



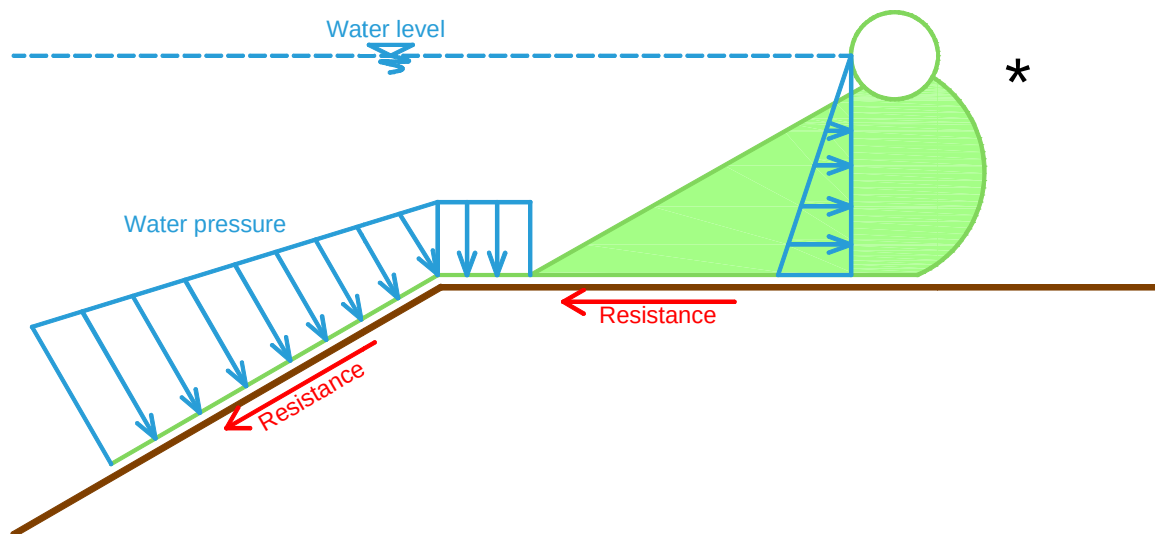
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BME Department of Geotechnics



A new way of flood protection

(English version)



Author:

Péter Szathmáry - BSc student, 4th year

Consultant:

Dr. László Nagy - BME Dept. of Geotechnics

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***Note:** The picture on the cover doesn't show the real working of the INFLATER. The details are under copyright and I don't have the permission to publish them.

1. Introduction

Nowadays it is a very crucial question to protect and how to protect our settlements from floods. In fact Hungary has 3 big and plenty of smaller rivers which can be very dangerous through the spring and autumn when most of floods are happening. For example only in 2010 the government spent more than 200 billion forints to the recovery of damages caused by floods in the spring. 600 people had to leave their houses and more than 9000 people's gas and electricity service was unavailable. After all, the old flood protection system was unsatisfying and a new solution was needed.

The INFLATER technology which is a special mobile flood protection equipment can be the solution we need to stop water from overtopping our embankments. The INFLATER itself is an easily transportable and quickly installable system which can substitute the former emergency flood defences. The point of the INFLATER is the uplifting force of water which raises an elastic wall with an air-filled balloon on top to stop water from overtopping. The entire system is made out of special polymers and is densely anchored to the embankment crest and wall. The technology was developed by the Hungarian Invention and Research Centre (MFKK) and in cooperation with the Department of Geotechnics of our university they made it to a real opportunity in emergency flood protection systems.

In the TDK I took a part of this developing and research work with laboratory and field tests to compare various materials' interaction on various soil types. The main goal of my research was to determine the scale of friction of these materials on the pre-defined soils. I examined five types of polymers with the most common embankment materials or embankment covering materials through laboratory and field tests.



Pic. 1: Another solution from the United States (Chapter 3.3.2.)

2. Hydrological capabilities of Hungary

2.1. The Carpathians

Hungary is a small country in East-Central Europe with a population of 10 million. Its area is 93 000 km² which can be divided into three main types: plains, hills and medium-height mountains, but 68% of its area is plain and 80% is lower than 200 m over sea level. The country has very good topsoil, a large system of rivers and significant amount of underground water resources.

All these could be great if we wouldn't take notice of the surrounding relief. Hungary is like a great catchment basin surrounded by the Carpathians and partly by the Eastern Alps. The country lies mostly on the Carpathian basin and 95% of its water supply comes from those mountains. The Carpathian Mountains are the greatest and second longest mountain chain in East-Central Europe stretching an arc from the Czech Republic through Slovakia, Poland, Ukraine, etc. to Romania with a total length of about 1500 km. The mountain ranges have a climate very similar to the Alps and with a high rate of precipitation: annually 600-1600 mm (in Hungary it's 600 mm), but on the southern slopes it is 100-200 mm higher at the same altitude. In the Carpathians the two-third part of precipitation comes in summer as rainfall and snow and the colder seasons are usually drier.



Pic. 2: A typical winter valley in the Carpathians: all the snow is ready to melt

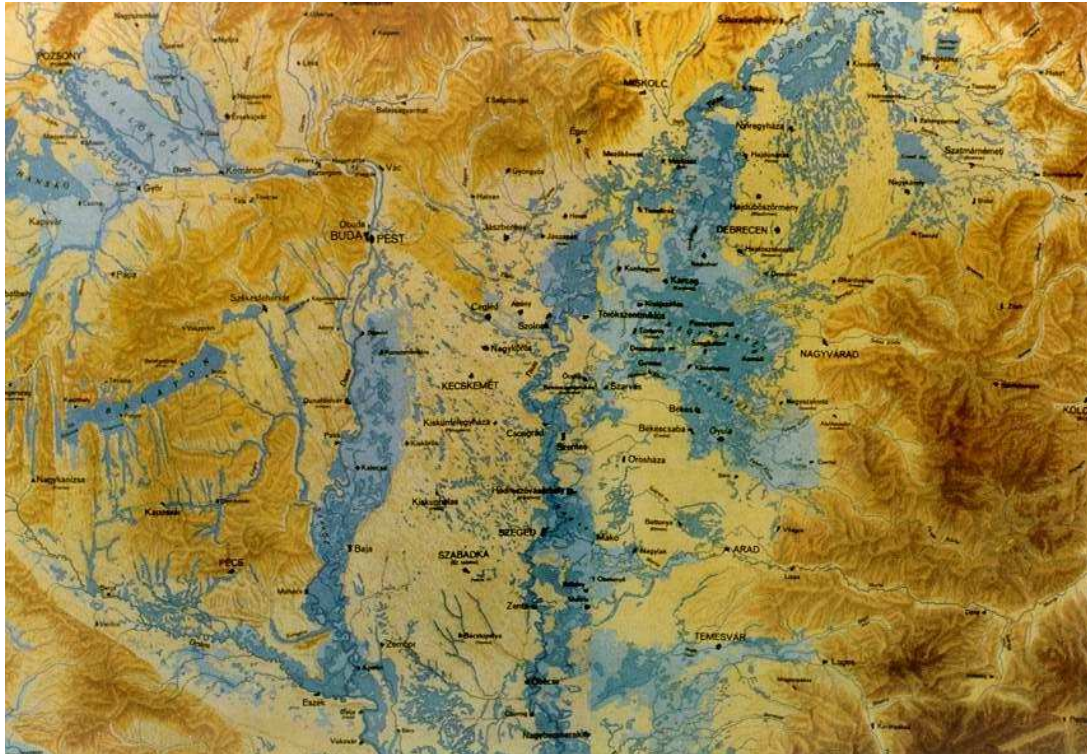
The Carpathians are rich in rivers. The greatest ones are the Dnister, the Danube, the Tisa (which is also a tributary of the Danube), the Prut and the Seret Rivers. These have hundreds of smaller and bigger tributaries and lakes making the whole mountain range a complex network of catchment areas. The rivers are fed mostly by snow and rainfall that means the highest runoff is probable in summer from April to August. Unlike the Austrian Alps the Carpathian Mountains haven't got so many barrages so its rivers are much more likely to flood at those times.

2.2. Our rivers

In Hungary there is a 4200 km long flood protection system which provides safety to more than 600 villages and cities. The country is rich of high runoff rivers such as the Danube, the Tisa or the Drava. Out of these three only the last one has its rise in the Alps.

<i>River name</i>	<i>Average runoff [m³/s]</i>	<i>Length in Hungary [km]</i>
<i>Danube</i>	6500 (by Budapest 2350)	417
<i>Tisa</i>	820	585
<i>Drava</i>	670	357
<i>Mura</i>	166	45
...	...	...

Table 1: Runoff and length of our rivers



Pic. 3: Floodplains in Hungary before the immense river control works

2.2.1. The Drava

The Drava has a very wide floodplain with lots of swamps and that is the cause why there weren't so many floods in the history. The last serious flood was in 15th of July 1972 when the water level reached 618 cm which has never seen before. Thanks to the fast reaction of the office dozens of organizations helped in reinforcing the available embankments and with hard work they succeeded to stop the flash flood. Nobody was hurt and the damages caused by the flood were minimal. After that the government decided to give an allowance to the repairs and improvements of the flood protection system. The construction has not been finished yet, but it seems to be a satisfying solution so far.



Pic. 4: The Drava in 1972

2.2.2. The Tisa



Pic. 5: Szeged under water in 1879

Hungary's most endangered settlements are all next to the Tisa. This river flows in a quite plain part of the country with giant floodplains and unfortunately there are lots of important cities near that. Thanks to cutting out of woods on catchment areas the flood runoff could be even bigger. Of all our big rivers the Tisa spills out the most often and we have to handle this danger to provide safety for people.

In the course of our history we lived through always bigger and bigger floods of the Tisa. For example the best-known incident is the flood in Szeged, 1879. The lack of maintenance of the embankments in the still times had very serious effects when it had to work properly and prevent water from overtopping. On 12th March the river broke through the levee and suffused the city of Szeged and its surroundings. The city has been almost totally destroyed and 151 people died in the flood. Nearly 6000 of Szeged's 6350 houses were made even with the ground and more than 70 000 citizens became homeless. Luckily at that time the world's nations were not worried about wars or epidemics and could send intense support to Hungary to rebuild the city and help "the 70 000 homeless".



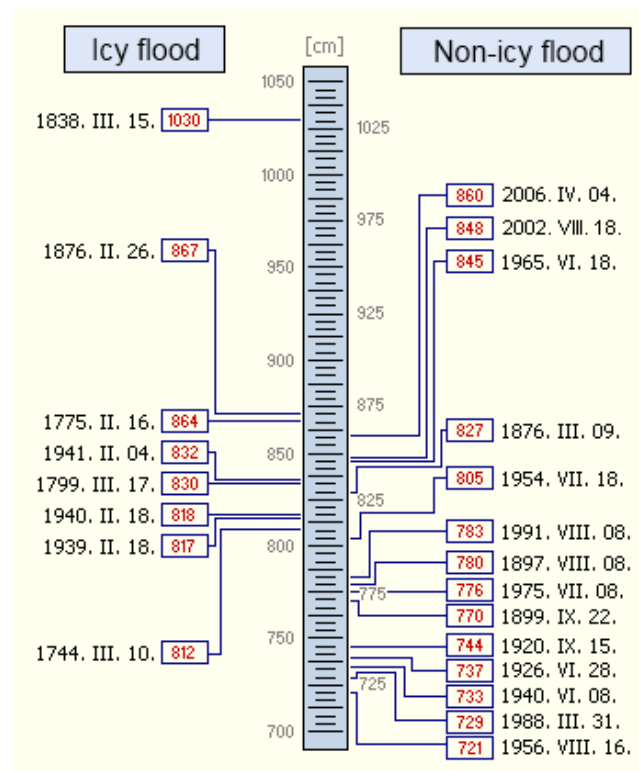
Pic. 6: Photo of a suffused neighbourhood near the Tisa

After that the flood protection of the river was drastically improved on the basis of the Vásárhelyi Plan. In the next decades there were dozens of high waters, but neither of them was as dangerous as the flood on 22nd March, 1999. The Tisa's water height reached and overstepped the hundred year old record, but thanks to the hard work the river didn't overtop the dyke. Two years later in 2001 the same thing happened: record water height and a near catastrophic situation alerted the government to make more effort to improve the flood protection system even more.

2.2.3. The Danube

Hungary's biggest or "main" river flows across the whole country from the Austrian border via Budapest to the southern borders. After the Volga it is the second longest river in Europe and it has important international water transport routes from Germany to the Black Sea. That's why the Danube's flood protection system is well-maintained and since the 19th century there weren't any serious floods.

In March, 1838 the melting of the upper Danube started way before the Hungarian stage. The water found its way under the ice uplifting it, breaking it apart and carrying the pieces with itself. The ice pieces then impacted to an obstacle (e.g. an island) making an ice barrage and raising the water's height behind it. This process is the birth of an even more dangerous type of flood: the icy flood. At midnight on 15th March the same thing happened near Budapest raising the water to a record height of 1030 cm. 40-90% of the houses were ruined or damaged in 24 settlements next to the river like Szentendre, Vác, Szigetmonostor etc. 153 people lost his life that night. Never before and after that was the water level so high. Even the second highest stand in 2006 was 1.5 m lower and not icy.



Pic. 7: Water levels on the Danube in Budapest



Pic. 8: The relief on the Ferences Church's wall shows Miklós Wesselényi's heroic fight of saving people on 15th March

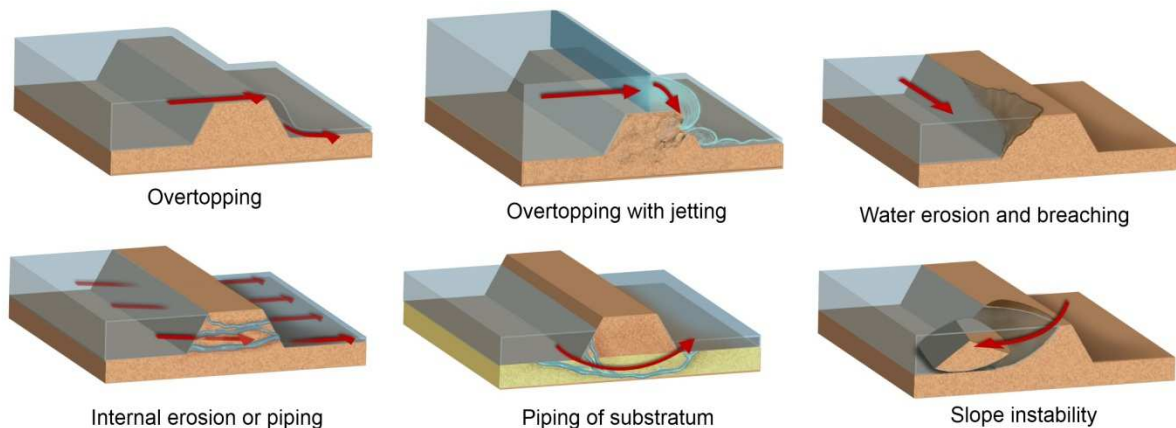
3. Flood protection

3.1. Most common causes of levee failures

Hungary is rich in water in every aspect, but as we saw in the historical examples the water can also be very dangerous if we can't lead it away from our settlements and worth. There are worldwide statistics about the embankment failures to help engineers improve the flood protection solutions. First of all we have to separate two various terms: the failure's cause and the failure's mechanism.

The failure's cause is the background process that leads to a mechanism. Most of the cases it can't be foreseen, but with an advanced monitoring system or frequent supervision we can increase our chances to react in time. These causes can be settling, internal erosion, bad compression, bad construction, earthquake, etc.

The failure's mechanism is the failure itself. We can definitely see it and most of the times the process is already irreversible or needs too much effort to handle. The most common ways of failure are listed in *Picture 9*.



Pic. 9: The most common failure mechanisms in Hungary

In the 20th century engineers started to collect information about dam failures to get acquainted with these mechanisms. In the US the government wanted to know more about the dangers of different dam types and urged them to investigate earlier dam failures – mostly large dams. The data engineers collected consists all of the failure modes listed above so we may also use them as a good starting point in examining the levees.

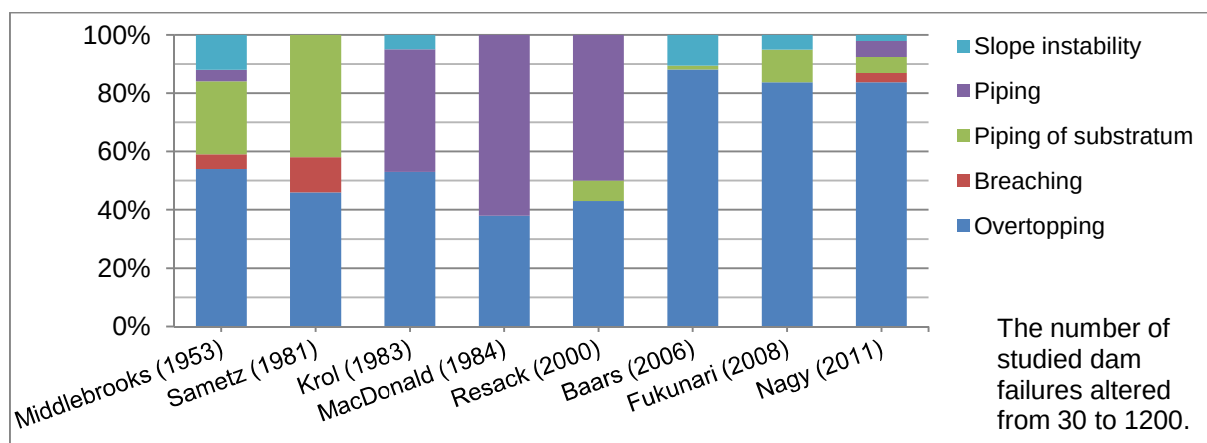


Fig. 1: Levee failure frequency by mechanism, consisting large dam statistics too

3.1.1. Slope instability

The most common failure mechanism by earthworks is the slope's stability loss. When the balance of the slope's internal forces doesn't come off, the loads increase or the resistance decreases the slope begins to slide down slowly under its own weight. We call the sheared surface between the moving and the still soil bodies the sliding surface. There are lots of calculation methods for this failure because the process is well known for a long time. Among others Bishop, Spencer, Janbu, Morgenstern-Price and Fellenius-Petterson created simplified iterative calculation methods for this problem. The spreading of finite element programs made the analysis much easier and faster and probably more precise.

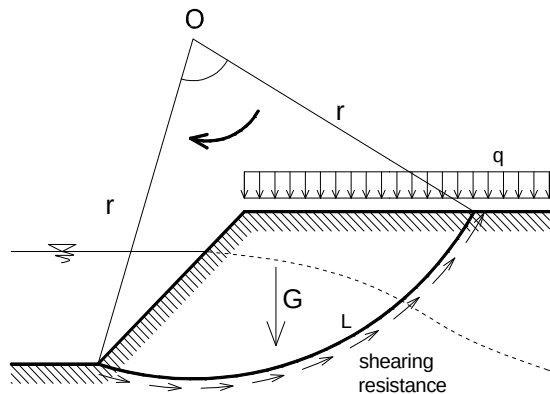


Fig. 2: A typical cross-section



Pic. 10: A giant landslide at Yallourn, Australia

The landslide can be prevented with proper monitoring and quick intervention. Lowering the embankment's loads or raising the sliding surface's shear resistance by charging on the lower part of the sliding body could be the easiest way to prevent the failure.

3.1.2. Piping

Another dangerous form of levee failure is the piping of the embankment or substratum. It is caused by too high water pressure in the embankment body which literally tears the soil grains apart letting through the water. It begins with a slight washout and leaking, but without attention piping can easily destroy the whole levee. Normally the water can't penetrate the embankment, but by high pressure it finds a leakage route short enough to go through while the flood goes on.

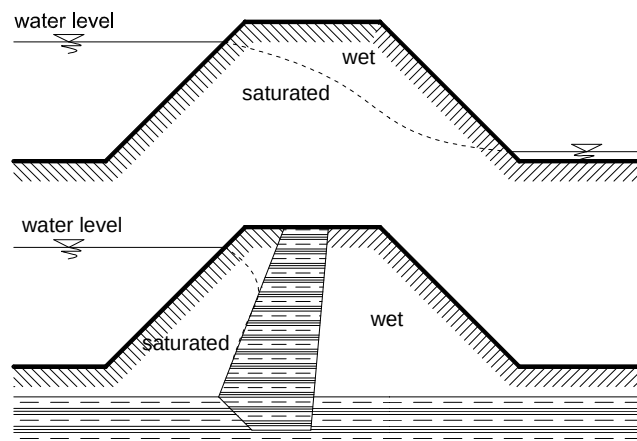


Fig. 3: How the impervious core works



Pic. 11: Piping near the Tisa

It can also happen under the embankment. Its only sign we can notice is that water appears in the protected side of the levee, but without jetting. We can prevent this happening by building a sealing into the embankment so long that it reaches the impervious subsoil under the levee. So the length which water has to flow to the protected side increases drastically just as the time it takes. This sealing can be pre-made of steel, polymers, etc. or built with the embankment as an impervious soil core.

3.1.3. Piping of substratum or sand boil

This failure is some way similar to the piping, but in this case water is nipped in a pervious layer between two impervious layers of the soil under the embankment. The water pressure in there rises with the water level and after it reaches the limit of the upper soil layer's strength, it breaches that. This can happen next to the levee or even 10-20 meters away – it depends on where the upper soil layer is thin enough. We can't prevent or reverse this process because blocking the spillage of water creates a new breach somewhere else. The traditional method of protection is to simply build a small dyke around it and let it go up until it can.



Pic. 12: Jetting near Tiszasas

The most drastic mode of embankment failure is breaching. The water saturates the embankment body and if it isn't compressed or constructed well, water starts to erode it, reducing its resistance against water pressure. It doesn't take much time for the embankment to lose its stability and in a sudden moment to breach. A huge amount of water flows through the hole broadening it even more and flooding the protected side of the levee. The only sign before going broke is the erosion of

3.1.4. Breaching



Pic. 13: A huge breaching near Tarpa in 2001

embankment wall. It's very hard to prevent breaching, but with divers laying some kind of polymer blanket on the bottom of the watercourse and embankment we could reduce the chances of erosion and breaching.

3.1.5. Overtopping

The most common failure is overtopping when water level rises above the height of the levee and water spills over it. But the main problem isn't just the fact that the levee can't serve its original function any more. The water saturates the whole embankment carrying dangers similar to the breaching and so damages can be similar too. Before reinforcing our levees overtopping was the most probable form of failure, scaring people for centuries. With the development of the flood protection systems the chances of overtopping reduced, but it's still the most feared failure mechanism.



Pic. 14: Consequences of overtopping on the Vistula

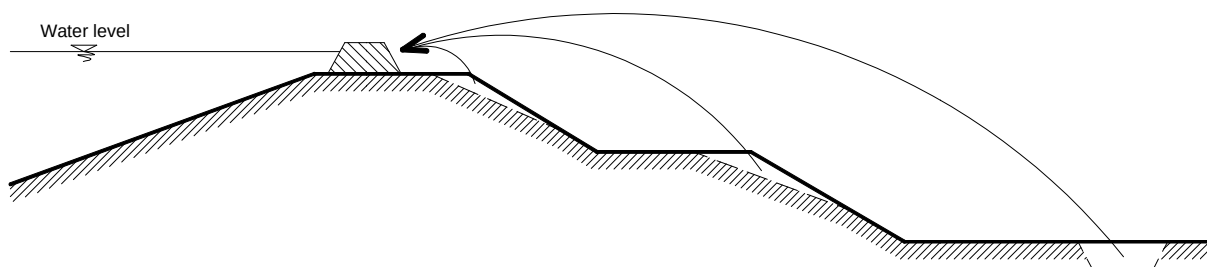
3.2. Commonly used methods of protection against overtopping

Of course the levees today can also be unsatisfying because sometimes they aren't high enough to prevent overtopping. So the engineers started thinking about how to expand the dykes, how to make them taller, but not weaker whilst the water level rises?

The problem is a bit more complicated than it looks. Somebody could say why don't we just build levees 1 or 2 meters higher than the highest water level ever? The answer isn't simple, but in short we could say: because that would make our country bankrupt, drastically change the terrain and the natural habitat of animals and plants around the rivers and also change the view of the region. On the other hand there are places which can't be protected even that way: cities and suburban regions. Therefore emergency solutions are needed and we just can't bypass them. Let's take a look at what we've got today.

3.2.1. Heightening with local material

If we have sufficient machines, material and time, the easiest way to heighten the dyke crest is "capping". While capping, an excavator grabs soil from pre-defined points of the levee and then a dozer or grader works that in the dyke crest. The whole thing is built on a layer of waterproof plastic sheet to prevent wave erosion. The material need of this method is horribly high, so in practice we use this mostly when only a short section of the levee needs extra height. The monitoring and traffic can continue on the new dyke crest, but after the flood the engineers used to remove it because its long-time unreliability.



*Fig. 4: Placing of barrow pits, where from we get the building material.
The closer is more dangerous*

3.2.2. Heightening with sandbags

The traditional emergency protection method is to heighten the levee with sandbags. These can be pre-filled with sand in a reserve deposit or near the levee if proper soil is available (according to *Figure 4*). People who are working on the dyke just have to place them correctly which makes this method quite quick. Sandbags are also used to stop the sand-boil's water spilling everywhere, particularly into the embankment body on the protected side which reduces its stability.

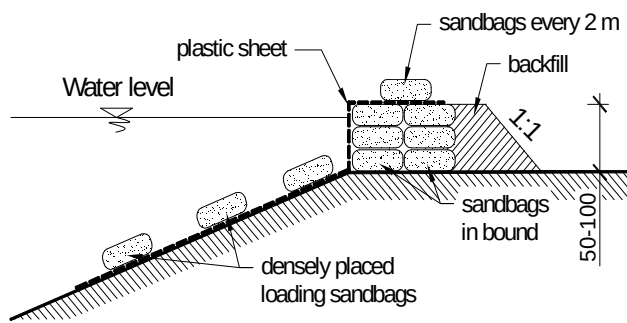


Fig. 5: The schematic drawing of the placement



Pic. 15: Heightening with rip-rap-reinforced sandbag wall at Szentendre before the flood - 2010

The most important part of this method is the correct placement of the sandbags. There are special arrangements according to the circumstances, like curved section, or narrow dyke crest, etc. The general idea is that sandbags should have placed in a single row if the height is no more than two of them and use more rows if higher. The total height of the solution can go up to 1.5 meter (*Tiszadob, 2000*) which requires proper design and accurate construction with six rows of sandbags. We should use backfill if the water level is high for a long time.

3.2.3. Waterside boarding

Waterside boarding is a bit more labour-demanding method because of the construction's complexity, but also a very fast way of flood protection. It stands of a plank wall on the dyke crest supported by two pales from the sides and a few sandbags or earthfill from the protected side. The solution needs much fewer sandbags than the previous method, but because of wooden boards it is suggested to have a transportation route on the dyke.

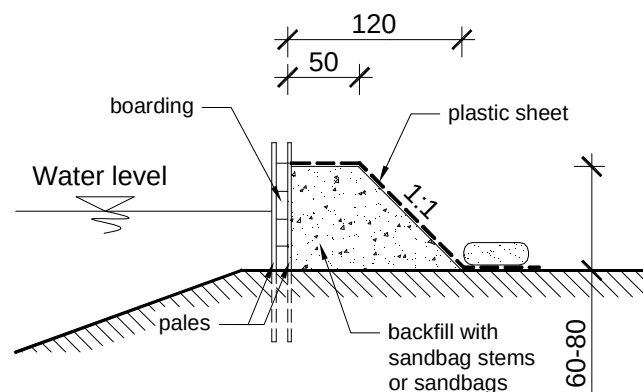


Fig. 6: Cross-section of the waterside boarding



Pic. 16: Waterside boarding under construction at Tiszasüly, 2000

The method itself can support protection up to the height of four boards which is about 80 cm and the reinforcement on the back can be upgraded if necessary. Using earthfill instead of sandbags reduces the resistance against waving, but in that way we can build the boarding without the need of sandbags.

3.2.4. Cofferdam

The most complex emergency solution is a cofferdam. Its construction needs the most labour and the most time to build, the most material and so it is used only in reinforcing short sections. But it also has its benefits: a cofferdam can be built more than 1 meter high and it is a very reliable method. It is quite similar to the waterside boarding, but this time the earthfill is laid between two rows of boards and there's no other reinforcement. To prevent the pales from opening because of the inner material's pressure, the whole structure is fastened together with iron bracing on one or two levels. The cofferdam has to be connected with the dyke imperviously, but not permanently and it has to be stable enough to withstand the water pressure.

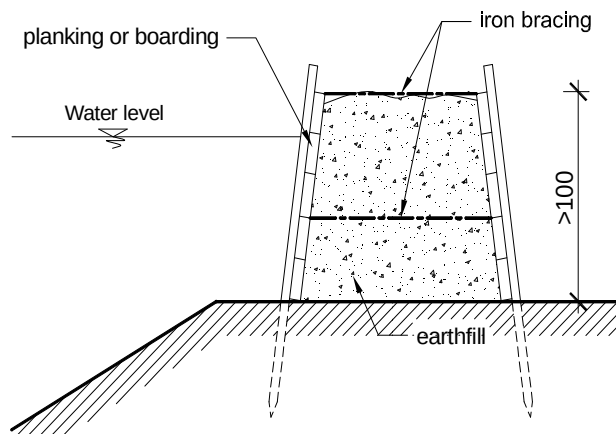


Fig. 7: Typical cross-section of a cofferdam



Pic. 17: Cofferdams were frequent in the '70s

3.3. New methods against overtopping

The examples above were used for centuries against overtopping, but from the '80s other methods using pre-fabricated modules were introduced. These are offering very fast instalment and a wide range of places where we can use them. We can divide them into two groups according to their needs and benefits:

- The mobile flood protection walls are vertical walls made of metals e.g.: steel or aluminium. These walls need proper foundation on the site where they'll be set up, but on the other hand they can provide a height of 3.5 m.
- The mobile flood protection dykes are typically made of some kind of polymers and are mostly inflatable. We can easily lay them on the surface, there's no need of a foundation, but maybe some anchors depending on the technology. The maximum height of these dykes is about 1.5 m.

3.3.1. Mobile flood protection walls

The mobile flood protection walls are mostly used in those areas where people don't want a permanent dyke during the low-water months of the year. This kind of flood protection isn't really giving an emergency solution against overtopping because it needs a proper foundation. This is typically some kind of steel curtain which also has a role in the prevention of leakage. The wall itself is only installed when the water level reaches its limit value. The flood protection wall can be 3.5 m high and several kilometres long. The separate wall modules can be easily assembled and with the pre-made foundation and struts it is a very strong structure.



Pic. 18: The graphic shows the length ratio between the wall and the foundation



Pic. 19: The chancellor is satisfied with the system



Pic. 20: The modules are ready to assembly



Pic. 21: Installing this kind of wall

The main disadvantages have their origin in the technology itself. First of all its price is very high because of the materials and the fact that it functions only in one or two months per year. Furthermore during the low-water months we need to store the modules somewhere which also has its price and the module's maintenance is not guaranteed. All in all it is the most expensive way of "emergency" flood protection, but sometimes the only that is suitable.

This method of flood protection is widely used in the European cities for example in Paris, Prague or Cologne.

3.3.2. Tube dykes

Tube dykes are the first type of mobile dykes invented in the '70s. These are long polymer tubes which have been filled with water. There are various forms of tube dykes according to the function and the manufacturer, but by mechanism they're all the same. Tube dykes are the opposite of the flood protection walls. These are cheap and easy to install on nearly any surface in no time, but their drawbacks are also serious: lower height (60-100 cm), hydraulic problems, the danger of vandalism and mostly unreliable sealing because of uneven surface. Some of the problems can be solved partially or totally, but it doesn't change the fact that the tube dykes are risky to use.



Pic. 22: The nominal diameter (here 1.5 m) is always bigger than the real because of the Poisson-effect



Pic. 23: A 3 tube high system under filling



Pic. 24: Well-placed tube dykes along a country road in the US

Tube dykes can show up in various forms: one tube, a system of 3 or more tubes, opened tube, triangular or even oval. The performance isn't really dependent upon the shape.

3.3.3. Triangular dykes

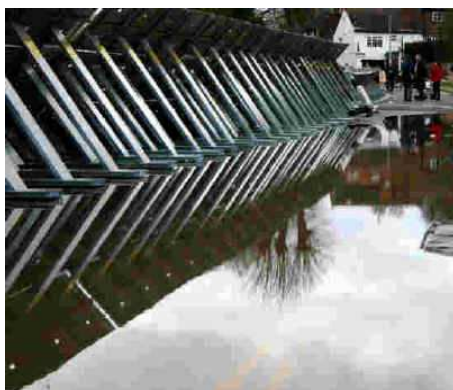
These dykes are made of triangular holders and covered with waterproof sheet to fulfil its hydraulic requirements. The holders can be metal plates on some kind of triangular struts, but can also be constructed with pallets which are always available in large numbers. The covering sheet ensures the imperviousness, but it has to be fixed with sandbags. The method doesn't need any preparations and can be useful up to 80 cm high.



Pic. 25: As the picture shows the solution can also withstand a flood of Hummers



Pic. 26: The triangular dykes are very flexible in point of placement



Pic. 27: Sometimes the leakage is too much



Pic. 28: The cheapest flood protection

The problem with this technology is the connection between the holders and the surface. In opposition with the tube dykes, the triangular dykes can be poorly “connected” to the flat surfaces like concrete or asphalt. The covering sheet with the sandbags is not always perfectly waterproof so the leakage is quite common on the protected side. Triangular dykes are somewhere between the flood protection walls and the tube dykes. This solution sums up the benefits of the two with some compromises.

3.3.4. Other solutions

Of course there are lots of other emergency flood protection methods, but we won't review them all because most of them aren't ready for real use. The thing we can do is to rehearse them in order to show what other solutions are in development today.

- Folding plates are somehow similar to the flood protection walls, but here two plates are connected with a hinge in 90° (Pic.29.). The main problem with this is the leakage under the bottom plate which can't be prevented without a foundation.



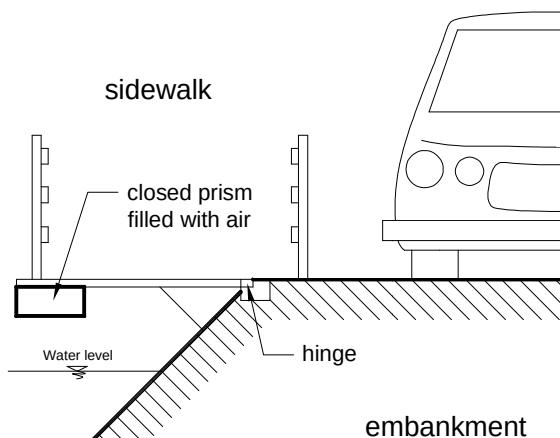
Pic. 29: Folding plates in their extended form

- The butterfly dykes are the ones which people can use for their own safety at home – at least theoretical. Its material can be a soft sheet-like polymer or a solid plastic L-form, but neither of them is suitable because of the leakage under them.



Pic. 30: Butterfly dyke can't prevent leakage even on a demonstration...

- Wire baskets are somewhat like the gabion slope supporting walls, but this one is made of watertight polymer. The real problem with them is the connection of baskets and the difficult construction.
- As usual, the Japanese couldn't withstand not to make a strange solution. It's a sidewalk tightened with hinges on any levees or next to roads. The sidewalk has air-filled prisms under its edge which raises the whole plate with the water's uplifting force.



Pic. 31: The sidewalk under normal use



Pic. 32: The water raising the sidewalk and it becomes a flood protection wall

4. The INFLATER technology

4.1. Introduction

As we could see above, there are lots of emergency solutions against the most feared levee failure, the overtopping. But all of them have serious drawbacks and the lack of monitoring which would be very important in a flood scenario. Therefore the MFKK (Hungarian Invention and Research Centre) designed a whole new method of flood protection which's name is INFLATER.

The INFLATER is somewhat similar to the tube dykes because it is made of some kind of polymers and also similar to the Japanese invention above because the INFLATER uses the water's uplifting force to withstand the water pressure. It is a mobile flood protection system which can be transported easily and set up quickly: it can be placed on an existing dyke crest with a watertight skirt. The INFLATER is designed to substitute the former solutions of overtopping protection such as the most common sandbags or the most expensive, but reliable mobile flood protection walls. It is capable of having electrical sensors measuring the water height and the state of the INFLATER to prevent overtopping and to help predicting the river's behaviour.

In the design three of our university's departments took part: the Department of Geotechnics, the Department of Structural Mechanics and the Department of Hydraulic and Water Resources Engineering, but the first one was responsible of creating the system specifications.

The Department of Structural Mechanics took part in the structural analysis which was necessary to define the requirements which the materials have to meet. After that the Department of Geotechnics did dozens of tests according to the probable failures. There are three significant types of failure: uplifting, overtopping and sliding. My part of testing was to determine the sliding resistance (friction and adhesion) between the materials and several types of underground. With the results of these tests we wanted to make an order among the materials to see which one is the most enticing. The Department of Hydraulic and Water Resources Engineering then picked this (or these) materials and performed large sample tests in a 3x1 m pool.

**Disclaimer:**

*Neither these tests, test results nor the technology itself are public. I don't have the permission to write about them more detailed than above. The only results I can show are my own test results which have been performed to define the shear resistance between the chosen materials and the most common types of embankment soils. **These are also under copyright and are not allowed to copy or share.***

5. Examining the soil-INFLATER interaction

As we mentioned before the INFLATER uses the uplifting force of water to stabilize itself against the water pressure. There are several other forces too which are acting at a moment, but we'll discuss only the resistance on the soil-INFLATER interface.

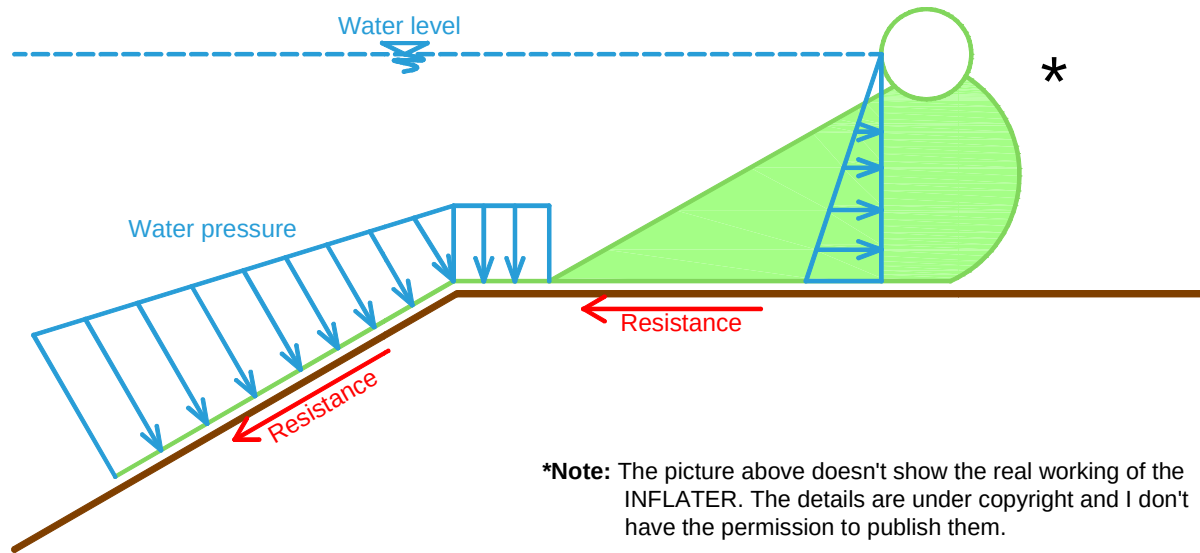


Fig. 8: The acting forces on the INFLATER at high water (sketch)

The INFLATER is entirely made of flexible, rubbery polymers. Due to this, the most probable failure mechanism is sliding. As you can see on *Figure 8* the only stabilizing force that is acting against a sideways sliding movement is the resistance on the skirt and the bottom of the INFLATER. This resistance stands of adhesion and friction which can be expressed with the relationship below:

$$\tau = \sigma \cdot \tan \delta + a$$

Where

τ is the developed shear stress (resistance) at the inflater-soil interface

σ is the normal stress acting on the interface (between the adjacent materials)

$\tan \delta$ is the friction coefficient at the interface (δ is the friction angle)

a is the adhesion between the two surface

So among these components of resistance only the adhesion is independent on the stress state. The scale of friction depends on the normal stress acting on the interface and therefore the whole resistance value is stress-dependent too. That means that the height of water linearly influences the mobilized resistance with the normal stress and the pore pressure of the underground. In order to determine the shear resistance we have to know the acting normal stress (which can be calculated very easily) and the two interface parameters: $\tan \delta$ and a . These two can be determined with laboratory tests discussed below.

5.1. Used materials

We have chosen five types of individual materials for these tests. All of them had individual properties of thickness, flexibility, roughness and manufacturing technology.

<i>Name (colour)</i>	<i>Product ID</i>	
Green	SIOEN	B7119
Light blue	SIOEN	S2501/5998
Beige, rough side	SIOEN	B9550/1617
Beige, smooth (back)side		
Dark blue	SIOEN	G7655/5287
Dark blue backside (nylon like)		

Table 2: The materials by colour and ID

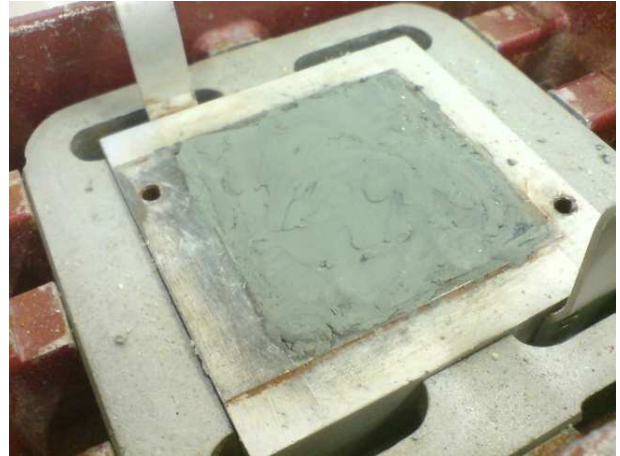
It must be noted that the interface behaviour of the system can't be simplified to a property of a material. We have to study the interaction of the skirt and the soil to compare the behaviour of these materials which is the aim of our tests.



Pic. 33: Dozens of samples were used in the laboratory tests

5.2. Laboratory testing method

The preliminary studies have shown that the most common building materials or embankment covering materials at flood protection sites are asphalt, concrete, clay, sand and grass (Pictures 34-37).

*Pic. 34: Asphalt sample**Pic. 35: Plastic grey clay sample**Pic. 36: Grass in clay bedding**Pic. 37: Wet sand*

In the laboratory tests we used a modified shearing test with a direct shear apparatus, so we could obtain the required interface parameters. The main difference between this and the normal shearing test is that we replaced the lower frame's soil with our embankment material and we attached the INFLATER material to the upper frame. During the test a normal stress was applied to the soil through a load plate while the lower frame was moved horizontally at a constant speed ($v=1.0 \text{ mm/s}$). A computer recorded and visualised the displacement and the required force to move the frame – this is the shear force. We repeated the test several times using every combination of the materials above and all of them with three different normal stresses on the frame. *Figure 9* shows the basic principle of the test.

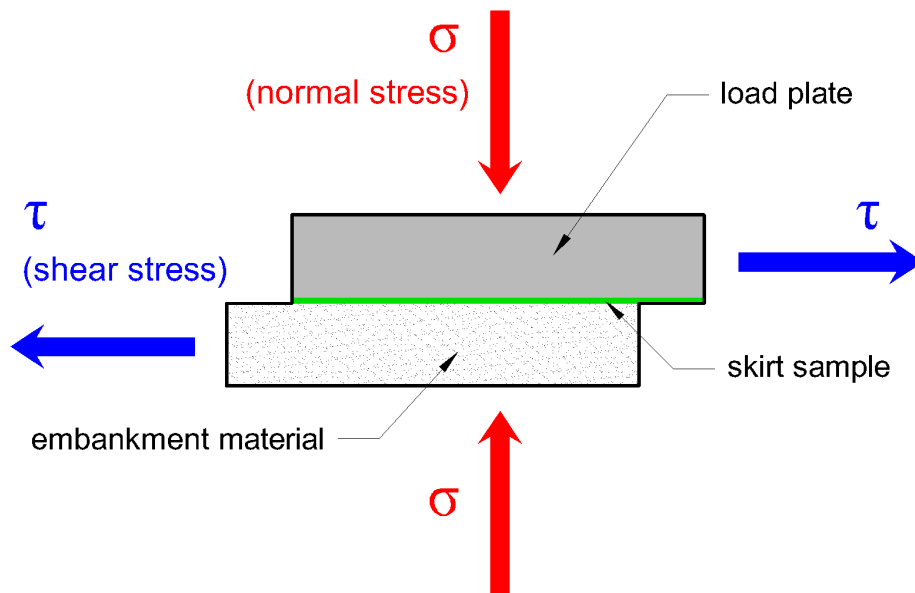
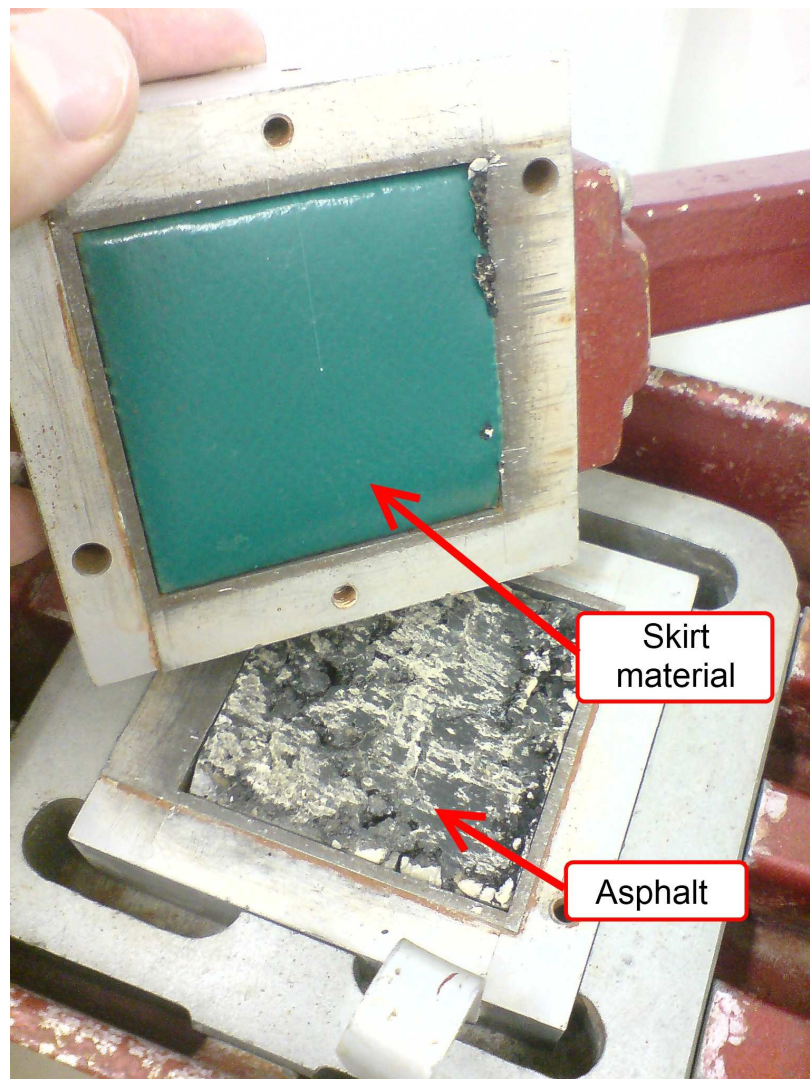


Fig. 9: The schematic sketch of the shearing test



Pic. 38: The placement of the skirt and the ground material

With the data of displacements, shear stresses (τ) and the known normal stresses (σ) we plotted the results in two various forms:

- During the test the computer plotted the shear force against the displacement. The shear force can be easily converted to shear stress by dividing with the contacting surface which is 60x60 mm. After that the plot illustrates the mobilization of shear stresses (*Figure 10*).
- Plotting the shear stress against the applied normal stresses is used to obtain the two parameters which we were looking for: the friction coefficient ($\tan\delta$) and the adhesion (a) (*Figure 11*).

The normal stress values were chosen according to the expected real normal stresses: 8.2 kPa, 15 kPa and 30 kPa. These are the borders of the expected strains since 30 kPa is equal to the hydrostatic pressure of 3.0 meter deep water.

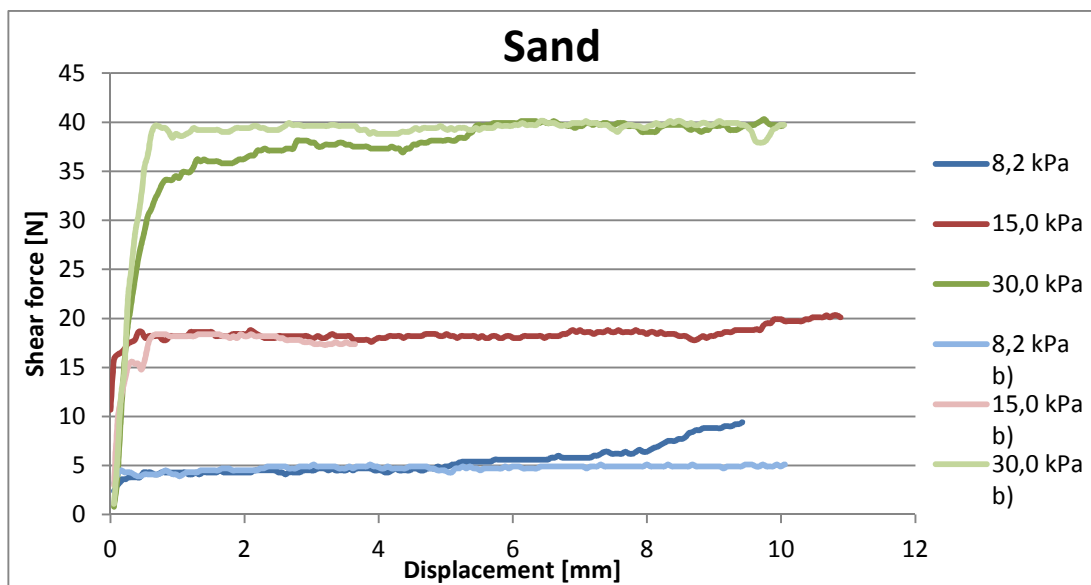


Fig. 10: Raw plot of mobilizing shear forces in the beige sample (lighter line means "backside")

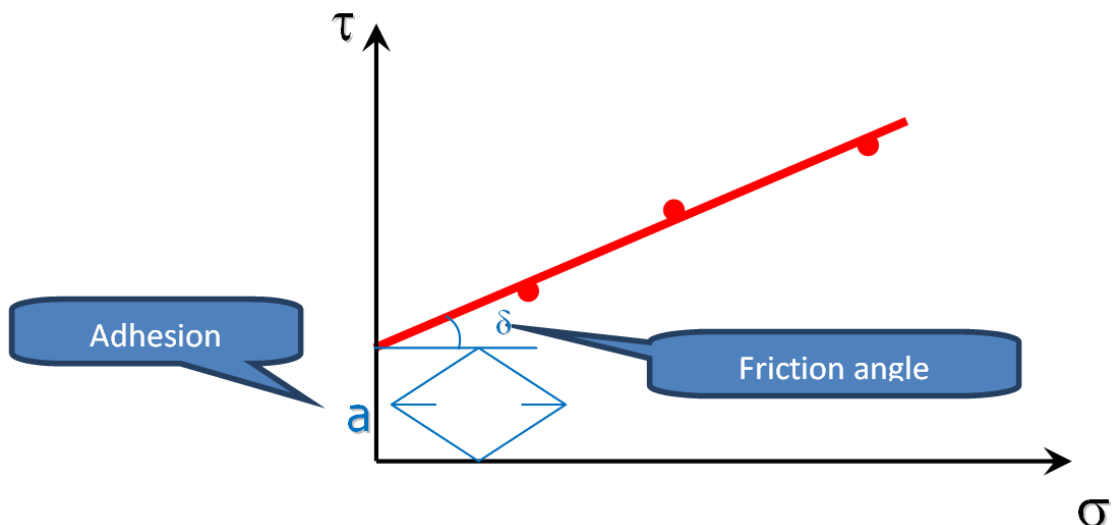


Fig. 11: Shear resistance against normal stress after three tests with the same sample

5.3. Laboratory test results

As we mentioned earlier the five embankment materials or embankment covering materials were chosen because they're the most common in Hungary. Of course on the site these can vary over a very wide range making it impossible to exactly predict the behaviour of the in-situ interface. The varying properties can be the soil type, density, consistency, the surface's roughness or by asphalt and concrete the condition, type, roughness, etc. It is clear that the number of variations is endless, but we have no time to do endless number of tests. So that our goal was to obtain information about the most common INFLATER material's behaviour and not to create a database, or provide general design parameters.

5.3.1. Asphalt

Asphalt is likely the most common in-situ material in urban environment, but is also likely to be found on the dyke crests. Therefore we can count on that the INFLATER will be frequently placed on asphalt surface. For the laboratory tests we used an asphalt core sample which was cut off from an existing road and its size is 60x60 mm. In *Figure 12* you can see the mobilization of the shear stress while testing the green INFLATER material with various normal stress levels on asphalt. After that, we plotted the maximal shear stress values against the normal stress levels in the σ - τ coordinate system. Then we fitted a straight line on these three points (per material) which assumes a linear relationship between the shear resistance and the normal stress. The summarized results and the interface parameters of the INFLATER materials are shown in *Figure 13*.

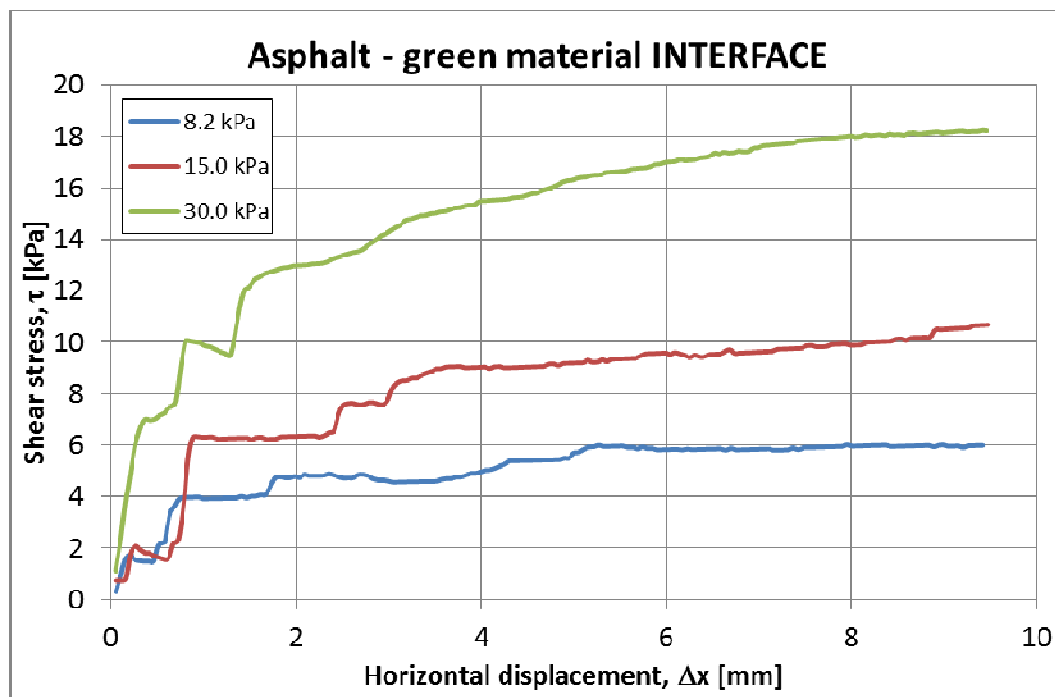


Fig. 12: Shear stress against horizontal displacement

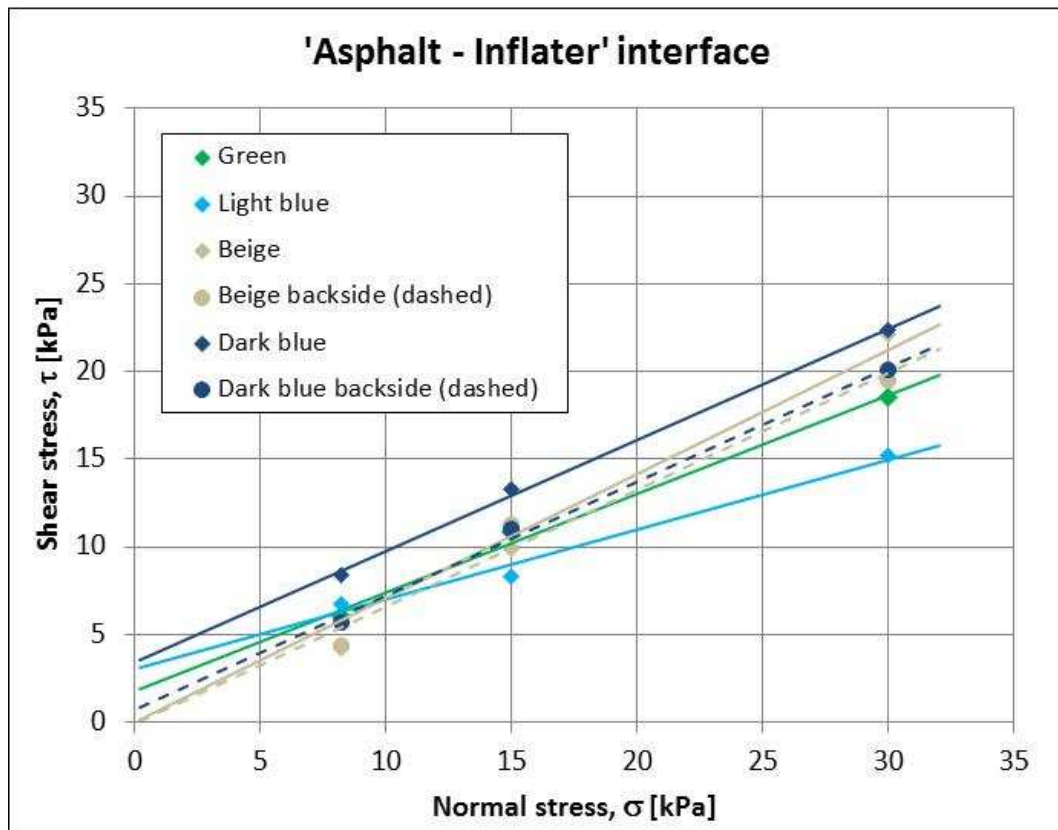


Fig. 13: Shear parameters of asphalt-INFLATER interfaces

INFLATER material	Friction coefficient, $\tan \delta [-]$	Friction angle, $\delta [^\circ]$	Adhesion, a [kPa]
Green	0.56	29.5	1.8
Light blue	0.40	21.8	3.0
Beige	0.71	35.4	0.0
Beige backside	0.67	33.8	0.0
Dark blue	0.63	32.4	3.4
Dark blue backside	0.63	32.4	3.4

Table 3: The resulted parameters of asphalt-INFLATER interfaces

The results show very similar trend for all INFLATER materials. The adhesion values are very small, the maximum value is 3.4 kPa, but for the beige material this value is zero. Thus the shear resistance of the interfaces is primarily provided by friction.

The measured friction angle varied between 21.8 and 35.4 degrees, but only the light blue material had lower friction angle, all the other values were within the 29-35° range. On the other hand the lower friction angle of the light blue material is compensated by somewhat larger adhesion, so the total shear resistance of all interfaces is quite similar at the typical 10-15 kPa normal stress level.

5.3.2. Granular materials: sand

Another potential group of embankment materials are the granular soils. By our laboratory test in a 60x60 mm wide shear box the maximal allowable grain size is about 4 mm which is sand. The grain size distribution of the used sand material is shown in Figure 14. Based on this curve the soil can be classified as uniform sand according to the MSZ EN 14688-1 and MSZ EN 14688-2 standards.

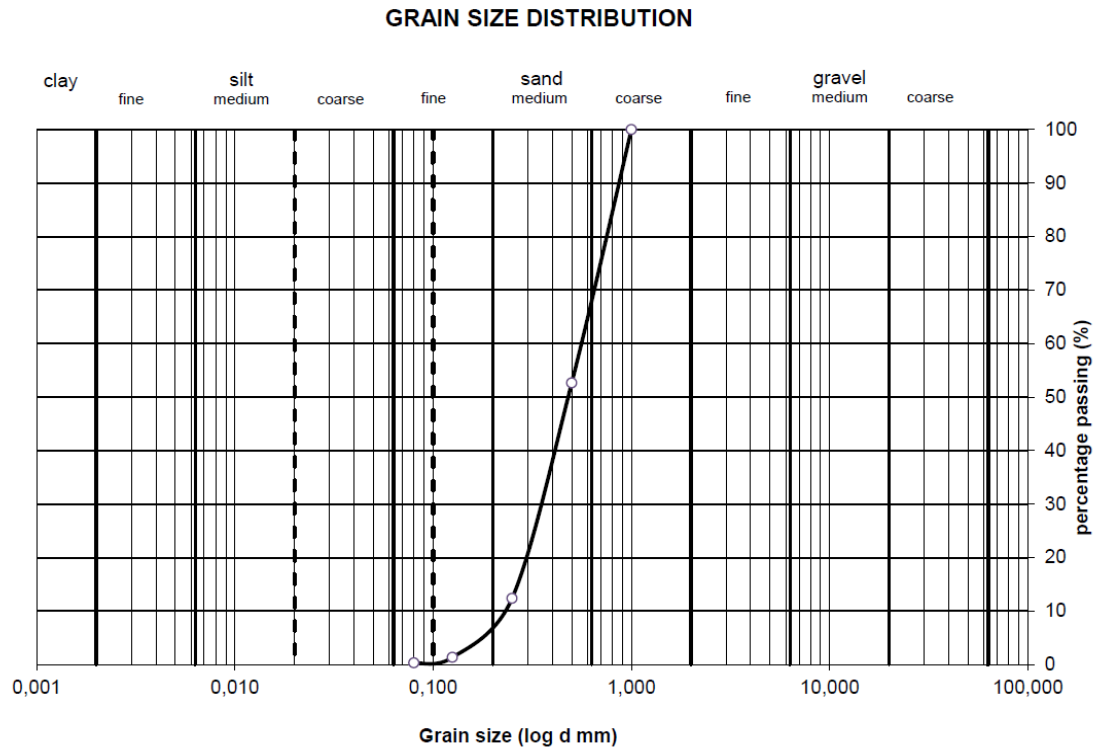


Fig. 14: Grain size distribution of the sand used for the laboratory tests

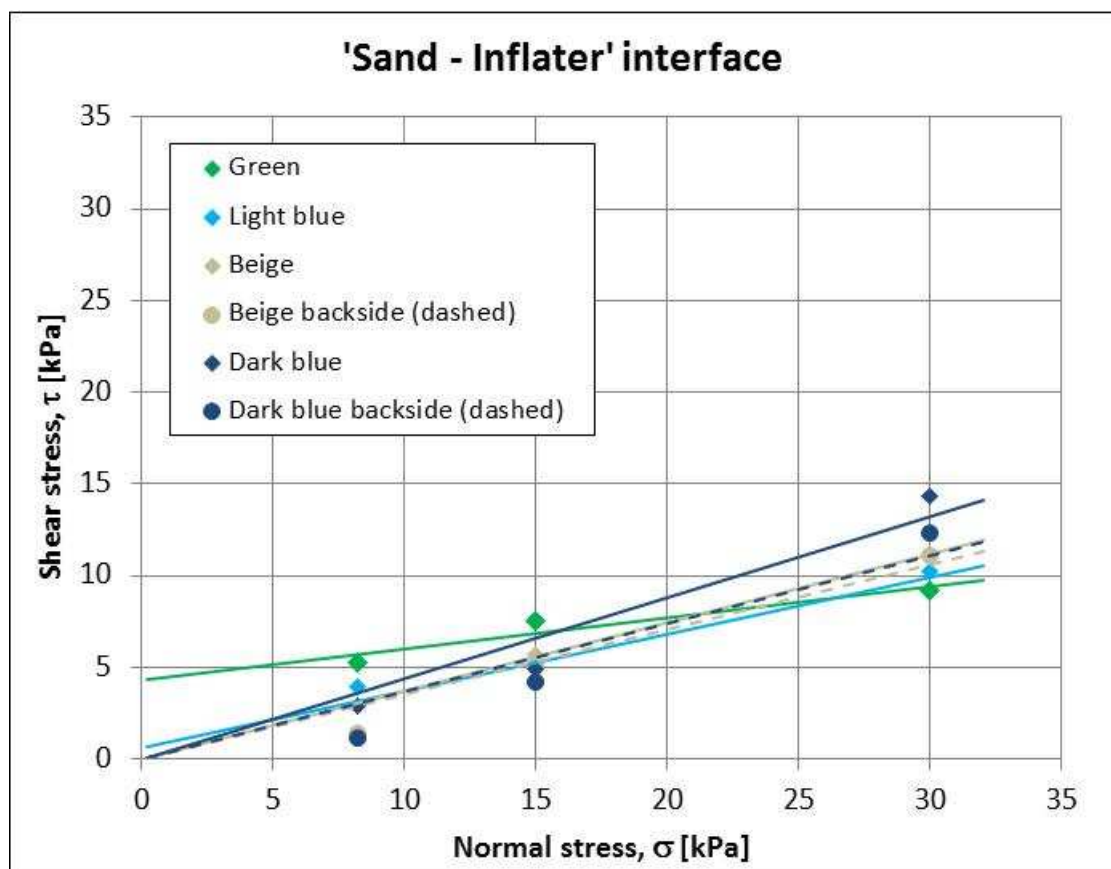


Fig. 15: Shear parameters of sand-INFLATER interfaces

The test results of sand-INFLATER interfaces are shown in Figure 15 and Table 4. We can see that some of the adhesion in some cases becomes negative which is physically impossible, so we substituted these values with zero. The measured values show us that this interface doesn't perform as good as the previous one, but according to the results the values are quite close to each other. The ± 4 kPa range was certainly caused by measurement uncertainties at low normal stress level and not by the interface behaviour.

<i>INFLATER material</i>	<i>Friction coefficient, $\tan \delta [-]$</i>	<i>Friction angle, $\delta [^\circ]$</i>	<i>Adhesion, a [kPa]</i>
Green	0.17	9.7	4.3
Light blue	0.31	17.2	0.7
Beige	0.37	20.5	0.0
Beige backside	0.35	19.6	0.0
Dark blue	0.44	23.9	0.0
Dark blue backside	0.37	20.4	0.0

Table 4: The resulted parameters of sand-INFLATER interfaces

In general it can be stated that the interface behaviour of these INFLATER materials with sand are very similar. Except the green material which is somewhat underachieving no significant differences can be found between the measured shear resistance parameters.

5.3.3. Cohesive materials: clay

The other typical soil type is the group of fine grained or the cohesive soils. The clayey material chosen for laboratory testing has the following properties:

Plasticity index: $I_p = 36\%$

Liquidity index: $I_L = 0.31$

Void ratio: $e = 0.78$

These properties indicate a high plasticity clay, in a plastic state and medium density. We have chosen and set the material's density and consistency to match the average clay's properties which is typical at flood protection sites. The shear behaviour and the parameters of the clay-inflater interface are shown in Figure 16 and Table 5.

The shear resistance of this interface is even lower than the previously tested ones. Just like in the case of sand the shear stress intercept scattered around zero between -2 and +3 kPa. This proves the idea that adhesion between these surfaces is negligible, and the shear resistance is caused only by the friction.

The determined friction angles varied in the range of 10-20°. The light blue material obtained somewhat lower values, but with the higher adhesion component the total interface shear resistance is similar to the other values.

The interface of clay and green material, in contrary to the other combinations has extremely small friction angles and also total resistances. Such low unit resistance requires larger skirt length, which obviously means higher material costs too.

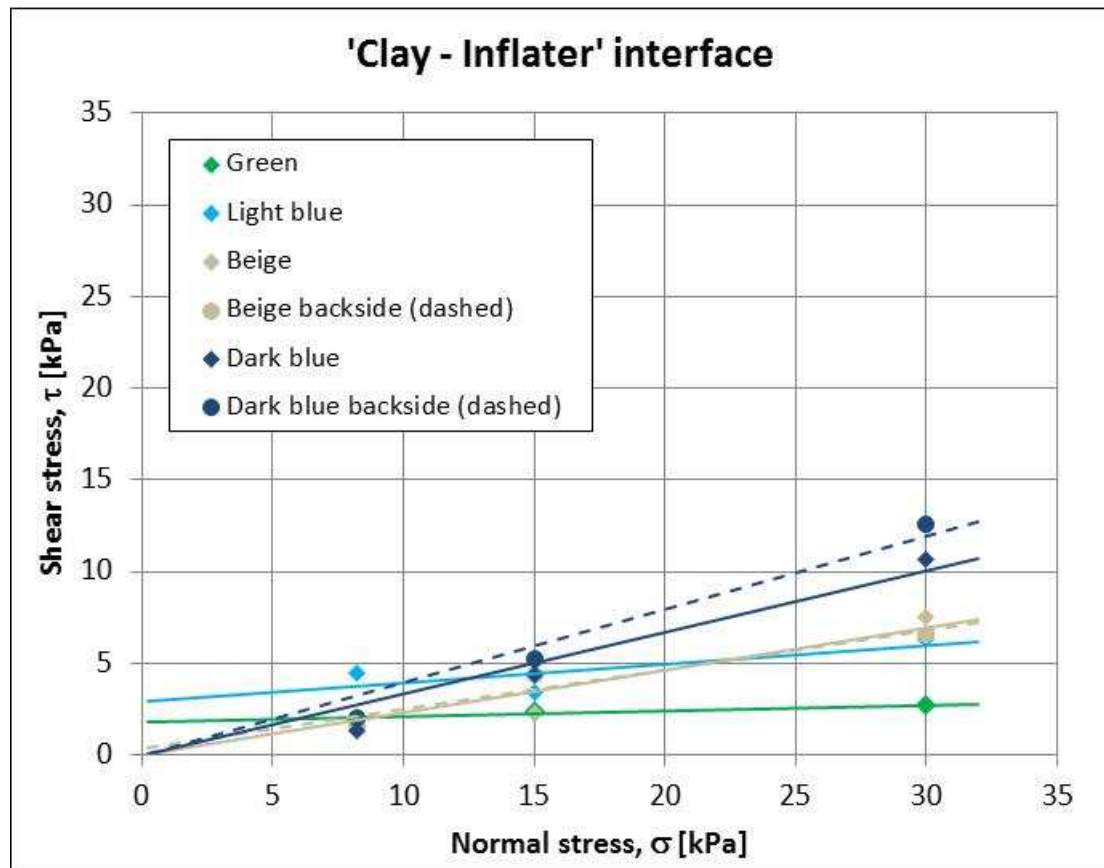


Fig. 16: Shear parameters of clay-INFLATER interfaces

INFLATER material	Friction coefficient, $\tan \delta$ [-]	Friction angle, δ [°]	Adhesion, a [kPa]
Green	0.03	1.8	1.8
Light blue	0.10	5.9	2.9
Beige	0.23	13.1	0.0
Beige backside	0.22	12.2	0.3
Dark blue	0.33	18.5	0.0
Dark blue backside	0.40	21.7	0.0

Table 5: Shear parameters of clay-INFLATER interfaces

5.3.4. Grass

At the potential application sites the soil is often covered by vegetation, mostly grass. This vegetation can significantly influence the inflater-subsoil interaction; therefore laboratory tests have also been performed to obtain the shear parameters of this interface too. Grass blades have been used and placed on clayey soil during the tests. The results of these tests are summarized in Figure 17 and Table 6. It can be stated that the shear resistance in general is the weakest of all



Pic. 39: Concrete, sand and grass in a short section

interfaces, so placing the inflater on an embankment covered by grass might be the most critical circumstance. Similarly to the previous cases minimal adhesion could be experienced on the interface, so even in this case the friction is the most important component of resistance.

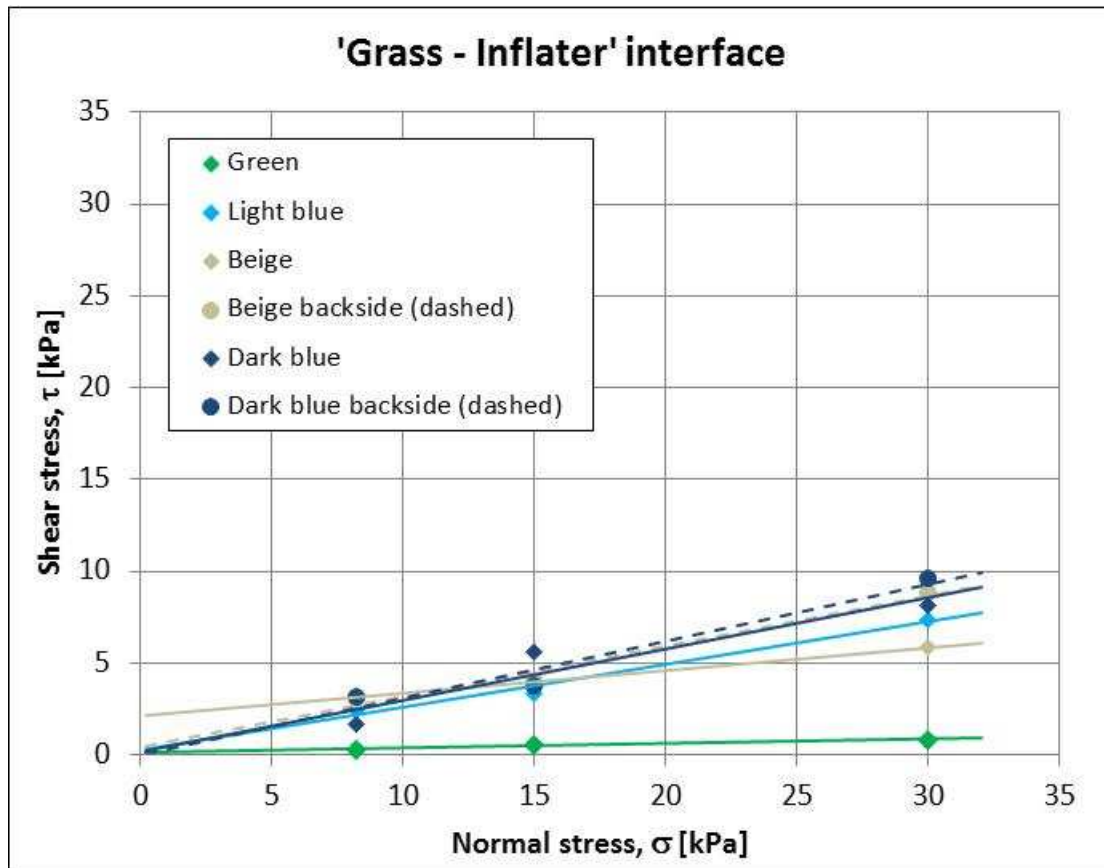


Fig. 17: Shear parameters of grass-INFLATER interfaces

In this case the friction angle varied between 7° and 17°, except by the green INFLATER material which underachieves with its 1.4°. Also this time the adhesion was again very small, so it can be stated that this interface has almost zero resistance against sliding even if significant normal stress (e.g. water pressure) is applied.

The other interfaces' results show similar friction angles, only the grass-beige material interaction seemed to be somewhat weaker. Its friction coefficient was only half of the other interfaces' values.

<i>INFLATER material</i>	<i>Friction coefficient, $\tan \delta$ [-]</i>	<i>Friction angle, δ [°]</i>	<i>Adhesion, a [kPa]</i>
Green	0.02	1.4	0.1
Light blue	0.23	13.1	0.3
Beige	0.12	7.0	2.1
Beige backside	0.27	15.4	0.4
Dark blue	0.28	15.5	0.2
Dark blue backside	0.31	17.3	0.0

Table 6: Shear parameters of grass-INFLATER interfaces

5.3.5. Concrete

We have fabricated a special concrete specimen for our test. With a maximal aggregate grain size of 2 mm we could make a concrete to match our requirements. In the age of one week we split it into a 60x60x15 mm prism and testing could begin. *Figure 18* and *Table 7* illustrate and summarize the obtained results.

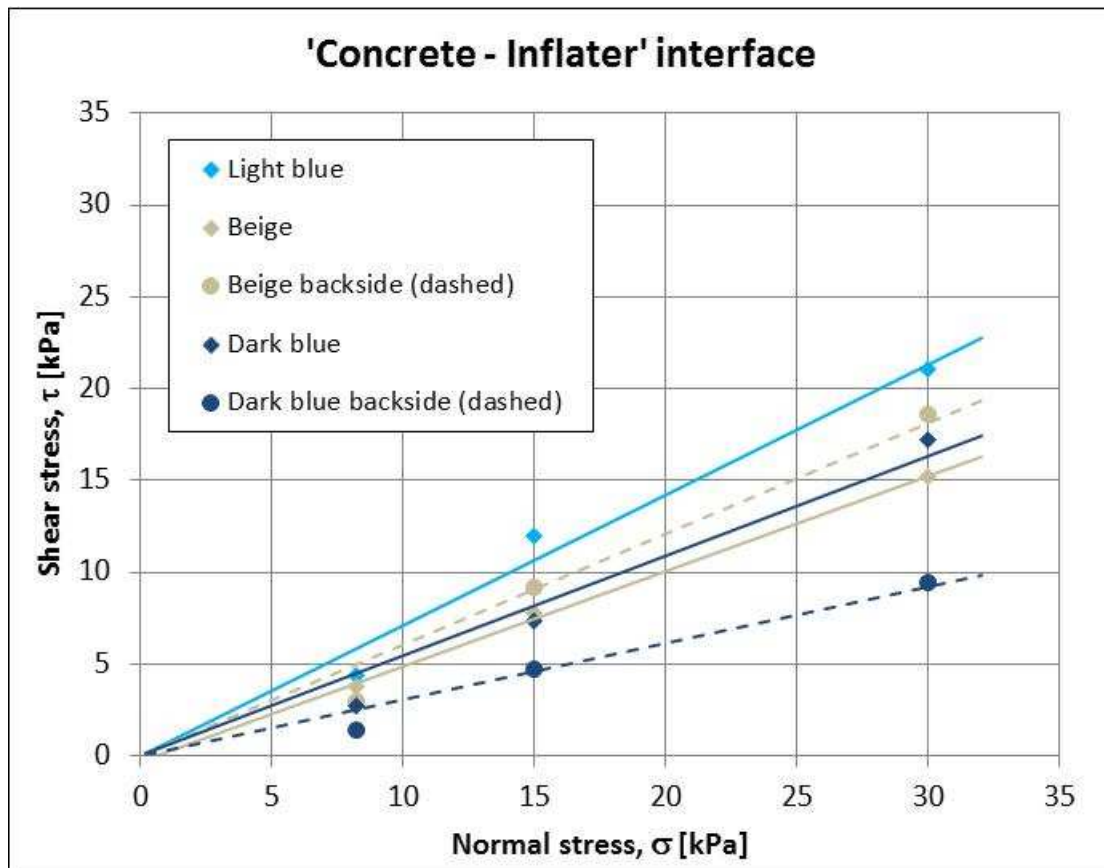


Fig. 18: Shear parameters of concrete-INFLATER interfaces

The shear stress intercept values are between -2 and 0 kPa “as usual”, so similarly to the sand-INFLATER and clay-INFLATER interfaces this value has been set to zero. The typical friction angle values vary over the range of 25-35°, only one lower value can be experienced for the backside of the dark blue material which is somewhat like nylon. The obtained friction angle values are very similar to the asphalt-INFLATER interaction.

<i>INFLATER material</i>	<i>Friction coefficient, $\tan \delta$ [-]</i>	<i>Friction angle, δ [°]</i>	<i>Adhesion, a [kPa]</i>
Light blue	0.71	35.5	0.0
Beige	0.51	26.9	0.0
Beige backside	0.61	31.3	0.0
Dark blue	0.54	28.6	0.0
Dark blue backside	0.31	17.1	0.0

Table 7: Shear parameters of concrete-INFLATER interfaces

5.3.6. Summary of laboratory test results

The laboratory tests clearly showed that out of the shear resistance's two components the friction angle is dominant and the adhesion is significantly smaller than that in these circumstances. The adhesion is often scattered around the zero value on both the positive and the negative sides. This implies that the resulted values of adhesion are rather test uncertainties than the interfaces' real physical behaviour. According to these statements we can simplify the original interface resistance equation by removing the adhesion:

$$\tau = \sigma \cdot \tan \delta$$

So there remains only one interface property which describes the shear resistance: the friction coefficient ($\tan \delta$) or friction angle (δ). This also means that the failure line in the σ - τ coordinate system have zero intercept on the τ (shear stress) axis.

It could be seen that the stiffer the INFLATER material the higher the interface resistance is and vice versa. Regarding to the potential INFLATER materials the following experiences can be concluded:

The tested INFLATER material – embankment material interfaces have a very similar behaviour in general, there were only a few combinations where a material performed significantly worse than the others. If we had to we would pick the green material which had the worst results in most of the cases, especially on grass. Such low resistance would require too large skirts to provide the stability of the INFLATER. Therefore this material is too uneconomical compared to the others and so the use of the green material is not recommended.

There were two materials (the dark blue and the beige) of which the front side and the back side evidently differed. By these two we used both sides for testing, but the results pointed out that there isn't much difference between the sides, we can't define any of them as "better". The performance of the front sides and back sides altered according to the in-situ material. Although there weren't significant differences, the back side of the dark blue material seemed to be a bit weaker when placed on concrete. Thus it has been concluded that the front side of the dark blue material is recommended, and the beige materials both side performs very similarly; so any of the two sides can be used.

Based on the experiences of the laboratory tests we have chosen three materials for further investigations: the light blue, the beige and the dark blue INFLATER material. To evaluate their performance larger scale field tests have been performed.

5.4. Field testing method

The field tests are very similar to the laboratory tests, but in a larger scale. The basic principle is the same: we place certain amount of load on the INFLATER material sample and then measure the applied force which is needed to get the whole thing move horizontally. The maximal value of shear force is recorded and with the known scales of the sample we can easily calculate the shear stress and the normal stress on the interface. The measured values are then plotted in the same way as for the laboratory test (i.e. the failure causing σ - τ stress combinations are plotted in σ - τ coordinate system) and the failure condition is defined by a linear σ - τ function.

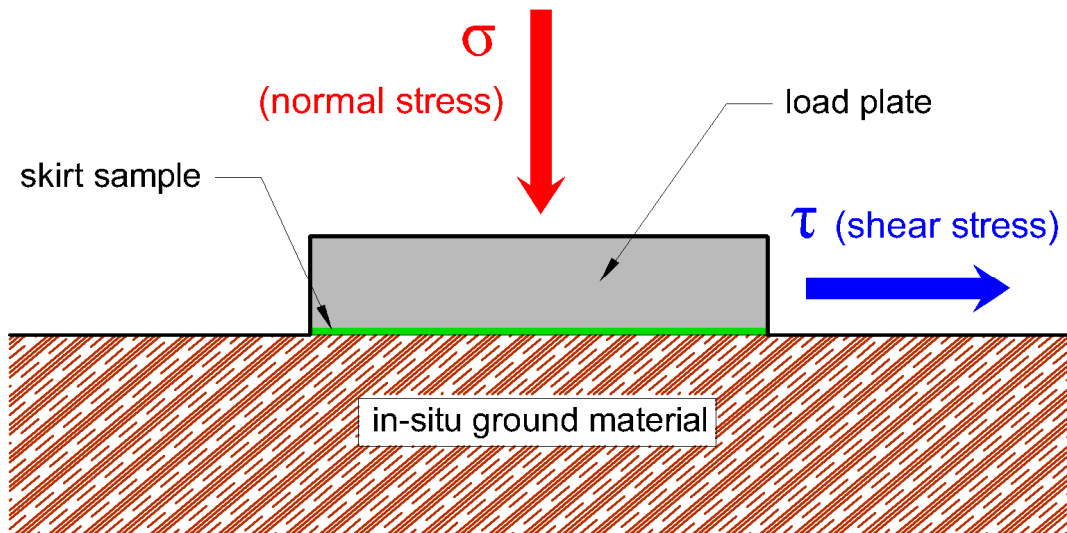
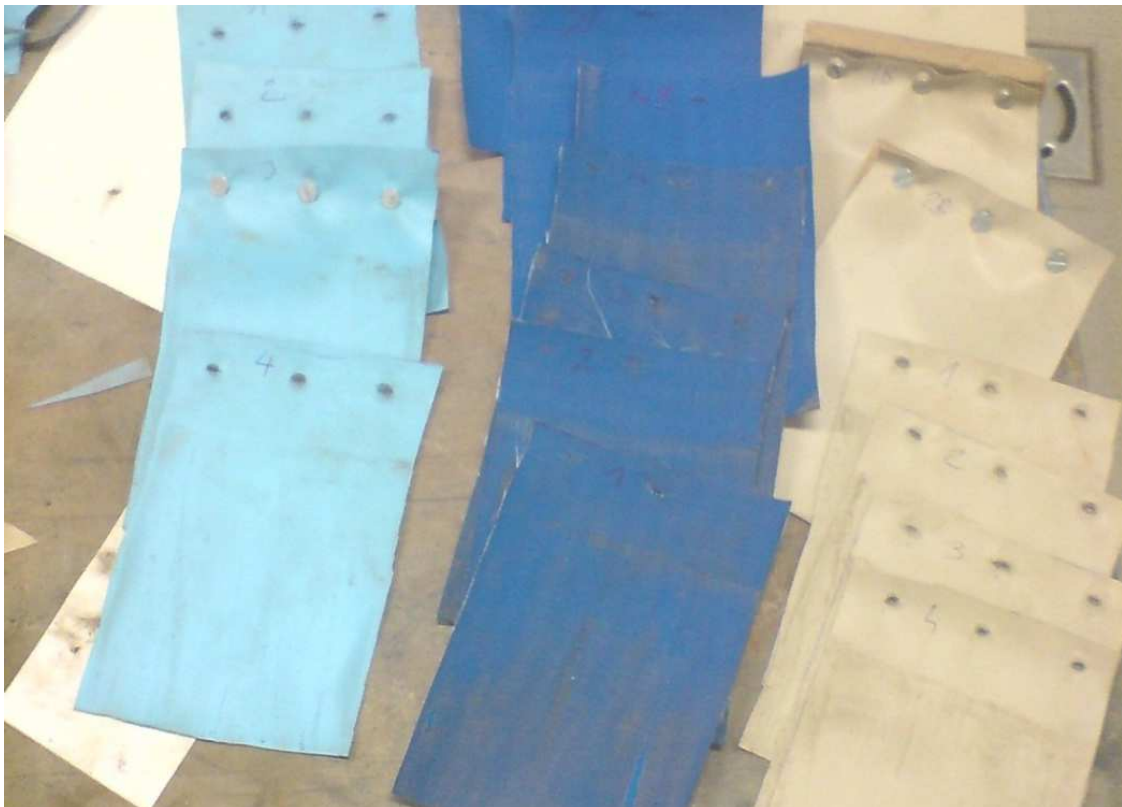


Fig. 19: The schematic sketch of the field shearing test

The sample size used for the field tests was 13x15 cm. For each test 12 measurements have been made using 3 different normal stresses (5.5 kPa, 10.2 kPa, 15.7 kPa). The INFLATER materials used for the tests are shown in Picture 40, the field test set up and the tested in-situ materials can be seen on Pictures 41-45.



Pic. 40: INFLATER material samples used for the tests



Pic. 41: Field test on well-maintained grass



Pic. 42: Field test on clayish soil under a tree



Pic. 43: Field test on a concrete prism



Pic. 44: Field test on sand next to a sandbox in a playground



Pic. 45: Field test on asphalt in the university's parking lot

5.5. Field test results

The results of field measurements ended up with similar experiences as the laboratory ones. Typically the shear stress intercept values (with other term the adhesion values) are again very close to zero, but with an even smaller scattering. Thus it seems obvious to say that the adhesion is negligible and the examined interfaces' shear behaviour can be exactly described by solely the friction coefficient.

The summarized results of field tests are shown in *Table 8* which includes the friction coefficient ($\tan \delta$) and friction angle (δ) values for all tested material combinations. In the bottom of the table you can also see the experienced maximum, minimum and mean values for each inflater material. All of these results are also showed in *Figures 20-24*.

The obtained values scatter within a quite small range and are also somewhat larger in general than the values of the laboratory tests. The average friction coefficient is very similar for all the tested INFLATER materials: they vary between 0.52-0.57. This indicates that there are no significant differences between the tested INFLATER materials. This statement is in good agreement with the laboratory test results. However the laboratory test results have shown larger differences among the various in-situ materials: for stiffer materials (for example concrete or asphalt) larger friction coefficients have been observed. In the case of softer materials (like clay or grass) lower interface resistance has been experienced. In this study we won't pan about this, but the contradiction of the measures needs further investigations.

The similar interface properties imply that all of the tested materials are by their shear resistance appropriate enough to use them in the INFLATER or in its skirt. The optimal solution can be chosen based on economical considerations or on individual material properties (for example workability, strength or durability).

Materials	Light blue		Dark blue		Beige	
	$\tan \delta [-]$	$\delta [^\circ]$	$\tan \delta [-]$	$\delta [^\circ]$	$\tan \delta [-]$	$\delta [^\circ]$
Concrete	0.54	28.3	0.57	29.5	0.55	28.8
Asphalt	0.54	28.5	0.63	32.2	0.52	27.4
Sand	0.48	25.6	0.53	27.8	0.51	26.8
Clay	0.56	29.5	0.69	34.8	0.59	30.4
Grass	0.58	30.2	0.44	23.7	0.44	23.6
Minimum	0.48	25.6	0.44	23.7	0.44	23.6
Maximum	0.58	30.2	0.69	34.8	0.59	30.4
Mean	0.54	28.4	0.57	29.6	0.52	27.4

Table 8: The results of the field tests

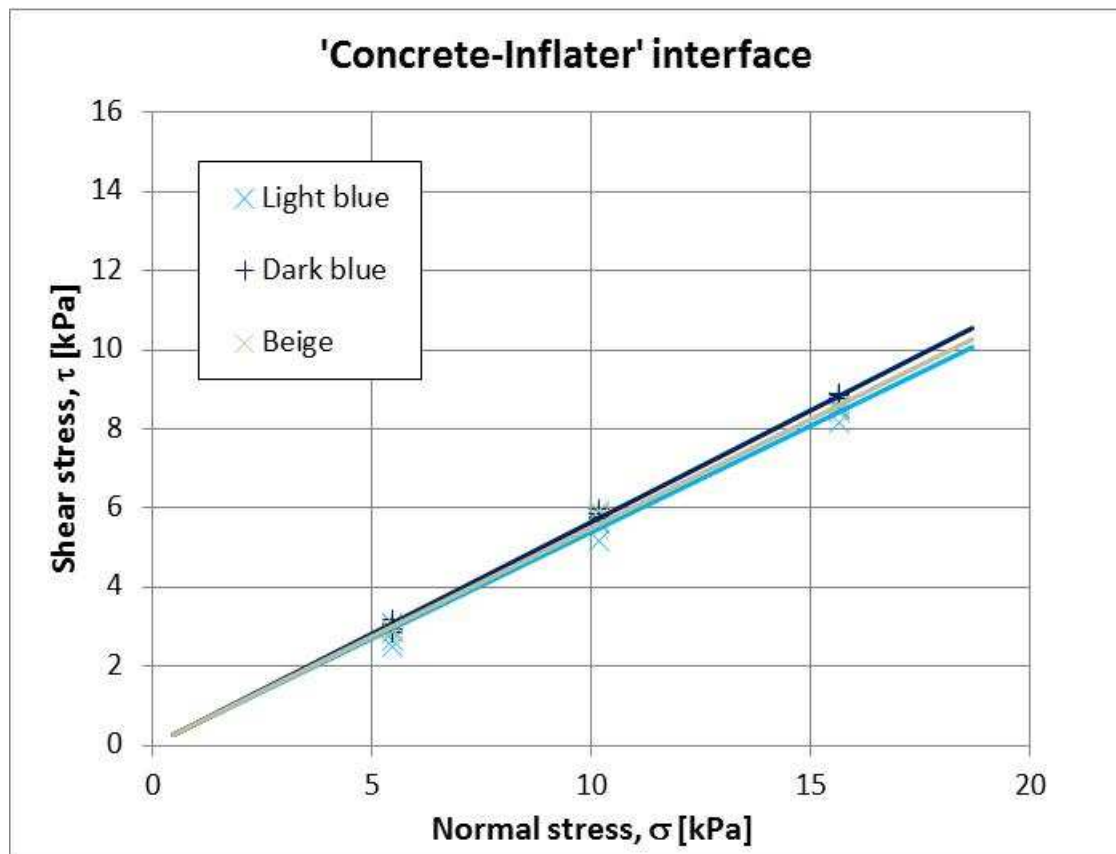


Fig. 20: Shear parameters of concrete-INFLATER interfaces in the field test

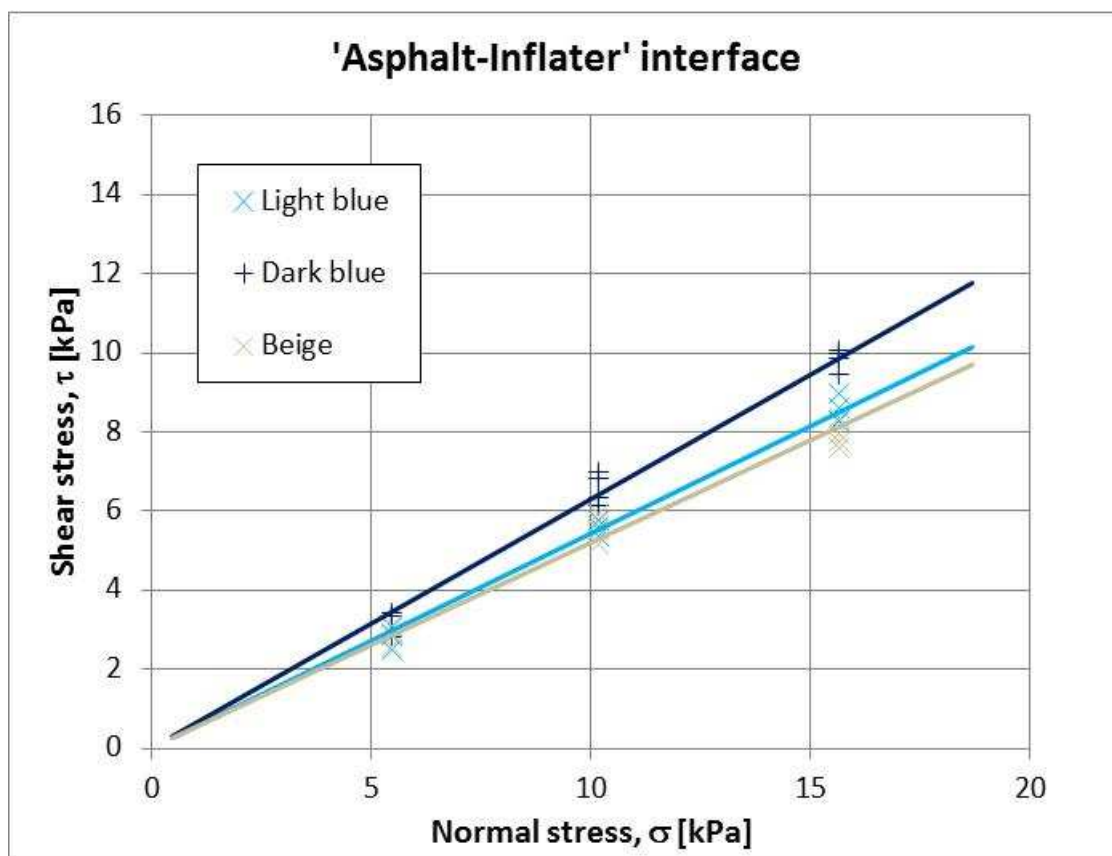


Fig. 21: Shear parameters of asphalt-INFLATER interfaces – field test

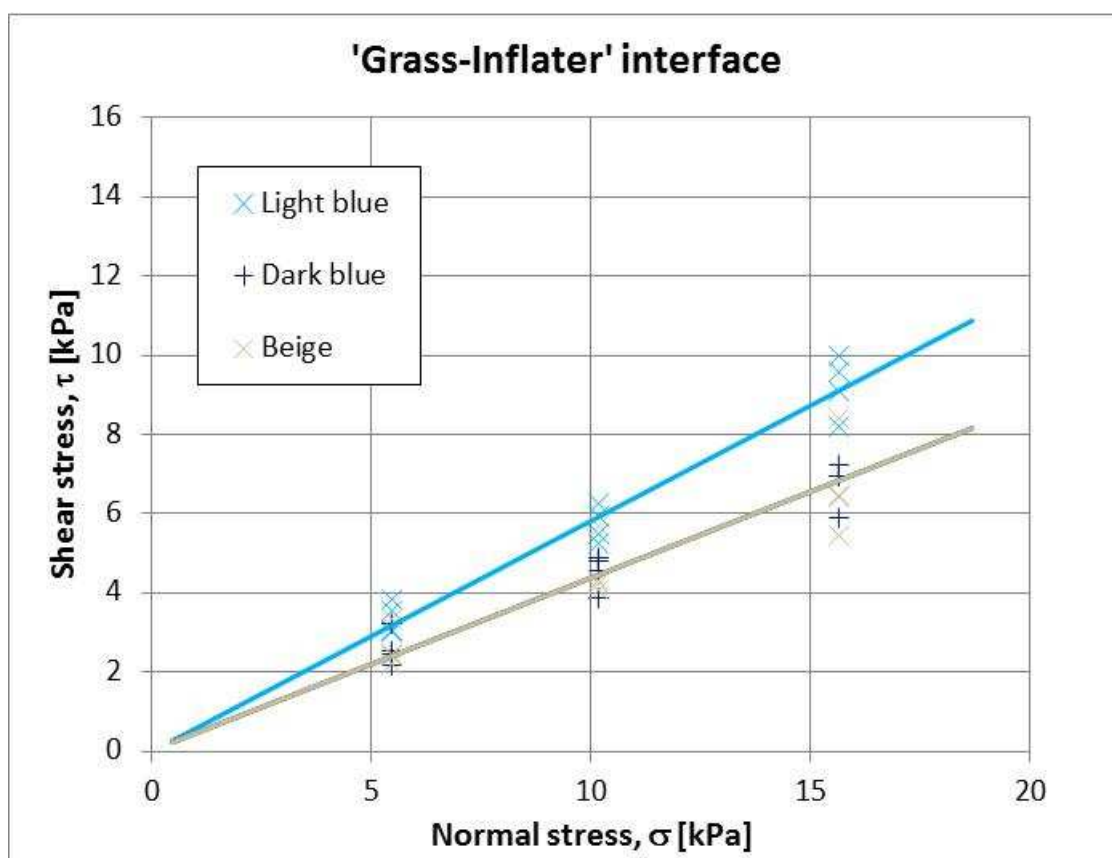


Fig. 22: Shear parameters of grass-INFLATER interfaces – field test

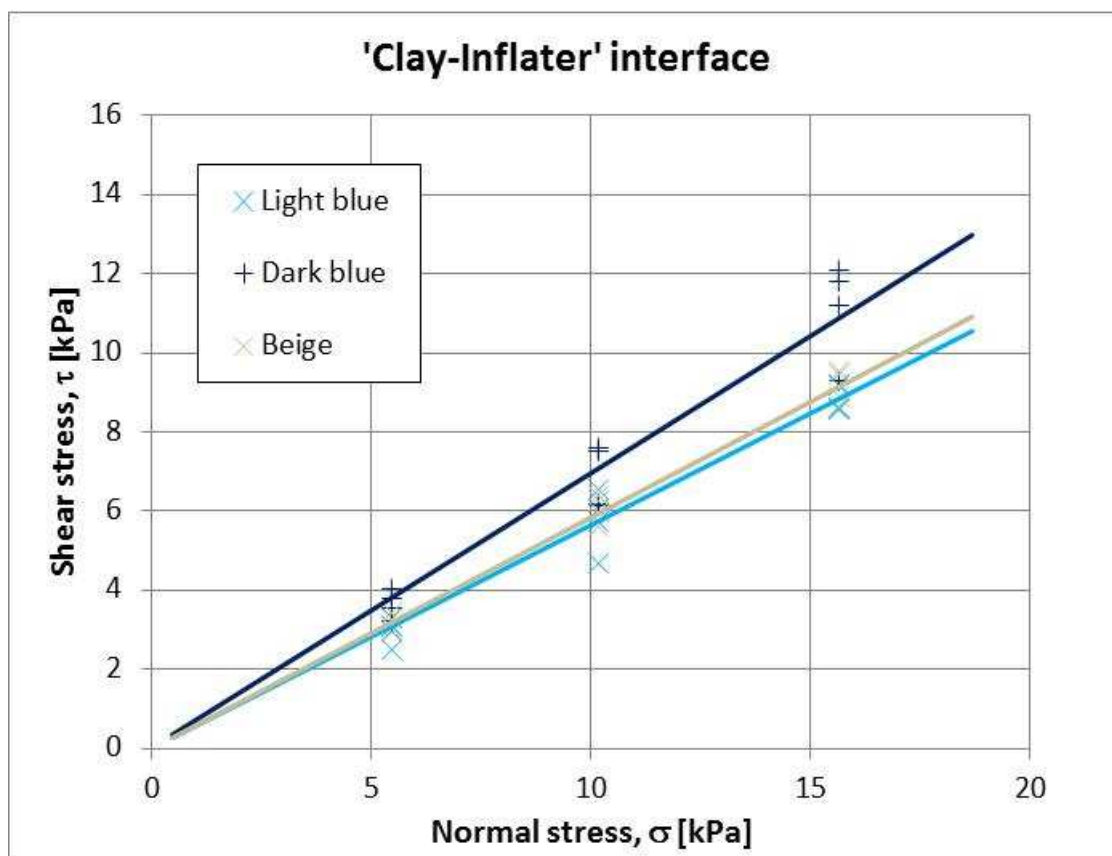


Fig. 23: Shear parameters of clay-INFLATER interfaces – field test

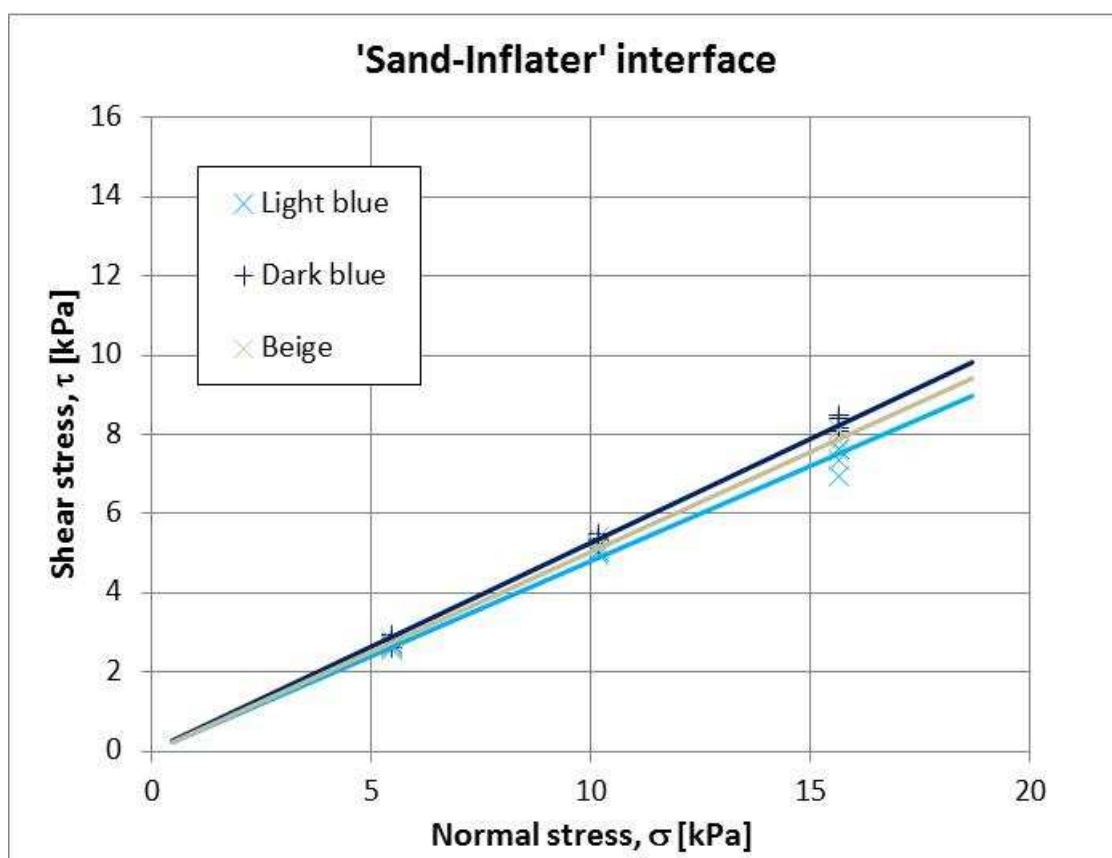


Fig. 24: Shear parameters of sand-INFLATER interfaces – field test

5.6. Summarizing the test results

All of the performed interface shear tests' results clearly indicated that the shear resistance between the INFLATER material and the underlying material is dominantly the function of friction. Our measures showed that the adhesion scatters always close around the value of zero. Thus neglecting it and defining the shear resistance by only its friction component is an acceptable and reliable way. In this case the value of shear resistance can be expressed with this equation:

$$\tau = \sigma \cdot \tan \delta$$

So the shear resistance of the interface is a function of the normal stress (σ) and the friction coefficient ($\tan \delta$). The normal stress can be typically determined with boundary conditions, such as the height of water level, flood durability, permeability of the underlying materials, etc. These conditions are time dependent: normal stresses are decreasing with time which means that the resistance is also decreasing.

The real interface property for a certain material combination is the friction coefficient. Series of laboratory and field tests have been performed to obtain the required parameters for different interfaces. The laboratory test results (*Tables 3-7*) pointed out that the stiffer the in-situ material the more shear resistance develops. The softer materials' resistance was always under these values. While the friction coefficients of stiff materials like asphalt or concrete varied between 0.4 and 0.7, the softer materials showed quite lower results with between 0.2 and 0.3.

Based on their performance three INFLATER materials have been selected for further investigations by field tests. This larger scale test method ended up with very similar results (*Table 8*). For each material combination the measured friction coefficients varied between 0.44 and 0.69 which means that we need the shear resistance to be 44-69% of the normal stress to prevent failure.

We experienced some kind of contradiction between the laboratory and the field test results. As we mentioned above by the laboratory tests we noticed that the stiffer in-situ material had higher shear resistance, but this didn't appear in the results of the field tests where all friction coefficients are nearly the same. Further investigation is needed to clarify this contradiction.

Based on the laboratory and field test results, none of these INFLATER materials could be classified as the best. They all had nearly the same values of shear resistance, so the point of selection has to be some other aspect e.g. cost, durability, workability, etc.



6. Conclusion

Hungary is the country of water which means floods will always happen here. It is up to us how hard we try to protect our settlements, our homes against these disasters. In the study I showed some historical examples of what is a river capable of and also showed those protection methods which were used by our parents and grandparents. But the flood damages are still too high and we have to look for other solutions as soon as we can. One of these opportunities is the INFLATER.

The development isn't ended yet and we still have a plenty to do until the INFLATER technology is ready to use, but it is definitely a very promising solution. Its most important benefits are quick installation and cost-effectiveness and we know from our history that after reliability these two are always the primal requirements. Hopefully in the close future this flood protection system will be set up on a levee and we can see it defeating the waves and saving people's lives.

29.10.2012.

Péter Szathmáry



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