach at a time when geotechnical engineering was still in limbo. During Terzaghi's time, the temptation was great to use empirical criteria, especially in a case, such as filtration, that seems to defy analysis. Terzaghi's criteria for granular filters are also remarkable because they are expressed very elegantly. Terzaghi used the fact that the permeability and the opening size of a granular material are related to the particle size distribution of the material to express both the permeability criterion and the retention criterion in terms of particle sizes. Essentially, Terzaghi used a common language for two criteria that are somehow opposite: permeability and retention. However, elegance has a drawback: the smoothness of the presentation tends to hide the hard reality of the physical mechanisms, just like the body of a car hides the engine. As a result, many users tend to forget that Terzaghi's criteria correspond to two basic mechanisms: retention and permeability.

Developing criteria for geotextile filters required going back to basics, because there are no simple relationships between the structure of a nonwoven filter, on one hand, and its permeability and opening size, on the other hand (i.e. no simple way to develop an elegant presentation for the retention criterion of geotextile filters). This was a blessing because, by rethinking the mechanism of soil retention, it was possible to develop a retention criterion for geotextile filters that was more advanced than the classical retention criterion for granular filters, i.e. a retention criterion that takes into account the internal stability of the soil to be retained and, in particular, significantly reduces the risk of piping in the case of soils having a large coefficient of uniformity. Essentially, departing from Terzaghi's expression (but being consistent with Terzaghi's approach) made it possible to make progress.

The progress made was not only of academic interest. This is illustrated by the example of the Valcros Dam, an earth dam constructed in France in 1970; it is the first dam where a geotextile filter was used. As shown by Giroud (2003), if the geotextile filter used at the Valcros Dam had been designed using the geotextile filter criterion derived directly from Terzaghi's retention criterion for granular filters (i.e. opening size smaller than or equal to the

d_{85} of the soil, as expressed by Equation 6), it would have been a disaster. The soil used to construct the Valcros Dam had a large coefficient of uniformity, such as the soil used in many other earth dams, and a filter with excessively large openings would have been selected. The use of a filter thus designed in the Valcros Dam would have resulted in piping and the dam would have failed.

Finally, the advanced retention criterion, which was necessary for geotextile filters, appeared to be also applicable to granular filters. As a result, a unified retention criterion can be used for geotextile and granular filters, thereby making obsolete one of the most awkward practices in geotechnical engineering, the practice that consists of arbitrarily eliminating particles greater than 4.75 mm when using the retention criterion for granular filters. This practice is inelegant and cumbersome; it is, at best, approximate and, in some cases, it leads to errors. The work that started as technology transfer from geotechnical engineering to geosynthetics engineering ended as technology transfer from geosynthetics engineering to geotechnical engineering.

3.3.8. Lessons learned

As mentioned in Section 3.3.1, following common sense that dictates using a filter with openings smaller than the smallest soil particles is absurd, because it would lead to using quasi-impermeable filters (which, of course, would not meet the permeability criterion). In fact, the analysis presented above shows that, in some cases, the filter openings can be larger than the largest soil particles and the filter will still retain the soil. Clearly, filtration can be understood using a rational approach including theoretical analyses; it cannot be understood by common sense.

The solution of geosynthetics engineering problems is not always to adopt, or even adapt, methods traditionally used in geotechnical engineering; the solution is to use a geotechnical engineering method as a starting point and conduct a rational analysis, which leads to a better method. In turn, the method thus developed for geosynthetics engineering can be used in geotechnical engineering. Thus, the retention criterion developed for geotextile filters has been adapted to granular filters, thereby eliminating the need for an arbitrary practice of geotechnical engineering, the truncation of the particle size distribution curve of the soil for filter design.

The fact that Terzaghi was involved in the development of criteria for the design of granular filters can inspire respect and even admiration because of the elegant presentation of the criteria. However, just imitating the great masters is not the best approach to solving modern problems. The best approach consists in using all the tools available today to analyze modern problems such as those posed by the emergence of new disciplines, such as geosynthetics engineering. We do not have to do today what Terzaghi would have done 50 years ago. We need to do today what Terzaghi would do today.

4. CONCLUSIONS

Some general lessons were learned from the cases discussed in this paper. The main lesson learned from failures is perhaps the importance of rational analyses; while the main lesson learned from successes is perhaps the synergy between traditional and innovative methods and techniques.

Learning from failures requires strict intellectual discipline. Forensic analyses should be based on rational deductions conducted with Cartesian rigor. It is clear from the examples presented in this paper that common sense should not be used in forensic analyses. Common sense is a random process that can have credibility only with those who prefer a veneer of satisfaction to the depth of understanding, and who prefer the comfort of illusion to the rigor of logic. Common sense, because of its preference for traditional solutions, is particularly detrimental in the case of a novel discipline such as geosynthetics engineering. It is clear that the use of common sense must be banished from all scientific disciplines, in particular those which are under development. But, understanding this recommendation may require more than common sense.

Learning from successes is as useful as learning from failures. The lesson learned from the technique of concrete dam rehabilitation using geosynthetics is that there can be synergy between traditional and innovative construction materials: the association of concrete and geosynthetics leads to enhanced performance and durability. A similar lesson is learned from the development of criteria for geotextile filters: technology transfer can work both ways between geotechnical engineering and geosynthetics engineering.

To end on a Terzaghian note, it should be noted that the development, by the author of this paper, of filter criteria different from Terzaghi's criteria should not be regarded as a rejection of the work done by Terzaghi. First, the criteria developed by the author were inspired by Terzaghi's criteria. Second, the best way to be faithful to Terzaghi's legacy is to respect the spirit rather than the letter of this legacy. The author believes that Terzaghi himself would not have kept his filter criteria unchanged in light of the lessons learned thanks to the emergence of geotextile filters. Terzaghi's work inspired the development of geotextile filter criteria; and the development of geotextile filters would have inspired Terzaghi's work if he had lived long enough to witness the geotechnical revolution brought by geotextiles.

The last word of this paper should refer to Terzaghi, as did the first word. The author hopes that lessons learned from applications of geosynthetics will complement the many lessons geotechnical engineers have learned from Terzaghi.

5. ACKNOWLEDGMENTS

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The Vienna Terzaghi Lecture in Yokohama

Geosynthetics engineering: successes, failures and lessons learned

by

J.P. Giroud

18 September 2006

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