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Performance of geosynthetic systems in waste landfills and validation through field monitoring

Analysis of a case study: “Chivasso 0” landfill (Turin)

Tutor/Correlator:
Prof. Ing. Claudio Oggeri
Prof. Dulce Gómez – Limón Galindo
Ing. Giuseppe Biolatti

Candidate:
Capozzo Simone

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Summary

1. Normative framework

A landfill, using the definition given by the Directive 1999/31/EC, is *"a waste disposal site used for the deposit of waste on or in the earth (i.e. underground), including*

- *an internal area for the disposal of waste (i.e. a landfill where the disposal of waste takes place on the same site where the waste was generated and by the person who generated it), and*
- *an area used permanently (i.e. for more than one year) for the temporary storage of waste (Directive 1999/31/CE) but excluding*
  - *facilities where waste is unloaded in order to be prepared for further transport to a recovery, treatment or disposal facility;*
  - *storage of waste awaiting recovery or treatment for a period of less than three years as a general rule;*
  - *storage of waste awaiting disposal for a period of less than one year".*

Within the circular economy involving waste and its reuse, a landfill is a temporary or permanent storage area for waste that can neither be recycled nor subjected to energy valorisation treatments such as biological treatment, pyrolysis, incineration.

In this chapter a legislative framework will be carried out to present the guidelines for the correct management of a landfill and the rules governing its correct location. It will proceed with a framework at EU level and then narrow down to national and regional level.

1.1 EU legislation

At EU level there are a number of reference directives that show the procedures to implement proper landfill management in order to prevent contamination of the different environmental matrices. The following directives present the technical-operational requirements to be met by landfills and the waste introduced.

- Directive No. 849/2018/EU,
- Directive No. 850/2018/EU,
- Directive No. 851/2018/EU
- Directive No. 852/2018/EU

These directives amend and supplement previous directives in terms of the management of landfills (1999/31/EC), packaging waste (1994/62/EU), and particular types of waste such as electrical and electronic equipment (2012/19/EU), end-of-life vehicles (2000/53/EU), and batteries, batteries and accumulators (2006/66/EU). These additions and amendments were made in order to update the regulations to the technological development of the materials industry, changing social behaviour and the environmental constraints to be respected. Correct waste disposal is a priority nowadays and the directives on landfill and waste are aimed at increasing the percentage of waste going to recycling (65% for municipal waste by 2035, 70% of packaging waste by 2030) and reducing the percentage of materials going to landfill (maximum 10% of municipal waste produced). This is just one aspect that characterises the well-established concept of the 'circular economy', a new economic model that envisages the reuse, recycling and then disposal of a material, seeking to increase the useful life of the object in question. The correct application of this model implies the collaboration of different entities and figures involved in the various states of the useful life of a material, starting with the selection and extraction of raw materials and moving on from the producer to the consumer. Following Directive 1991/31/EC, waste disposal establishments are divided into three categories:

- landfill for hazardous waste
- landfill for non-hazardous waste;
- landfill for inert waste.

Directive 1999/31/EC contains the definition of waste (Article 2) and the guidelines that must be followed to obtain a permit to operate and construct a landfill. Annex 1 of Directive 1999/31/EC will be presented below in order to show the criteria and parameters required for the proper siting of a waste disposal site.

1.1.1 Annex 1 – Construction criteria for inert waste landfill sites

In the annex mentioned before, the constraints that an inert waste landfill must comply with for proper siting are presented. In fact, the landfill cannot be located in:

- areas affected by seismic and volcanic activity (presence of active faults)
- karst formations and sinkholes
- areas affected by superficial geomorphological changes due to erosion, landslides, riverbed modifications.
- areas potentially affected by overflowing and flooding of watercourses during flooding phenomena with a return time of 50 years.
- natural areas subject to protection and preservation

For each selected area that respects the constraints described above, it is necessary to assess the distances from sensitive bodies such as:

- built-up areas
- road infrastructures
- relevant historical, artistic, archaeological and landscape assets

In addition, it is preferable to locate the landfill in degraded areas that can allow the storage of waste in stable conditions.
Within the same annex, the requirements to be met to prevent the disposal site from polluting the environmental matrices such as soil, underground and surface water, and the atmosphere are also listed. The geological protection of the environmental matrices mentioned above must be achieved by means of a natural geological barrier and a drainage layer capable of draining the percolation fluids produced in the landfill (leachate) during its lifetime. An appropriate protective layer can be placed between these two barriers.

The natural (geological) barrier must provide sufficient protection to prevent pollution of the environmental matrices. The substrate of the base and banks of the landfill, for example the geological formations, must meet the permeability and thickness requirements listed below and validated through field surveys:

- $K \leq 1 \cdot 10^{-7} m/s$
- $Thickness \geq 1m$

The drainage layer consists of a CE-marked aggregate composed of gravel/chippings, with a grain size between 16 and 64 mm. The aggregate must have a uniform grain size and must comply with the content values of substances and elements such as carbonates, sulphates. The thickness of this drainage layer must be greater than 0.5 m and must have a suitable permeability coefficient to drain the leachate produced during landfill operations.
In order to assess the correct location of a landfill site, the substrate where the landfill is located must not only meet requirements related to permeability and layer thickness but must also be able to withstand the subsidence and loads related to the disposal site without reaching the operating and ultimate limit states that could compromise the stability of the complex.
The deformation modulus value must be greater than or equal to 50 N/mm^2 and calculated in the load range between 0.15 and 0.25 MPa, at the first load cycle.
In addition, again following the approach of the NTCs in force, the stability of the face of the waste delivered to the landfill, of the roofing system and of the base and banks of the landfill must also be assessed during the design phase, and these must be respected from the opening phase to the closure phase of the landfill.

Furthermore, it is necessary to mitigate the impacts that the landfill and its operations may have on the surrounding environment. This aspect is mainly related to:

- Dust and odour emissions
- Transport due to wind of landfilled materials and waste
- Impact on the road system
- Noise
- Fire risk

1.2 National legislation

At the national level, this section discusses the Italian directives and in general all documents that regulate the management of a landfill at the national level. A reference legislation is Law No. 117 of 4 October 2019, which concerns reforms related to landfills and waste. With regard to waste, they concern the classification and definition systems of waste, updates on the criteria and hazard bands and the application of concepts related to the circular economy model. On the other hand, with regard to landfills, the focus is mainly on the criteria for the eligibility of waste in landfills, technological updates of the plants in order to achieve the objectives proposed by EU directives.
In implementation of Law 117/2019, there are other legislative decrees transposing the aforementioned directive, such as Legislative Decree No. 121 of 3 September 2020.
At the national level, there are two other legislative decrees to consider:

- Legislative Decree No. 152/2006
- Legislative Decree 36/2003
- Legislative Decree No. 121/2020

Legislative Decree No. 152/2006, also called the 'Environmental Code', deals with soil and river basin protection, air quality and the concept of environmental impact assessment. More specifically, part 4 of the 6 that make up the text focuses on waste management.
Legislative Decree 36/2003 incorporates the same requirements shown in Annex 1 of Directive 1999/31/EC. It ensures a gradual reduction in the landfilling of waste suitable for recycling or other energy recovery processes, in order to apply the core concepts of the new circular economic model. Furthermore, the purpose of this regulation is to fulfil the requirements set out in Legislative Decree 152/2006 and the prediction of operational and technical requirements of procedures to prevent hazardous scenarios for the environment and its matrices (water, air, soil).
Legislative Decree No. 121/2020 incorporates Decree 26/2003 and amends parts of it.

1.3 Regional Legislation

The main objective of national and EU legislation is the recovery of materials and energy. Regional legislation will mainly have the task of implementing national and EU legislation through short, medium and long-term planning, with a focus on waste treatment. Furthermore, the second task will be to specifically draw up criteria for the location of landfills, adapting them to regional geomorphological conditions.
As far as the Piedmont region is concerned, it is necessary to initially assess the type of waste to be landfilled, since the location criteria change according to the type.
The analysis of the territorial context is conducted in order to identify the elements of vulnerability of the area affected by the settlement of the complex, such as the presence of infrastructures, cultural heritage, climatic factors, water bodies, biodiversity and other issues taken into account during planning.
The criteria for selecting suitable locations for the siting of facilities are set out in the Regional Special Waste Management Plan drafted by the Regional Council 16 January 2018. Within this plan there are criteria for special and municipal waste treatment plants. The regional plan regarding location criteria has undergone several subsequent amendments and additions over the years.

The criteria and procedures for identifying areas unsuitable for landfills are outlined in Regional Law 13.4.1995, no. 59 - Article 2.
For the correct location of the plant, at regional level it is necessary to identify areas to be excluded and which are therefore not taken into consideration, and to create an unambiguous methodology to arrive at comparable results at the detailed stage.

The correct procedure for identifying a suitable area is divided into 3 phases.
The first phase, also called the exclusion phase, consists of the identification of unsuitable areas that will then be eliminated from the preliminary selection. The exclusion criteria concern:

- Presence of urban centres (due to danger of pollution and bad smells)
- Presence of natural areas: the landfill cannot be located within 300m of public parks and protected natural areas
- Presence of ecological areas where there is the presence of protected species
- Presence of protected areas (soils, groundwater)
- Presence of Areas with stagnant surface water

This phase is a regional competence referring to the Regional Waste Management Plan.

The second step concerns the attribution of numerical values to the areas according to the different criteria. The values attributed vary between 0 (insignificant criteria) and 2.5 (very important criteria) and are attributed to the following criteria:

- Proximity of urban centres
- Proximity to water areas
- Proximity to road infrastructure
- Proximity to airports
- Proximity to green areas
- Vicinity to military, industrial areas, usable by the population
- Geological, hydrogeological constraints

These criteria may be modified by the landfill designer to give more or less emphasis to criteria specific to the specific landfill. This phase is carried out at provincial level, applying the rules contained in the Territorial Coordination Plan.

The third and final phase concludes the operation of selecting suitable areas by evaluating the scores assigned to different areas, with a final value corresponding to the contribution of all the individual factors. The criteria used for the final selection are:

- Environmental criteria
- Territorial criteria
- Landscape criteria
- Political and legal criteria
- Economic and financial criteria

After this phase, and after a detailed analysis for the solutions deemed suitable, the final area where the disposal site will be located will be selected.
The choice of location for disposal facilities must also consider the homogeneous distribution of disposal facilities throughout the territory in order not to overload one facility rather than another. In general, moreover, it is always preferable to upgrade and technologically adapt existing plants in order to minimise the territorial impact. Still for the same concept, the selection of degraded areas is preferable.
The characteristics of the areas selected for the location of disposal plants relate to the average topographical height of the excavation bottom on which the lower layer of the confinement barrier is grafted, which must be positioned above the maximum height reachable by the water table.
As far as the geological barrier separating the landfill from the substrate on which it is grafted is concerned,

the permeability value remains unchanged with respect to what is reported by European and national standards, while the thickness varies from 1m to 1.5m with respect to what was proposed by European and national standards. The other characteristics remain theoretically unchanged, unless field surveys show that the requirements are not met.

1.3.1 Criteria for the correct ubication

The location criteria are summarised in this paragraph.
Regarding urban and spatial aspects:

- It is forbidden to build around cemeteries within an area with a radius of 200m from the perimeter of the cemetery area.
- It is forbidden to build within buffer zones for road infrastructures, which correspond to 60m for motorways, 40m for highways, 30m for medium communication roads, 20m for local roads, 30m for railways, 300m for airports.
- It is forbidden to build near power lines, gas pipelines, aqueducts
- It is forbidden to build near industrial areas
- It is forbidden to build in the vicinity of other facilities in order to easily assess the source of possible contamination

For land use aspects:

- It is forbidden to build within agricultural and natural land where it is possible to affect the productivity and conservation of the area; of agricultural land intended for the cultivation of AOC products; of land with low water consumption irrigation systems
- It is forbidden to build within the buffer strip, equal to 300m from the external perimeter of the area, of agricultural areas dedicated to the cultivation of PDO, PGI products
- It is forbidden to build within areas with woods and forests
- It is forbidden to build in areas with hydrogeological constraints

Regarding the protection of water resources:

- It is forbidden to build on land with a surface water table
- It is forbidden to build on land with the presence of water for human consumption, such as areas with large wells, aqueducts, reserve areas
- It is forbidden to build in the presence of dolines and other geomorphological formations affected by the karst phenomenon

With regard to the protection of natural resources:

- It is forbidden to build near nature reserves, public parks
- It is forbidden to build near wetlands such as lakes, ponds, marshes, artificial reservoirs
- It is forbidden to build near oases for the protection of wildlife and natural areas sensitive to anthropic phenomena
- It is forbidden to build within the buffer zone (300 m from the shoreline) of areas with landscape restrictions
- It is forbidden to build within the buffer strip (150m from the foot of the banks) of rivers
- It is forbidden to build within the buffer zone of mountain ranges and Apennines and near glaciers
- It is forbidden to build in areas with buildings and infrastructures of public interest, areas of historical importance, areas of very high aesthetic value
- It is forbidden to build in areas of high scenic and visual value

For protection from disasters and calamities, it is forbidden to build in areas affected by erosion, instability, flooding of watercourses.

For the protection of the population:

- It is forbidden to build close to inhabited areas and it is mandatory to protect inhabited areas from the propagation of bad smells due to the presence of landfills in the vicinity.
- It is forbidden to build anaerobic digestion plants at less than 500m from built-up areas.

2. Geosynthetics: functionality in environmental engineering

Geosynthetics have become increasingly popular in the engineering world in the last 40 years thanks to their ability to solve engineering problems such as instability of slopes and soils. They can be used in combination with natural materials to maximize the performance of engineering projects. However, most human activities and construction methods have a negative impact on the climate balance. In this scenario, geosynthetics offer such a solution and can be used in conjunction with traditional methods achieving the same mechanical, technical and design results.
Geosynthetics can perform a variety of functions, including reinforcement, filtration, drainage, separation, protection, waterproofing and confinement. They can perform several functions simultaneously, and one function can be performed by multiple geosynthetics materials. These functions can be categorized into primary and secondary functions, where the primary function is the main purpose of the geosynthetic application, and secondary functions are additional functions performed by the same material.

For instance, a geotextile can be used to separate two different granular materials placed vertically under a road pavement, while also facilitating filtration to prevent water accumulation that could increase hydrostatic pressure on the materials. In the next paragraphs the individual functions performed by geosynthetic materials will be examined in detail.

2.1 Reinforcement

Geotextiles and geogrids are the most commonly used geosynthetics in engineering applications. Geocells may also be used to perform similar functions. To ensure proper functioning, these materials must interact with a backfill material to develop stabilizing tensions that make slopes safe. Reinforced soils walls are an economical and efficient solution that provides the same functionality as a traditional concrete wall.
Fibre-reinforced soil is considered a homogenous material where the arrangement and the position of the fibres can influence strength values. The fibres provide isotropic or anisotropic resistance, which can limit the occurrence of instability mechanisms on potential planes. The use of geosynthetics can also help to prevent slope erosion.

Fig. 1 - Use of geosynthetics to prevent slope erosion (Oggeri and Capozzo, 2022)

However, it's important to provide a correct dimensioning and design for any scenario in which they are used to avoid the collapse of the affected area.
Overall, geosynthetics offer a sustainable solution to traditional engineering methods. By using geosynthetics in conjunction with natural resources, engineers can achieve the same results as traditional methods while minimizing negative impacts on the environment. Careful consideration and design are necessary to ensure the successful implementation of geosynthetics in any engineering application.

2.2 Filtration

Geotextiles materials are used to perform the filtration function, allowing liquid to move in a transverse direction to the plane of the geosynthetic while retaining soil on the upper layer. The permittivity of geosynthetics material and the retention capacity of the soil or overlying waste in landfills play a crucial role in filtration. Stable conditions must be maintained to prevent the flow of liquids through the geotextile from causing an unsustainable loss of soil. The size of the voids in the geotextile is critical, as soil particles could otherwise be transported into these voids and lead to the collapse of the overlying soil. As flow decreases, finer particles are retained within the geosynthetic, leading to clogging of the geotextile ports and a decrease in hydraulic conductivity.
Since, as stated earlier, the flow is perpendicular to the plane of the geosynthetic, the hydraulic conductivity

to be considered will be in the direction normal to it. Permittivity is defined as

$$\psi = \frac{k_n}{t}$$

where ψ is the permittivity, k_n is the hydraulic conductivity of the transverse plane and t is the thickness of the geosynthetic at a given normal pressure.
The designer must size the geotextile and its positioning correctly to avoid excessive clogging, and laboratory tests are strongly recommended for major applications. Tests like the apparent opening size test (AOS) can be used to determine the opening size of the geotextile, and tests like the slope ratio test, long-term flow test or hydraulic conductivity ratio test must be performed to determine hydraulic permittivity. Maintaining contact between the geotextile and soil is necessary to maintain the filtration phenomenon and prevent the development of voids behind the geotextile, and the material to be filtered must be consistent with the strength of the geotextile to prevent mechanical degradation and damage during operation.

2.3 Drainage

The primary function of geotextiles and geocomposites is to provide the drainage by allowing the flow of liquid within them in a direction parallel to the geosynthetic laying surface. This is important because it helps to prevent the accumulation of excess water within the soil, which can lead to soil instability and erosion. As with filtration, it is important to consider the retention of the overlying soil during the drainage process to prevent excessive loss of soil within the geosynthetic.
Thick non-woven geotextiles have a significant void space within their structure, which allows them to transport liquids in their plane at a rate of approximately 0.01 – 0.1 liters per second per meter of wide geotextile. Geocomposites, on the other hand, can drain liquids at a rate one or two orders of magnitude higher than geotextiles.
To assess drainage, it is necessary to investigate the transmissivity parameter, expressed by the following equation:

$$\theta = k_p \cdot t$$

where θ is the transmissivity, k_p is the hydraulic conductivity in the direction parallel to the plane of the geosynthetic, and t is the thickness of the geosynthetic at a given normal pressure.
This equation describes the rate at which water can flow through the geosynthetic material under a given hydraulic gradient.
It is important to state that the transmissivity of a geosynthetic can be influenced by several factors, including the type of material used, its thickness, and the hydraulic gradient applied. As a result, it is important to carefully evaluate these factors when selecting and designing geosynthetics for drainage applications.
The thickness of a geosynthetic material is an important factor to consider when evaluating its ability to perform drainage functions. The greater the normal pressure applied on the geosynthetic, the thinner it needs to be in order to maintain its transmissivity. The transmissivity of a geosynthetic is defined as its ability to transmit fluids in the plane of the geosynthetic and is inversely proportional to the normal pressure applied and the time factor due to creep phenomena. One practical application of geosynthetics for this reason is in the base lining and cover system of a landfill site, where the drainage layer is crucial in collecting leachate and directing it towards treatment facilities. Similarly, the drainage layer in the cover system is necessary to remove rainwater that infiltrates the landfill body, which can contribute to the formation of leachate.

2.4. Separation

The separation function is an important aspect of geosynthetics in engineering applications. It involves the placement of a flexible, porous geosynthetic material between different materials, such as soil and aggregate in order to maintain their functionality and integrity without any physical interaction. The primary geosynthetic material used for separation is the geotextile, which is designed to prevent mixing and

intermingling of adjacent materials, ensuring their stability and long term performance.
The separation function is particularly useful in civil engineering projects such as road construction, where the placement of a geotextile layer between the subgrade and aggregate layers can prevent soil particles from migrating into the aggregate layer and vice versa, as shown in fig. 2.

Fig. 2 - Separation function in road construction project (Jorge G. Zornberg, 2017)

In a similar way, in the construction of landfill liners, a geotextile layer is placed between the compacted clay liner and the overlying protective layer to prevent damage to the liner and maintain its effectiveness.
To realize in a proper way an intervention of separation, the selection of the appropriate geosynthetic material should be based on factors such as the type of materials being separated, the desired level of separation and the specific requirements of the projects.

2.5 Protection

The protection function of geosynthetics involves the use of flexible and porous materials, such as geotextiles, to protect other materials from mechanical damage resulting from the application of tension states over time. In landfill and liquid containment systems, geotextiles can play a crucial role in protecting geomembranes from contact with waste or other substrates.
For instance, in landfill base linings, a geotextile layer can be placed between the waste and the geomembrane to prevent damage to the membrane caused by the waste. On the other hand, a geotextile layer can be placed under the geomembrane in landfill covers to protect it from perforation or mechanical damage caused by contact with waste.
The selection of the appropriate geotextile material depends on several factors, such as the type of polymer and its characteristics, the expected service life of the project, and the manufacturing process of the geosynthetic. These factors should be considered during the design stage, and appropriate laboratory and field tests must be conducted to evaluate the geosynthetic's response to different situations. By taking these factors into account, designers can ensure that the protection function is properly performed by geosynthetics in the long term.

2.6 Waterproofing

The function of waterproofing is crucial in the construction of infrastructure, as it prevents water penetration and avoids unstable situations that can compromise the integrity of the structure. Composite geosynthetic liners, such as Geosynthetic Clay Liners (GCLs), are the main materials used to perform this function. However, it is important to note that the material that performs this function can deteriorate over time, so proper sizing of the material is crucial to ensure long-term effectiveness.
GCLs are commonly used in the waterproofing of irrigation basins, and they consist of a natural sodic clay liner between two geosynthetics, which provides excellent performance. When sizing the material, geological and geotechnical conditions must be considered, as well as the implementation of filtration and drainage networks that are established prior to the installation of a waterproofing layer.
The waterproofing system is typically composed of three components:

- a separation and/or protection layer made of a geotextile with a grammage of 500g/m^2 to protect the underlying geomembrane
- the geomembrane itself (PVC, PP or PE)

- a protection layer consisting of a geotextile combined with a layer of sand or cement to shield the waterproofing layer from external factors such as vandalism or UV radiation

During the installation process, welding must be carried out carefully to avoid contamination of the area. It is crucial to carefully study the context and functions that the waterproofing system needs to perform to ensure that the system functions optimally over time.

2.7 Confinement

The function of confinement is often performed by geosynthetics such as geomembranes and GCLs. These materials are chosen because of their very low hydraulic conductivity, which enables them to effectively contain fluid movement. There are several applications for these materials, such as the construction of road pavements to isolate the pavement from moisture and temperature changes, and to prevent cracks from appearing in the pavement. They are also used in storage tanks, channels, and containers that contain hazardous fluids that could contaminate groundwater.
In addition to their primary functions, these materials can also replace traditional methods or complement them, providing the same level of performance with less volume. For instance, they are used as base liners and cover systems in landfills to prevent leachate from reaching the underlying soil, as well as to prevent rainwater from infiltrating into the cells of the landfill and gases from escaping from the waste disposal plant. Overall, the use of geomembranes and GCLs for confinement requires careful consideration of the specific requirements of the project and the potential response of the materials to certain situations. Proper design, testing and installation are necessary to ensure the effective performance of these geosynthetics in their confinement function.

3. Geosynthetics: constituent materials

Geosynthetics are materials that are produced through industrial processes and are mainly composed of polymers. They are used in geotechnical and engineering applications where they interact with soil and rocks. The term "geosynthetics" is a combination of "geo," which refers to the earth, and "synthetic," which indicates materials produced through industrial processes. These materials can be found in different forms such as rolls, sheets, panels, plates, strips or three-dimensional structures, and are classified according to their structure or function.
The use of geosynthetics has many technical and economic advantages, leading to a significant increase in their use, sometimes replacing traditional methods in some contexts. For example, in the multilayer configuration of the bottom of a landfill, geosynthetics can be used instead of gravel and granular material. The use of a geomembrane (to counteract infiltration) and a few millimeters of geocomposite (as a drain) offers the same benefits as a layer of gravel and a few millimeters of granular material (used as a filter layer).
Geosynthetics are used in various applications, such as road pavements, storage tanks, channels transporting water or other substances, and containers containing hazardous fluids. They can also be used in base liners and cover systems of landfills, preventing leachate from reaching the underlying soil and preventing rainwater from infiltrating into the cells of the landfill and gases from escaping from the waste disposal plant. The materials used to produce geosynthetics come almost exclusively from the plastics industry, although fibreglass, rubber, steel and natural materials are sometimes used.

3.1 Geotextiles

Geotextiles are a type of geosynthetic material that is made up of fibers or polymer filaments that are combined to create the final product. Depending on the type of fiber used, geotextiles can be classified into monofilaments, multifilaments, staple filaments, and slim fit. Monofilament geotextiles are made by extruding molten polymer through small-diameter holes, cooling it, and then stretching it to increase the strength of the filament. Multifilament and staple filament geotextiles are made by cutting extruded molten polymers into small portions, spinning them together to form a yarn, and then creating a circular cross-section. Slim fit geotextiles are created by extruding a continuous sheet of polymer and cutting it into filaments using knives or air jets. These filaments have a rectangular and flat cross-section.

Fig. 3 – Geotextile

Geotextiles can also be categorized into wovens and nonwovens based on the weaving process. Woven geotextiles can be created using different methods, but needle punching and melt bonding are the most commonly used. The needle punching process mechanically interconnects the fibers to create a stable configuration, while melt bonding fuses and pressurizes the fibers to bond them together. The stitching and agglomeration of fibers depends on the intended use of the material, with sewn geotextiles being suitable for landfill filtration but not as a separation element, while thermal bonding allows for filtration and separation.
Most geotextiles are made from polypropylene resin, but other polymers such as polyester, polyethylene, nylon, and other resins can also be used. As geotextiles have different properties, various tests have been developed to evaluate their characteristics. These material tests are essential in verifying the correct use of geotextiles, which can be characterized by index and performance index properties.

3.2 Geomembranes

Geomembranes are thin and flexible sheets made of polymers like polyethylene, polyvinyl, and polypropylene. They have very low hydraulic conductivity, which is an important property that determines

their use in various applications. The most common types of geomembranes are high-density polyethylene (HDPE), very flexible polyethylene (VFPE), polyvinyl chloride (PVC), and flexible polypropylene (fpp).
HDPE geomembranes are rigid and have excellent resistance to stress, making them ideal for base and cover coatings. VFPE geomembranes are characterized by their linear structure due to the polymerization process and include types like ultra-low density polyethylene (VLDPE) and low-density linear polyethylene (LLDPE). PVC geomembranes are economically advantageous and can be produced in large panels, making installation easier. FPP geomembranes combine polypropylene and ethylene propylene elastomer (EPE) to offer similar flexibility to PVC geomembranes.

Fig. 4 – Geomembrane

Geomembranes are widely used in landfills as a base or cover coating in addition to low hydraulic conductivity soils, due to their chemical resistance, long-term durability, easy positioning, waterproofing, deformation, stress integrity, and more (fig. 5). They can have a thickness between 0.75 and 2.5 mm, depending on the required abrasion resistance, response to differential settlements, and effective welding required.
Seaming methods for geomembranes include extrusion welding, thermal melting, chemical melting, and adhesive seaming. Tests are conducted to evaluate the properties and performance of geomembranes, including tests for raw polymer properties like density, melting index, and chemical identification, and tests for geomembrane sheet properties like thickness, tensile behavior, puncture resistance, and environmental stress crack.


Fig. 5 - Use of geomembrane for base lining of a landfill site

3.3 Geogrids

Geogrids are a type of geosynthetic material that is composed of a structure with evenly distributed openings between its longitudinal and transverse elements. These openings enhance the interaction between the geogrid and the soil, as the geogrid makes direct contact with both sides of the sheet. The materials used to make geogrids include polypropylene, polyethylene, polyester, and coated polyester. Polyester-coated geogrids are typically woven or knitted, with a PVC or acrylic coating that protects the filaments from construction damage and maintains the grid structure. Polypropylene geogrids are extruded and polyethylene geogrids are exclusively perforated.

Fig. 6 - Geogrid

Geogrids have several physical, mechanical, and durability properties that make them useful in a variety of applications, including soil reinforcement, waterproofing, separation, and stabilization. For example, in landfills, geogrids can be used to support a coating system on a weak substrate, to support the land covering the final landfills on steep slopes, or in the design of landfills built on old landfills. In such cases, a layer of geogrids is necessary to provide a new landfill base, as the highly compressible waste already present would constitute an unstable and poor base.
To characterize geogrids, various test methods are used, including those for other types of geosynthetics. A key parameter is tensile strength, which is typically significantly lower than its ultimate tensile strength. The permissible tensile strength is determined by dividing the final tensile strength by partial factors that consider installation damage, creep deformation, chemical degradation, and biological degradation. The partial factors for installation damage, chemical degradation, and biological degradation vary from 1.0 to 1.6, while the partial safety factor for creep ranges from 1.5 to 3.5. When designing geogrids, it is necessary to consider legislative regulations and apply the appropriate partial factors according to the place of application.

3.4 Geocells

Geocells are versatile three-dimensional panels made of strips that can be used as a flexible structure to provide tensile reinforcement to materials. The panels are typically 5 to 10 cm wide and filled with compacted materials such as gravel, sand, concrete, aggregates, or plant soil. When filled, geocells provide confinement to the fill material, which improves its structural and functional behavior.

Fig. 7 - Geocells

Geocells can be placed directly on the substrate surface and compacted using a hand-operated plate compactor. They are especially useful on soft soils as a stable base for gravity structures with static and dynamic loads. Geocells can also be used for protection and stabilization of slopes and steep surfaces, as well as coating of structures.

One of the main advantages of using geocells is that fillers acquire better properties, including consistent shear strength and stiffness. This is achieved through the confinement voltages effectively induced in a Geocell by the strength of the circle developed by HDPE cell walls. The overall improvement of the system is due to the resistance of the cell wall, the friction between the filling soil and cell walls, and the passive resistance of the filling material in adjacent cells, which allows stress to be distributed on the cells instead of the substrate.

Geocell selection of infills depends on the expected work stresses, the availability and cost of materials, and aesthetic requirements for a fully vegetated appearance. A complete cell confinement system may also include geotextiles, geomembranes, geonets, geogrids, integral polymer tendons, erosion control blankets, and a variety of ground anchorages.

Fig. 8 - Geocells filled with granular soil

3.5 Geonets

Geonets are polymeric materials that serve as a replacement or enhancement of natural materials used for fluid transport. They are often used in place of aggregate drainage systems in landfills and surface seizures, as well as for foundations, retaining walls, and under bridges.
A geonet is a profiled mesh made by extruding two sets of polymer strands to form diamond-shaped openings between adjacent sets of ribs. These geonets are typically made of polyethylene, with a density ranging from 0.935 to 0.942 g/cm^3. The thickness of geonets can vary from 4 mm to 7.5 mm, depending on the extrusion process and whether they have solid or foamed ribs.

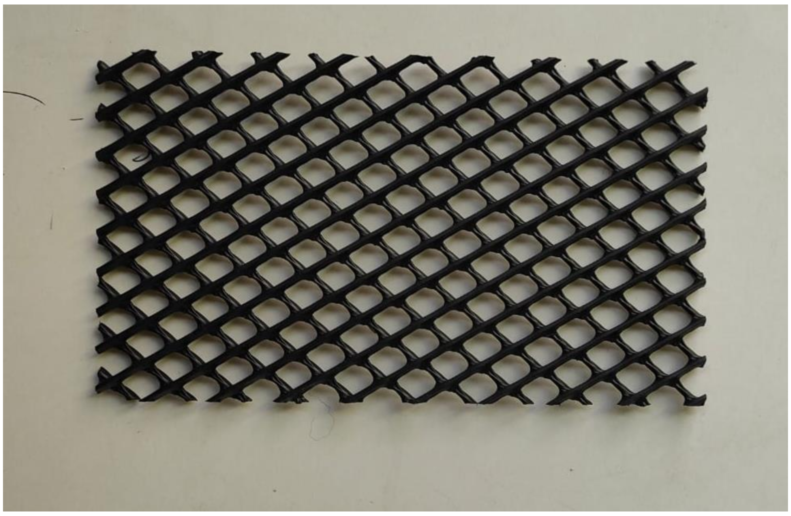


Fig. 9 - Geonet

Geonets are designed to replace or enhance natural materials used in drainage systems, as their three-dimensional structure promotes a planar flow. They are selected based on their flow capacity, which

depends on factors such as normal pressure, field profile, and hydraulic gradient.
Geonets are commonly used in foundations, retaining walls, and landfills, where they collect and drain leachate. When used in slopes or structures, geonet rollers must be tied together through plastic bonds to maintain their functional characteristics or allow for the flow of leachate. The recommended overlap of geonet rollers is 10cm, and plastic clamps should be used every 150cm to bond the rollers. The tensile strength of these plastic bonds is approximately 267N, and the effective strength of the seams must be evaluated to ensure continuity of flow.

3.6 Geomat

A geomat is a type of geosynthetic material that is created through extrusion of polymer, specifically polyester or polyethylene. It takes the form of a grid that can be incorporated into the structure of an extrusion geometry as a reinforcement. The geomat is composed of two parts: the structural part or geogrid, and the geomat itself. The geomat is highly flexible and has a vacuum index of over 90%. It can also be combined with a non-woven material or geotextile on one or both sides. To evaluate and dimension geomats, various tests are conducted according to ISO standards. These tests show that the average thickness of a geomat is 12.45 mm, with a Tensile strength per unit Machine Direction of 3.93 kN/m and per unit Cross Direction of 1.33 kN/m.

Fig. 10 - Geomat

Geomats are commonly used to reinforce road construction and protect against erosion of embankments, sloping earth, waterways, bridge pillars, and torrent beds. The physical interaction between the roots of vegetation and the geomat helps to support the shear forces generated by water erosion and stabilize the underground soil. However, a problem with geomats is their degradation due to exposure to UV radiation present in sunlight. This degradation can occur through UV, thermal, oxidative, and synergistic effects of these mechanisms.

To analyze the exposure of geomats to UV radiation, tests of radioactive aging are conducted in the laboratory to accelerate aging and recreate the conditions that a geosynthetic is exposed to in the field. Samples are compared in their intact form, as well as aged samples in a UV aging chamber and field samples. Thermal analysis is also used to study the physical or chemical properties of materials when subjected to temperature changes related to material decomposition. However, it can be difficult to correlate exposure period in the laboratory to natural (field) weathering, as UV weathering depends on many variables.
A potential issue with geomats is their poor performance in the PC section due to a lack of proper contact between the material and soil surface. Therefore, installation of the product in real situations must be done with high attention to avoid problems due to contact between geosynthetics and soil.

3.7 Bentonite geocomposites

Geocomposite clay coatings, known as GCLs, are hydraulic barriers that consist of a layer of high-quality bentonite clay attached or adhering to geotextiles or geomembranes. They have been used in environmental and geoengineering since 1980 and have since been developed to improve performance and solve a wide range of problems. GCLs are characterized by their low permeability, making them suitable for replacing soils with low permeability coatings or clay. They offer numerous advantages, such as increased resistance to drying and frost-thaw cycles, and can be classified according to their physical properties, such as bentonite type, thickness, coating, and moisture content, as well as their structural formatting.

Fig. 11 - Geocomposite

GCLs are easy to install, take little time, and are used in many environmental and geotechnical applications. They act as a hydraulic barrier in disposal sites, coating, and roofing systems. However, GCLs have some disadvantages, such as the possibility of breakage resulting in loss of bentonite, which can be disadvantageous from both an environmental and performance point of view. The permeability of GCLs is equal to 10^{-11} m/s, making it a potential substitute for natural clay barriers, with different advantages in terms of occupied volume.
Bentonite is responsible for the GCL retention property because it is mainly composed of montmorillonite, a mineral clay with great expansive capacity when hydrated. The hydraulic conductivity of GCL is regulated by the bentonite voids index, and there is an inverse proportionality between the hydraulic conductivity of GCL and the expansion volume of bentonite. The behavior of GCLs depends on the thickness of the adsorbed liquid, and different behaviors can be observed depending on the composition of the permeated

liquid.
Laboratory tests have been conducted to assess GCLs' performance with different types of permeated liquids, such as ethanol, hydrocarbons, and organic solvents. GCLs have better waterproofing performance when fully hydrated, but their hydraulic conductivity increases when the permeated liquid is different from water. Modified bentonites with different quantities and types of polymers are available on the market to achieve different performance targets, which can allow for the use of GCLs in leachate containment, fuel losses, or other contaminated waste. However, modified bentonites can be more economically disadvantageous than natural bentonites.

4. Dimensioning of geosynthetics

The interaction between soil and geosynthetic reinforcement, and between geosynthetic reinforcement and waste mass, is a key aspect for the correct design and dimensioning of landfill components. These interactions are not easy to assess and depend on the nature of the material, the properties of the geosynthetics and the loading conditions. Geosynthetic reinforcements are used to increase the safety factor in landfill stability checks, but at the same time they can cause instability if not designed, sized and applied correctly. In order to gain a precise understanding of their behaviour and to study their interaction with the different materials present in the landfill (soil, waste), a number of laboratory tests were conducted accompanied by theoretical foundations. Different failure mechanisms can occur in a non-cohesive material, such as:

- Material sliding on the geosynthetic surface
- Lateral deformations
- Shear deformations
- Extraction of the geosynthetic

Which can be studied and investigated with laboratory tests such as:

- Direct shear test
- Flat deformation test
- Extraction test
- Ramp test

Laboratory tests have their limitations, linked to the fact that the conditions present in situ cannot be reproduced slavishly (in fact, many tests are carried out on reworked samples), to the boundary conditions and to the confinement linked to the equipment used for the tests. In this chapter, direct shear tests and ramp tests will be discussed in depth.

4.1 Direct shear tests

Direct shear tests are laboratory tests used to obtain the correct dimensioning of geosynthetics by calculating the shear resistance to ensure a stable condition of the cover soil. The strength calculation is performed by evaluating the sliding of the granular material (soil, waste in the case of landfill) on the geosynthetic layer.
The standard equipment consists of a shear box consisting of a lower and upper part, within which the granular material is placed. The geosynthetic, or reinforcement in general, is placed between the lower and upper part. A normal load is applied to the soil sample and a lateral tension that causes one half of the shear box to be displaced relative to the other.
The different tests described in the literature differ due to the different conditions under which they are conducted (mode and sequence of load application, configuration of the internal walls of the shear box). In addition, the way in which the geosynthetic is anchored, the type of geosynthetic and the boundary

conditions of the tests are decisive. Each methodology has advantages and disadvantages that must be taken into account in the final considerations.

Usually, three laboratory tests are carried out by varying only the value of normal tension acting on the sample, so that there is a maximum value, a minimum value and a value between the two of the mechanical characteristics. The latter is taken into account for the study of the parameters, while the other two values are used to establish a confidence interval for the test. The conditions under which the test is carried out, although difficult, must be as similar as possible to the reality to be investigated, and an attempt must be made to reproduce the initial moisture conditions of the material considered.

Direct shear tests with geosynthetic reinforcement at medium height between two soil samples in the box show different results depending on the type of material used.
About the interposition of a geotextile between two soil samples, if the reinforcement is correctly anchored to the shear box, a uniform shear mechanism is appreciated along the reinforcement-granular material interface. The extent of the shear mechanism depends on the granulometric and mechanical characteristics of the materials involved.
In the case of the interposition of a geogrid, the deformations will be distributed between the particles of the granular material between the openings of the geogrid. In general, the distribution of shear stresses will depend on how the geosynthetic is fixed to the test equipment.
It is also very interesting to evaluate the deformation behaviour of the geotextile itself, before considering the interaction between geosynthetic and granular material.
Through the application of a shear test under different conditions (fine sand only, sand - geotextile, geotextile - geotextile), it can be verified how the initial shear displacements are solely dependent on the geosynthetic material and subsequently the interaction with the granular material comes into play. This consideration is relevant for numerical studies of reinforced soil structures as the shear stiffness at the interface is required as an input parameter.
The concept explained above can be observed in fig. 12. The direct shear test was conducted with a non-woven geotextile placed in the middle of the shear box and anchored to a rigid block at the bottom of the equipment (Tupa, 1994).

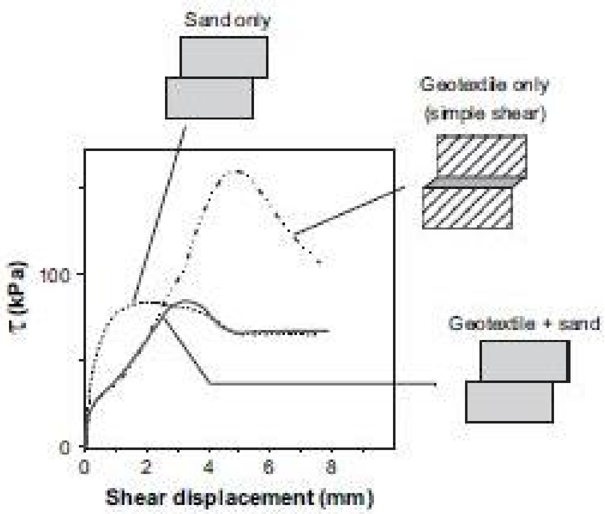


Fig. 12 - Influence of geosynthetic distortion on the results of a direct shear test (Tupa,1994)

It is also possible to interpret results of direct shear tests performed under different conditions, i.e. with the geosynthetic reinforcement crossing the shear plane. This condition can be analysed using photoelastic tests, which show the influence of the presence of the geosynthetic on the tensional state of the material analysed (fig. 13).
In the tests conducted by Dyer, the soil was replaced with glass shatter and the vertical tensional state was

reproduced using a rigid plate. Three different conditions were evaluated, namely the unreinforced and reinforced test with steel grid (to simulate the behaviour of a geogrid) inclined and vertical.
Bright regions indicate areas of high compressive stress, while dim regions indicate areas of low compressive stress. It can be seen how the presence of a reinforcement can change the distribution of the tensional state within the sample.

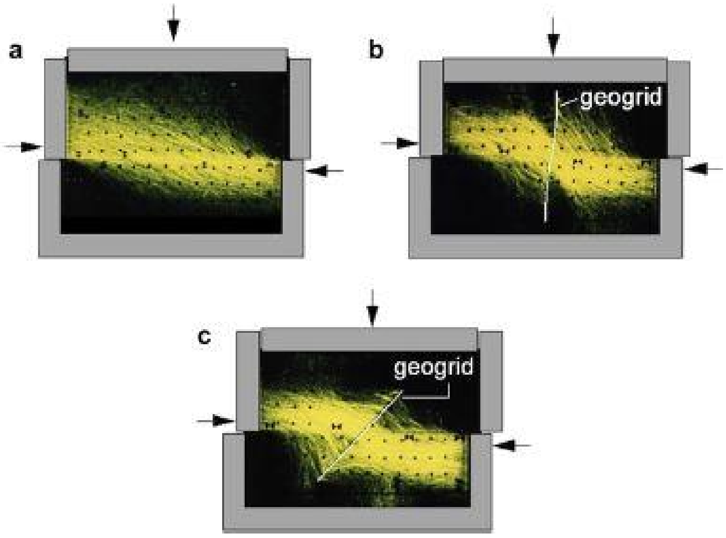


Fig. 13 - Photoelastic interpretation of a direct shear test: (a) Unreinforced; (b) Vertical reinforcement; (c) Inclined reinforcement (Dyer, 1985)

Within a landfill site, direct shear tests are widely used to assess the behaviour of waste over time. However, these tests show obvious limitations due to the impossibility of testing a sample under real, undisturbed conditions. The calculation of shear strength is necessary to assess the stability of the linear components of the landfill, i.e. the geosynthetics.
There is a big difference between soil and waste, so it is considered a mistake to apply results obtained from soil tests to situations where waste of various kinds comes into play. However, it is possible to apply various geotechnical concepts and laboratory tests, with appropriate considerations, to waste.
The first test, conducted in 1991 by Del Greco et al., was conducted by analysing the linear sliding of two bales of waste in a shear box, as in the standard direct shear test. The friction angle value obtained was 32°. From this test, equipment and methods for carrying out other tests have been refined, allowing a more precise evaluation of the shear strength of the waste.

Future tests were also conducted on waste with a different state of compaction in order to assess how the latter affects the shear strength parameter.
The equipment used for these tests consists of a steel frame anchored to a concrete base to contrast the vertical force, while the horizontal restraint consists of a wall. The loads, applied manually, are measured with a pressure gauge. The material contained in the cutting box consists of waste with a volume of $40cm \cdot 50cm \cdot 60cm$ and a weight of 50 kg.
The test, conducted in the same way as the standard direct shear tests, was carried out separately on two waste masses with different compaction states (p_1 = 400 kg/m^3; p_2=600 kg/m^3).
The result of these tests can be evaluated in the figure, where the experimental data and two possible interpolations of them are shown.

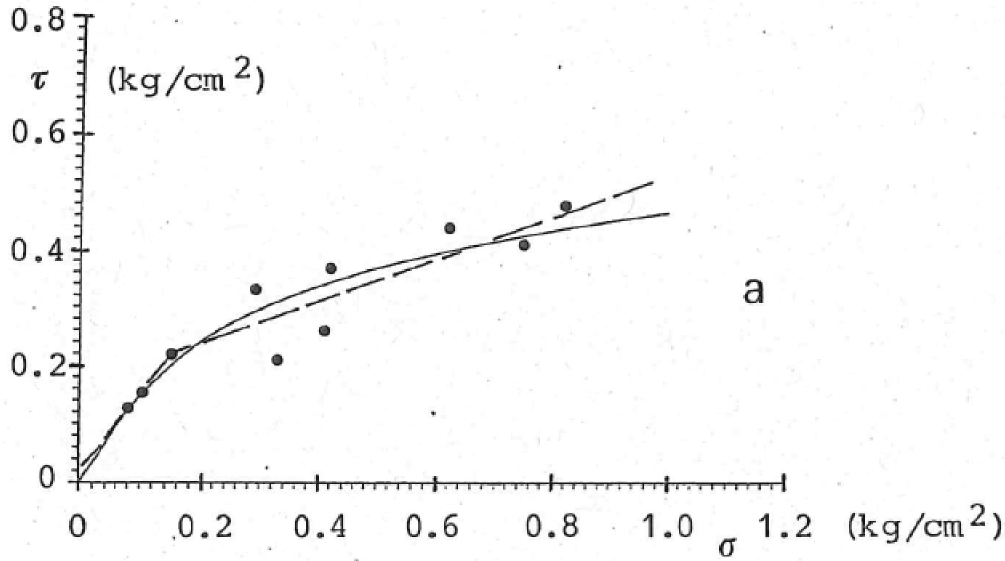


Fig. 14 - σ – τ diagram of tests on low density bales at direct contact (Del Greco and Oggeri, 1993)

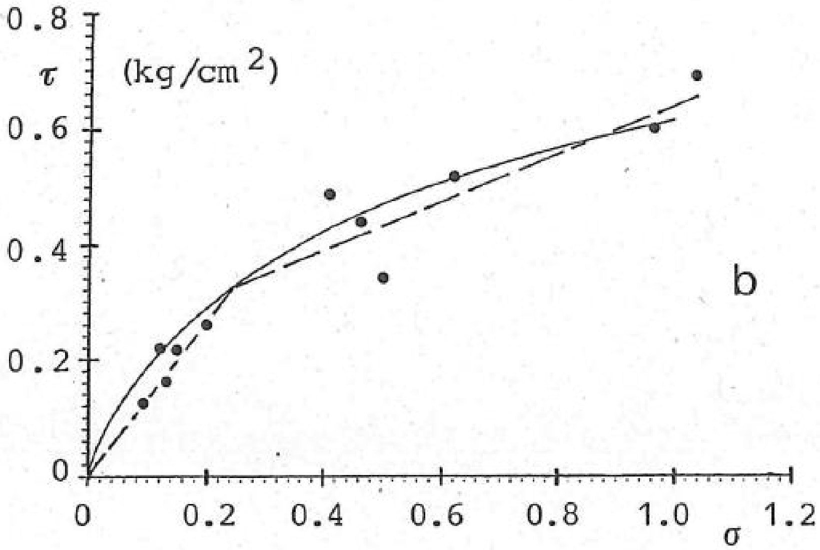


Fig. 15 -– σ – τ diagram of tests on high density bales at direct contact (Del Greco and Oggeri,1993)

Further tests were conducted to evaluate the interaction between waste and geosynthetic reinforcement composed by a HDPE geomembrane (a) and between waste and sandy gravel soil (b).

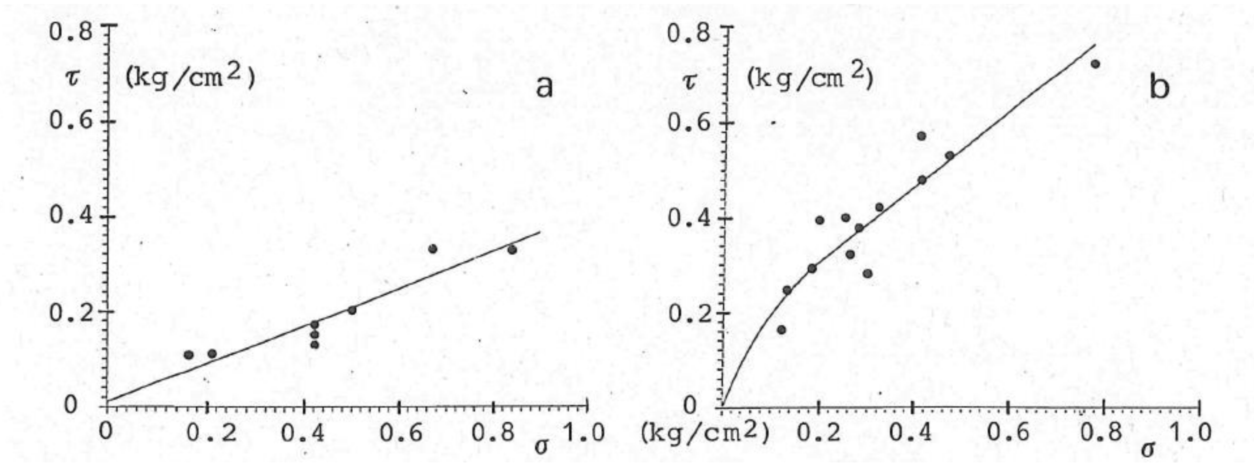


Fig. 16 – σ – τ diagram of different tests: (a) waste – HDPE geomembrane; (b) waste – sandy gravel soil (Del Greco and Oggeri,1993)

The relationship between geomembrane and waste is markedly linear, with an uncertainty related to the less than perfect contact of the surfaces. Linear displacements were always very evident, regular and rapid in these situations.

Another situation analysed was the presence of sandy-gravel soil in contact with the waste, a situation that often occurs in landfills for the purposes of protection, waterproofing and separation. It can be observed that the characteristics of the interposed material influence the shear parameters, since sliding movements often occur in the interposed soil layer.
A summary table is included below to summarise the results obtained under the different test conditions.

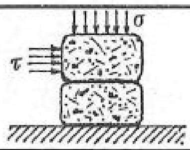
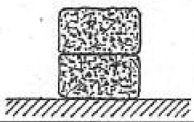
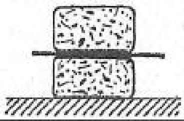
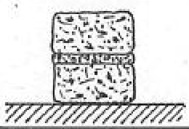
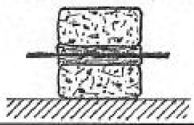
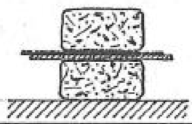
Test types	Test layouts	Shear parameters c (kg/cm2)	φ
Waste-waste coupling ($\gamma = 0.5$ kg/dm3)		0.16	21°
Waste-waste coupling ($\gamma = 0.7$ kg/dm3)		0.24	22°
HDPE geomembrane-waste coupling		0.0	17°
Sandy gravel soil - waste coupling		0.15	38°
HDPEgeomembrane - clay coupling		0.08	26°
HDPE geomembrane - geotextile coupling		0.0	14°

Fig. 17 – Different shear parameters for different test types and layouts (Del Greco and Oggeri,1993)

4.2 Ramp tests

Ramp tests are carried out to assess stability and erosion on slopes due to their ease of execution. They can also be used to assess stability along landfill banks and along cover systems. Given the use of geosynthetic materials within the cover and lining of a landfill site, these tests are useful for investigating the interaction between geosynthetic and soil due to the possibility of simulating moderate tensional states at the interfaces, which are typical of cover soils in waste disposal areas.

Due to the presence of different types of geosynthetic reinforcement in what is called a 'multilayer covering and overburden system', the interface strength of several layers can also be assessed.

The ramp test basically consists of increasing the slope of the ramp until a portion of the soil is slipped over the geosynthetic layer anchored to the surface of the ramp (fig. 18).

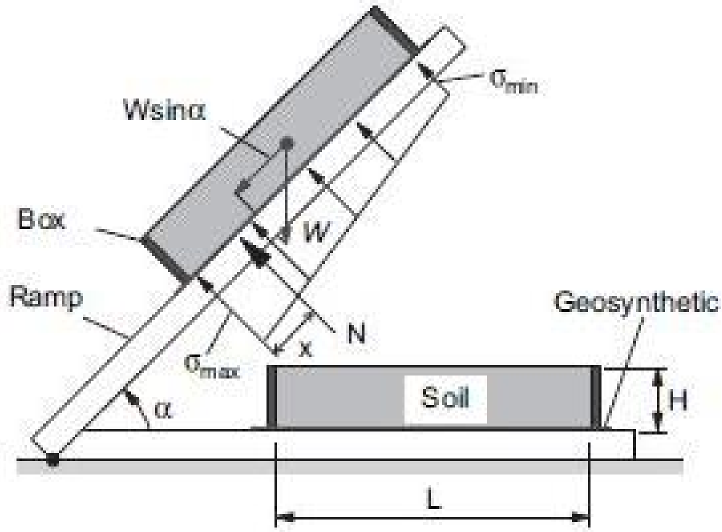


Fig. 18 - Ramp test: conditions of equilibrium (Palmeira et al., 2002)

Through the study of the boundary equilibrium condition and considering as hypothesis that the normal stresses to the geosynthetic layer are distributed according to a trapezoidal conformation, through the equations below it is possible to obtain the maximum and minimum normal stresses.

$$\frac{\sigma_{max}}{\sigma} = 4 - \frac{6x}{L}$$

$$\frac{\sigma_{min}}{\sigma} = \frac{6x}{L} - 2$$

with

$$\frac{x}{L} = \frac{\cos[\alpha + \tan^{-1}(h/L)]}{2\cos\alpha} \cdot \left[1 + \left(\frac{h}{L}\right)^2\right]^{0.5}$$

and

$$\sigma = \frac{W\cos\alpha}{L}$$

Where

- σ is the mean normal stress at the interface,
- σ_{max} *and* σ_{min} are the maximum and minimum normal stresses at the interface;
- x is the distance between the point where the normal force is applied and the edge of the soil block,
- α is the angle of inclination of the ramp calculated from the horizontal direction,
- h and L are the geometric characteristics of the soil block examined,
- W is the weight of the soil

Analysing the results obtained from the equations, the tensional state depends on the geometric characteristics of the soil. The greater the ratio of sample height to sample length, the less uniform the stress distribution will be at the interface. This suggests the use of very long soil boxes in the test equipment.

To assess how the force in a geosynthetic layer is mobilised and evolves, numerical analysis can be used. A ramp test was performed on a sandy soil sample with:

- L = 2 m
- h = 0.23 m

- E = 20 MPa
- v = 0,3
- W = 17 kN/m^3
- Friction angle = 35°.

This sandy soil is placed on a geotextile with res. tensile strength of 70 kN/m and anchored to the ramp.
For the application of the Mohr-Coulomb criterion, the respective angles of friction are:

- Ground-gear friction angle: 35°.
- Ground-geosynthetic friction angle: 31°.
- Geosynthetic-geosynthetic friction angle: 26°.

The shear resistance values are:

- Soil-geosynthetic-shear resistance: 3000 kN/m^3
- Geosynthetic-shear resistance: 25000 kN/m^3

The execution of the test shows that an increase in the inclination of the ramp leads to a mobilisation of the tensile force on the geosynthetic reinforcement, which is stretched along its full length only when the alpha angle reaches the critical value of 31°.

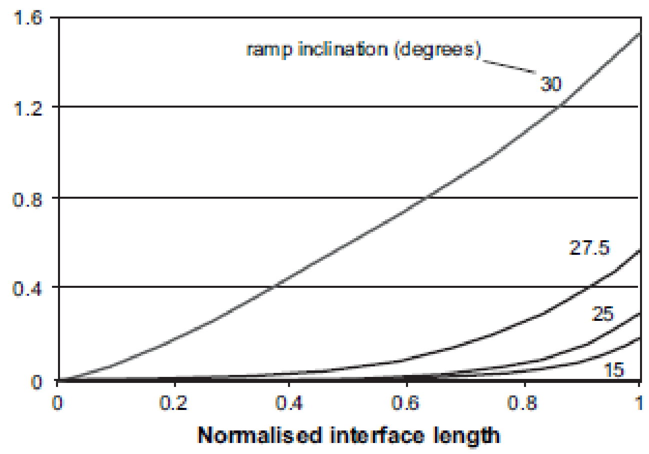


Fig. 19 – Mobilisation of the tensile force on the geosynthetic reinforcement in a ramp test (Palmeira et al., 2002)

To decrease the magnitude of tensile forces in a geomembrane on a slope of a landfill site, one solution could be the use of reinforcement in the cover soil, since the use directly on the geomembrane, although simpler, is not the most efficient solution.
The goodness of this solution can be assessed by means of a ramp test.
In this case, the reinforcement consists of a geogrid placed inside the cover soil. Both the geomembrane and the geogrid are anchored to a rigid frame and the tensile forces developed during the execution of the test can be measured (fig. 20)

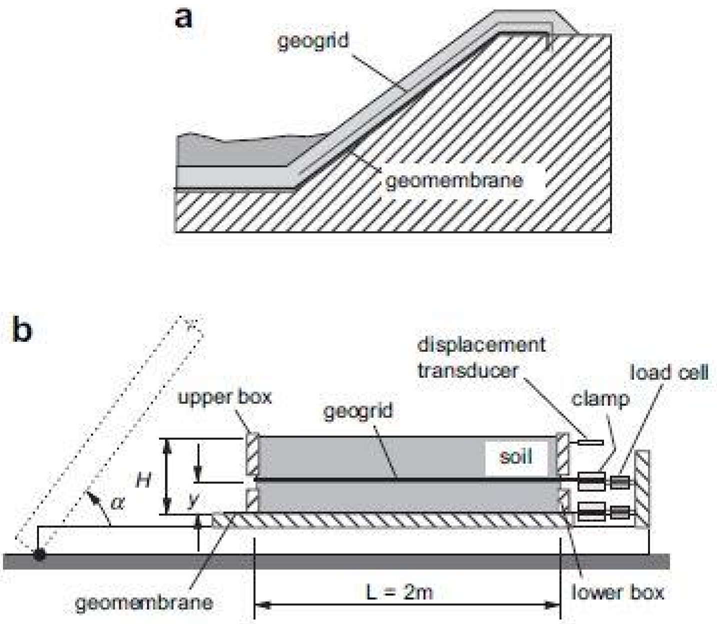


Fig. 20 – Reinforcement of cover soils of landfills: (a) Situation in site; (b) Ramp test equipment. (Palmeira and Viana, 2003)

.
Other cases were evaluated to gain a better understanding of how the situation may change depending on the possible presence and position of the reinforcement. The cases evaluated are:

- Roofing soil without reinforcement above a geomembrane
- Cover soil with reinforcement (geogrid) at medium height (y/H = 0.5)
- Cover soil with reinforcement (geogrid) over a geomembrane protected by geotextile

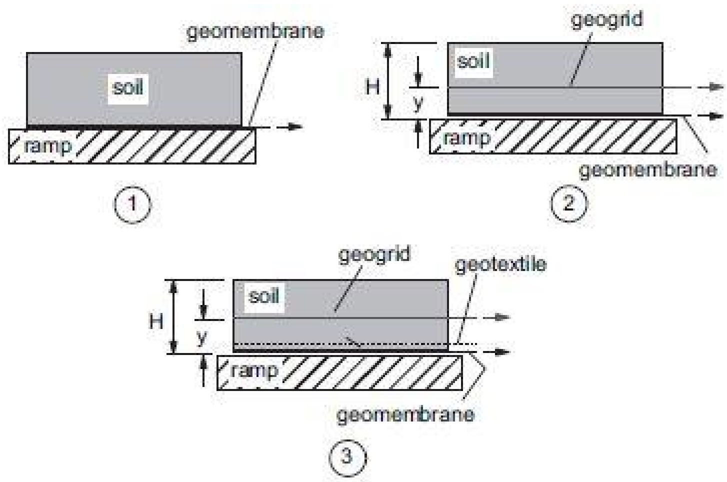


Fig. 21 – Different types of soil reinforcement (Palmeira and Viana, 2003)

The latter configuration was designed in order to decrease the mechanical damage of the geomembrane. The geogrid has a mesh with

- 20mm x 20mm openings
- thickness of 1.1 mm
- tensile strength value of 200 kN/m.

The non-woven geotextile, on the other hand, is made of polypropylene with a mass per unit area of 200 g/m^2. The geomembrane is made of HDPE with a smooth surface. The cover soil consists of a uniform sandy granular soil.

The application of the test made it possible to evaluate the different inclination angles of the ramp under different test conditions. Under unreinforced conditions, the inclination of the ramp for which the ground is slipping is 26°. This value is increased up to 34° in the case of reinforcement with only geogrid. The presence of geotextile allows a slight increase in the inclination of the ramp and provides a less deformable system, as the box displacements begin when the inclination angle reaches the value of 28°. This shows how the presence of reinforcement in a cover soil is essential in order to be able to have more inclined banks and to have more useful volume to use for waste storage.

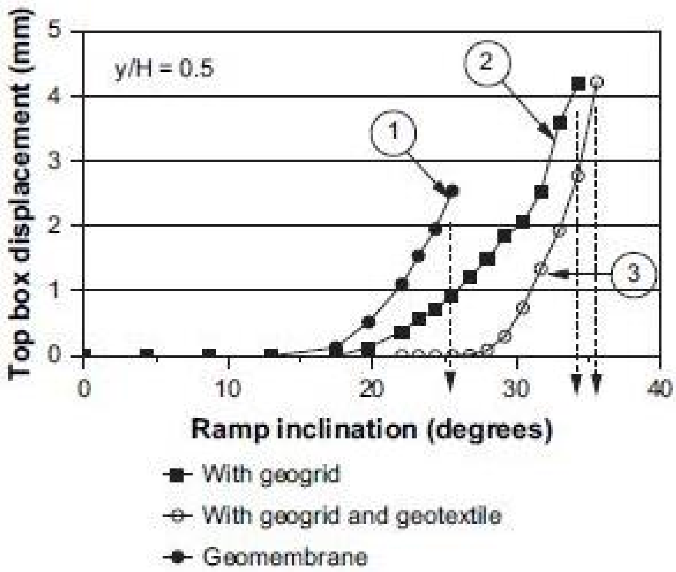


Fig. 22 – Effect of the presence of reinforcement on cover soil stability (Palmeira and Viana, 2003)

Analyses were also conducted to see the evolution of the system as the y/H ratio changes, for example as the position of the reinforcement within the cover soil changes.

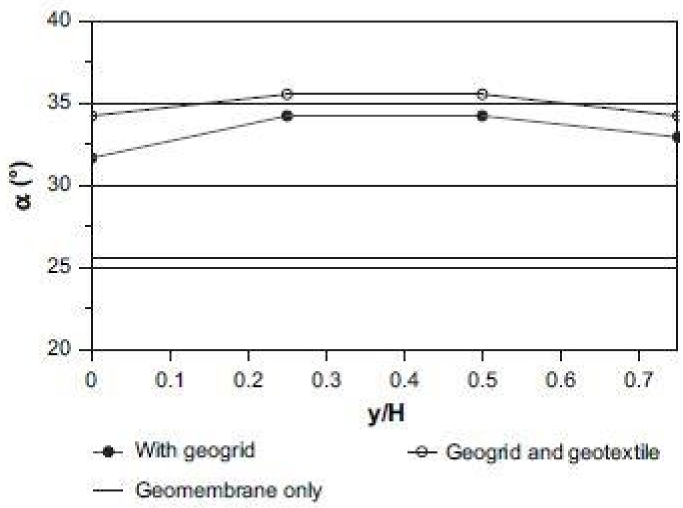


Fig. 23 – Influence of the ratio y/H on the cover soil stabiloty (Palmeira and Viana, 2003)

It can be seen that the maximum efficiency is obtained for ratios between 0.3 and 0.5, and that the system of reinforced soil and geotextile to protect the geomembrane is the configuration that brings the greatest benefit to the stability of the system.

5. Mechanisms of interaction and interfaces

The subsidence of soils constituting the lining of a landfill usually occurs along a linear surface. The interface affected by displacement is the one with the lowest interface friction angle among those considered on the same cross-section. Given the linear geometry of the rupture mechanism, stability calculations are simpler because there is no need to know radius and test centres and because there is no need to solve simultaneous equations or to consider simplifying design assumptions.

5.1 Boundary equilibrium concepts

The free body diagram of an infinitely long slope with homogeneously thick cohesionless overburden on an incipient planar shear surface, such as the top surface of a geomembrane, is shown in the following figure.

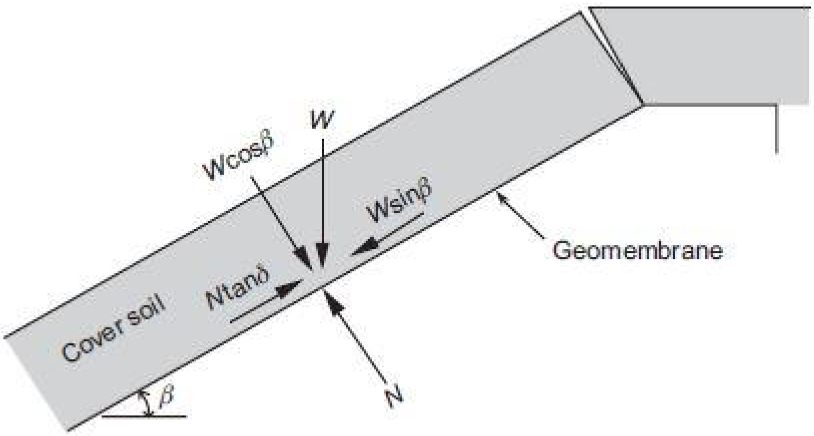


Fig. 24 – Forces involved in an infinite slope analysis for a cover soil (Koerner and Soong, 2005)

The slope safety factor is calculated as the ratio of the summation of the components parallel to the slope of the resisting forces to the summation of the components parallel to the slope of the stabilising forces.
It results in:

$$FS = \frac{\sum Resisting\ forces}{\sum Driving\ forces} = \frac{N \cdot \tan\delta}{W \cdot \sin\beta} = \frac{W \cdot \cos\beta \cdot \tan\delta}{W \cdot \sin\beta}$$

Which simplifies to:

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

The factor of safety therefore simply reduces to a ratio between the tangent of the interface friction angle of the cover soil to the upper surface of the geomembrane and the tangent of the slope angle of the soil beneath the geomembrane.

5.2 Situations causing instability

The situation previously described is simplistic, however, as it does not take into account other conditions that may be encountered in the design phase of landfills, such as:

- Presence of equipment loads on the slope;
- Presence of infiltration forces within the cover soil;
- Presence of seismic forces acting on the cover soil;

Furthermore, the assumption of an indefinitely long slope is not particularly true in landfill contexts.
The specific situations listed above will be dealt with in the following paragraphs first at a theoretical level and then a numerical example and design graph will be presented.

5.2.1 Cover soil gravitational force on a slope of finite length

As the first condition illustrated, the situation of a cover soil of finite length and uniform thickness placed on a slope with inclination β was analysed. The division of the soil into an active and a passive part is useful for the calculation of the factor of safety in the slope stability assessment. It is possible to note the presence of a tension crack and a passive wedge on the toe. The scenario was analysed by applying the equations of Koerner and Hwu (1991), which were compared with the equations of Giroud and Beech (1989), McKelvey and Deutsch (1991) and others. The scenario analysed in this section is illustrated in the figure.

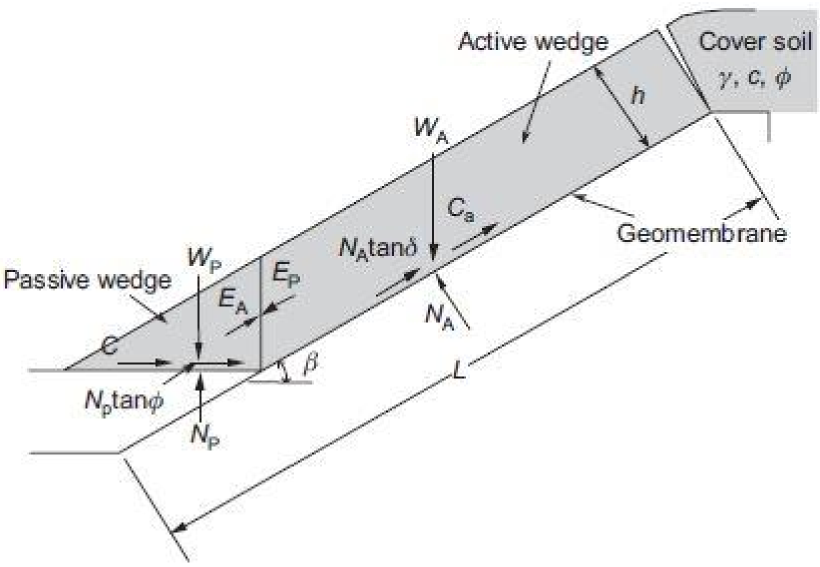


Fig. 25 – Limit equilibrium forces involved in a finite length slope of cover soil (Koerner and Soong, 2005)

With:
- W_A is the weight of the active wedge;
- W_P is the weight of the passive wedge;
- N_A is the force perpendicular to the active wedge at the interface ;
- N_P is the force perpendicular to the passive wedge on the interface;
- γ is the unit weight of the overburden;
- h is the thickness of the cover soil;
- L is the slope length of the geomembrane;
- β is the slope angle of the soil under the geomembrane;
- ϕ is the angle of friction of the cover soil;
- δ is the angle of interface friction between the cover soil and the geomembrane;
- C_A is the force of attraction between the active wedge cover soil and the geomembrane;
- c_a is the adhesion between the active wedge cover soil and the geomembrane;
- C is cohesive force along the failure plane of the passive wedge;
- c is the cohesion of the cover soil;
- E_A is internal reaction that the passive wedge exerts on the active wedge;
- E_P is internal reaction that the active wedge exerts on the passive wedge;
- FS is creep safety factor of the cover soil on the geomembrane.

Before determining the equations to derive the factor of safety, the equations necessary to describe the forces acting on the active wedge and the passive wedge are given. On the active wedge, the acting forces are.

$$W_A = \gamma h^2 \left(\frac{L}{H} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right)$$

$$N_A = W_A \cos\beta$$

$$C_a = c_a \left(L - \frac{h}{\sin\beta}\right)$$

By balancing the forces in the vertical direction, the internal reaction formulation E_A can be obtained:

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta\,(FS)}$$

On the passive wedge, the forces to be considered are:

$$W_P = \frac{\gamma h^2}{\sin 2\beta}$$

$$N_P = E_P \sin\beta + W_P$$

$$C = \frac{ch}{\sin\beta}$$

By balancing the forces in the horizontal direction, the internal reaction formulation E_P can be obtained:

$$E_P = \frac{C + W_P\tan\phi}{\cos\beta\,(FS) - \sin\beta\tan\phi}$$

Following simplifying assumptions, as considering $E_A = E_P$, and balancing the forces in the vertical and horizontal directions, an equation of the following second-degree factor of safety can be obtained:

$$a(FS)^2 + b(FS) + c = 0$$

$$FS = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

The parameters a, b and c are a function of the slope characteristics and the forces involved:

$$a = (W_A - N_A\cos\beta)\cos\beta$$

$$b = -[(W_A - N_A\cos\beta)\sin\beta\tan\phi + (N_A\tan\delta + C_a)\sin\beta\cos\beta + \sin\beta\,(C + W_P\tan\phi)]$$

$$c = (N_A\tan\delta + C_a)\sin\beta\sin\beta\tan\phi$$

The critical safety factor, which represents the limiting condition for which stability is guaranteed, is equal to 1. This means that the safety factor must have a value greater than 1. The precise value is decided at the design stage based on site-specific conditions and other assessments. Depending on the various angles of inclination and the angle of friction of the interface, and with the various geometric and mechanical characteristics given in the legend, the development of the factor of safety is shown.

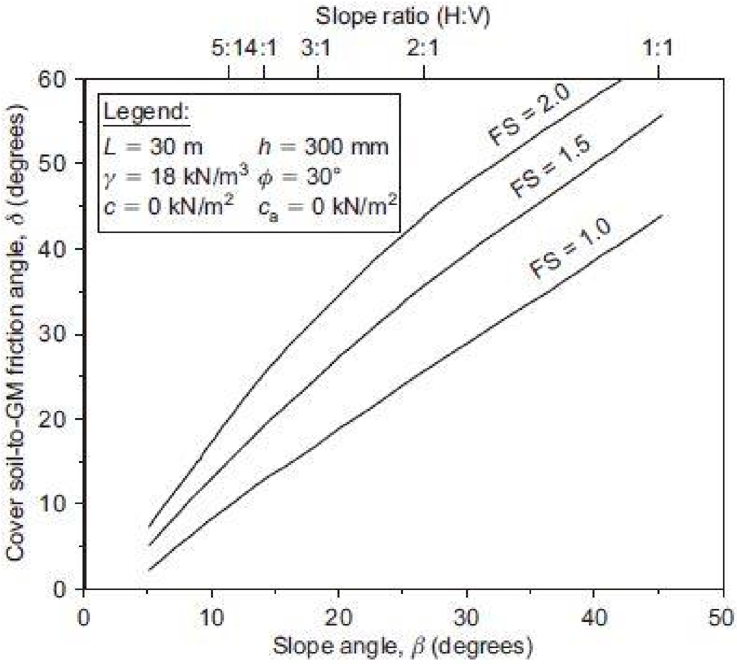


Fig. 26 – Relationship between slope angle and cover soil-to-GM angle for different safety factors (Koerner and Soong, 2005)

5.2.2. Forces of construction equipment

The second condition analysed considers the presence of machines on the slope. This is a very recurring situation in landfills, for example, where equipment is used for levelling waste, for laying geosynthetics, and for backfilling other material. The recommended method of laying topsoil is shown in the figure. The movement from the base towards the apex of the slope allows the development of forces in a direction parallel to the slope, consisting of a geomembrane, but with resistant action. The resistant action consists of soil compaction as gravitative action opposes displacement, and results in the stability of the active and passive wedges.

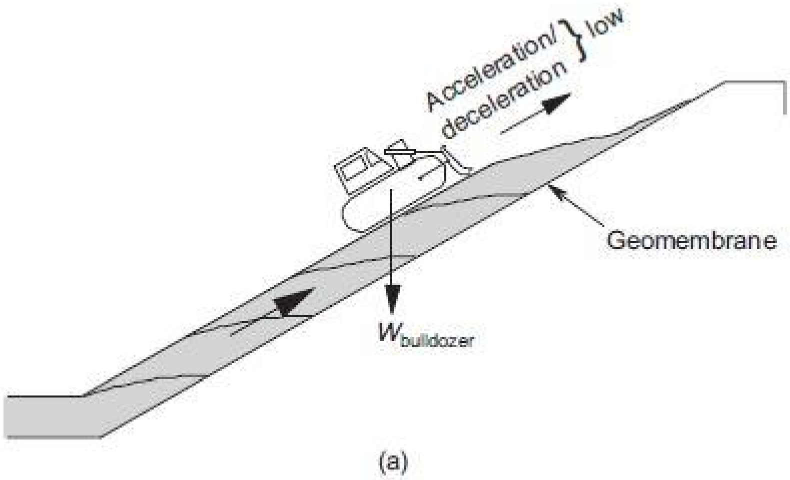


Fig. 27 – Influence of equipment backfilling up cover soil slope reinforced with geosynthetic (Koerner and Soong, 2005)

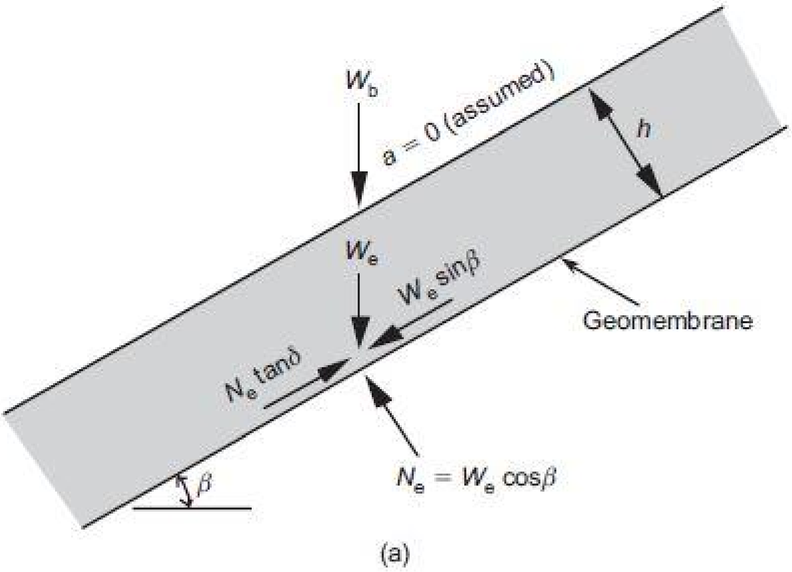


Fig. 28 – Balance of forces with equipment backfilling up slope (Koerner and Soong, 2005)

The free body diagram depicted in the fig.28 shows the distribution of forces in the first scenario presented, while in the fig.30 the second scenario is represented.

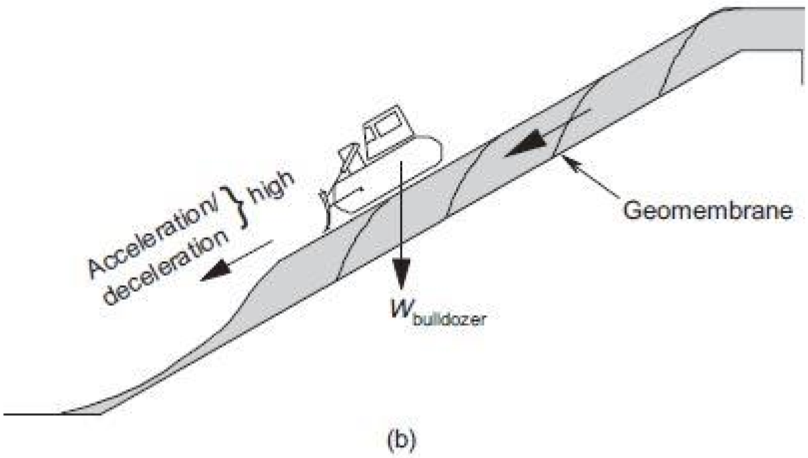


Fig. 29 - Influence of equipment backfilling down cover soil slope reinforced with geosynthetic (Koerner and Soong, 2005)

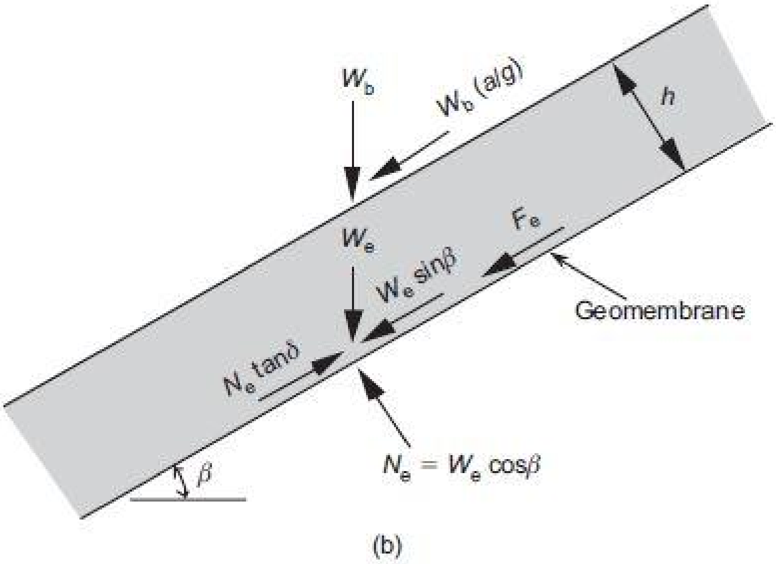


Fig. 30 - Balance of forces with equipment backfilling down slope (Koerner and Soong, 2005)

In this case, the machine exerts a vertical tension due to its own weight and a parallel tension (F_e) to the geosynthetic reinforcement related to the acceleration of the machine, the acceleration of gravity and the W_e force.
The F_e component can be written as:

$$F_e = W_e \frac{a}{g}$$

The analysis proceeds by applying the same method as used in paragraph 5.2.1 The contribution of the machine used is calculated using the Boussinesq theory according to the following equation:

$$W_e = q \cdot w \cdot I = \frac{W_b}{2 \cdot w \cdot b} \cdot w \cdot I$$

Where:

- W_e is equivalent to the machine force per unit width at the geomembrane interface;
- q = load applied by the machine
- W_b is the effective weight of the equipment (e.g. a bulldozer);
- w is the track length of the equipment.
- b is the track width of the equipment;
- I is the influence factor at the geomembrane interface.

The value "I" can be deduced from fig. 31 , which depicts the values of I useful for dissipating the surface force from the cover soil to the interface at which the geomembrane is located.

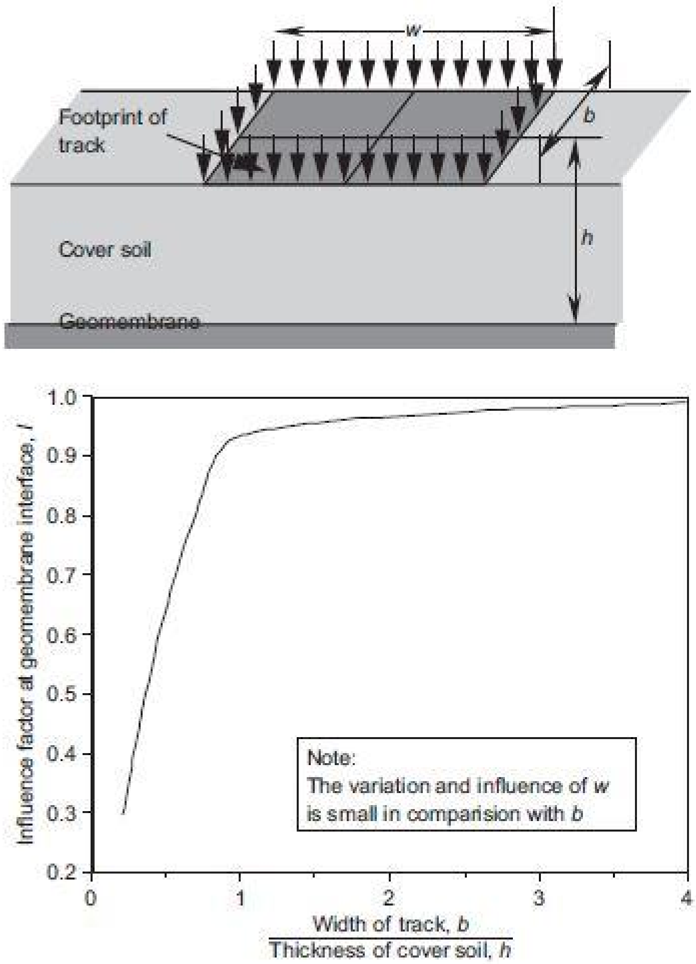


Fig. 31 – Graph to state the influence factor at the geomembrane interface (Poulos and Davis, 1974)

In practice, the movement of the equipment is represented by the term W_E, which generates a resistant action. The treatment thus carried out does not take acceleration forces into account. The parameters of the second-degree equation required for the calculation of the safety factor, after taking into account the balances of forces in the horizontal and vertical directions and after considering the same simplifying assumption regarding internal reactions, are given here:

$$a = [(W_A + W_e)\sin\beta + F_e]\cos\beta$$
$$b = -\{[(N_e + N_A)\tan\delta + C_a]\cos\beta + [(W_A + W_e)\sin\beta + F_e]\sin\beta\tan\phi + (C + W_p\tan\phi)\}$$
$$c = [(N_e + N_A)\tan\delta + C_a]\sin\beta\tan\phi$$

5.2.3. Consideration of infiltration forces

The study of the configuration in which the seepage forces are considered involves a little further investigation of the characteristics of the soil. In the previous analyses, the soil was always considered in such a way as to be able to effectively evacuate the water present. This mechanism can be achieved by a suitable drainage layer or a material with a certain permeability. The amount of water to be removed is a condition to be assessed for each site and under different climatic, morphological, etc. conditions.
When these conditions are not met, i.e. there is:

- Presence of soils with inadequate hydraulic permeability to ensure drainage;
- Inadequate drainage capacity due to geometric conditions of the slope (very long slopes)
- Obstruction of the drainage layer by fine particles
- Freezing and subsequent melting of ice, mobilising infiltration forces

Infiltration forces are generated, which leads to a change in the diagram of the forces at play on the slope in question.

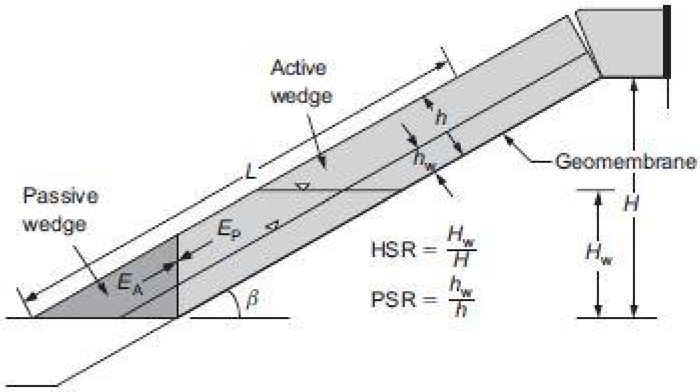


Fig. 32 – Influence of infiltration forces on a cover soil slope (Koerner and Soong, 1995)

Unlike the examples given in the previous paragraphs, in this situation a portion of the soil, of thickness h_w, is in a saturated condition, while the remainder is in an unsaturated condition. This results in the presence of hydrostatic forces and the need to consider the saturated weight parameter of the cover soil. The factor of safety for two different situations, i.e. with horizontal seepage build-up and for parallel to slope seepage build-up, always assumes the same formulation (after imposing the horizontal and vertical balance of forces, and considering $E_A = E_P$ in this case as well). It is therefore necessary to calculate the coefficients a, b and c as follows:

$$a = W_A \sin\beta \cos\beta - U_h \cos\beta \cos\beta + U_h$$
$$b = -W_A \sin\beta \sin\beta \tan\phi + U_h \sin\beta \cos\beta \tan\phi - N_A \cos\beta \tan\delta - (W_P - U_V) \tan\phi$$
$$c = N_A \sin\beta \tan\delta \tan\phi$$

The two cases in which the formulas corresponding to the terms given in the previous equations will be differentiated below.

Horizontal seepage build-up

The force diagram in the horizontal seepage condition can be seen in the figure 33.

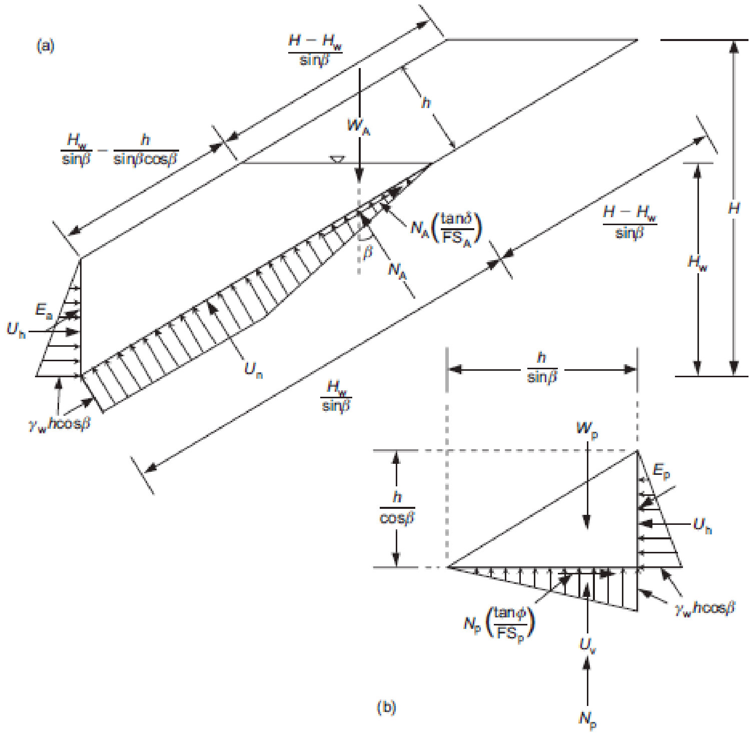


Fig. 33 - Diagram of forces involved in cover soil with horizontal seepage build-up: (a) active wedge; (b) passive wedge (Koerner and Soong, 2005)

As stated earlier, the presence of water in the soil and thus the occurrence of hydrostatic pressures must be considered at this time. In the diagram showing the distribution of hydrostatic pressures there are the following components:

- H is the vertical height of the slope measured from the foot of the slope
- H_W is the height of the free surface of the water
- U_h is the interstitial pressure exerted on the surface between the two wedges
- U_n is the interstitial pressure normal to the slope interface
- U_v is the resultant interstitial pressure acting on the passive wedge
- γ_{sat} is the density weight of the soil under saturated conditions
- γ_{dry} is the density weight of the soil under unsaturated conditions
- γ_w is the density weight of water

The useful expressions for calculating the forces are:

$$W_A = \frac{\gamma'_{sat\ d}(h)(2H_w \cos\beta - h)}{\sin 2\beta} + \frac{\gamma_{dry}(h)(H - H_w)}{\sin\beta}$$

$$U_n = \frac{\gamma_w(h)(\cos 2\beta)(2H_w \cos\beta - h)}{\sin 2\beta}$$

$$U_h = \frac{\gamma_w h^2}{2}$$

$$W_P = \frac{\gamma'_{sat\ d} h^2}{\sin 2\beta}$$

$$U_v = U_h \cot\beta$$

Parallel to slope seepage build up

In the figure 34, the force diagram in the "Parallel to slope seepage" condition can be observed.

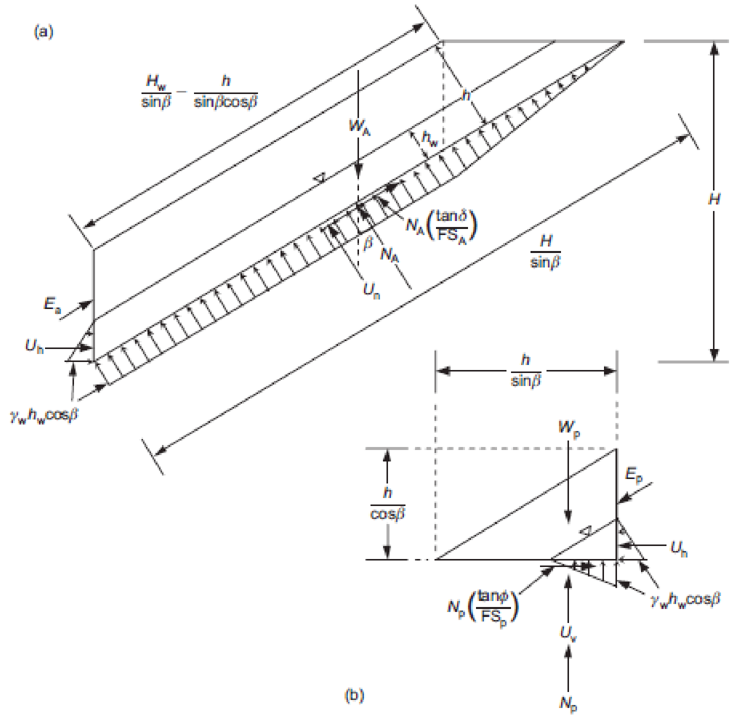


Fig. 34 - Diagram of forces involved in cover soil with parallel seepage build-up: (a) active wedge; (b) passive wedge (Koerner and Soong, 2005)

The useful expressions for calculating the forces are:

$$W_A = \frac{\gamma_{dry}(h-h_w)[2H\cos\beta-(h+h_w)]}{\sin 2\beta} + \frac{\gamma'_{sat\,d} h_w(2H\cos\beta-h_w)}{\sin 2\beta}$$

$$U_n = \frac{\gamma_w h_w \cos\beta\,(2H\cos\beta-h_w)}{\sin 2\beta}$$

$$U_h = \frac{\gamma_w h_w^2}{2}$$

$$W_P = \frac{\gamma_{dry}(h^2-h_w^2)+\gamma'_{sat\,d} h_w^2}{\sin 2\beta}$$

5.2.4. Consideration of seismic forces

In some areas it is necessary to consider the contribution of seismic forces. The process for calculating these forces consists of two parts:

- Through a pseudo static analysis, a horizontal force acting on the centroid of the cover soil cross-section is added and the FS is calculated.
- If the FS calculated in the previous step is less than 1.0, a deformation analysis is carried out, which may cause damage to the cover soil cross-section. This situation can either be accepted or a modification of the slope can be designed in order to achieve the stability condition.

The first point corresponds to the same approach used in the previous paragraphs. In this point, the contribution of the horizontal force applied to the centroid of the cover soil is added. To consider this contribution, it is necessary to introduce the average seismic coefficient, equal to the ratio between the acceleration in the rock substrate and the acceleration of gravity, obtained from maps depicting seismic activity in the area in question.

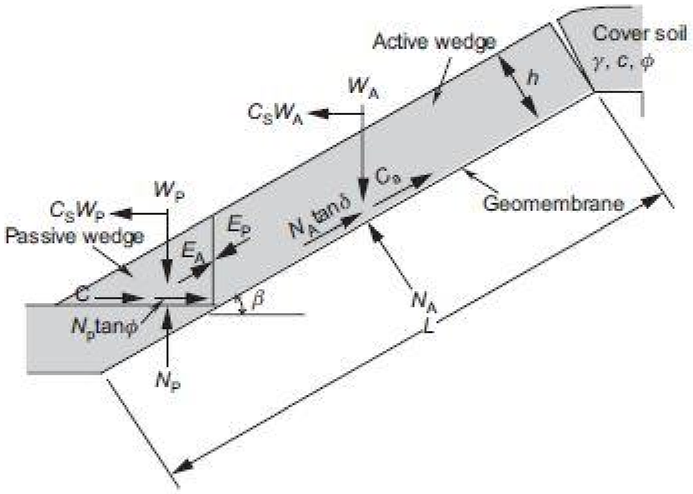


Fig. 35 – Influence of seismic actions on a cover soil slope (Koerner and Soong, 2005)

By balancing the horizontal and vertical forces shown in figure 35, and considering the internal reactions to be equal and opposite, the coefficients of the second-degree equation with the unknown factor of safety are calculated as follows:

$$a = (C_S W_A + N_A \sin\beta)\cos\beta + C_S W_P \beta$$
$$b = -[(C_S W_A + N_A \sin\beta)\sin\beta\tan\phi] + (N_A \tan\delta + C_a)(\cos\beta)^2 + (C + W_P \tan\phi)\cos\beta$$
$$c = (N_A \tan\delta + C_a)\cos\beta\sin\beta\tan\phi$$

It is also possible to draw a design curve in a general situation, based on the data given in the legend, as a function of the seismic coefficient.

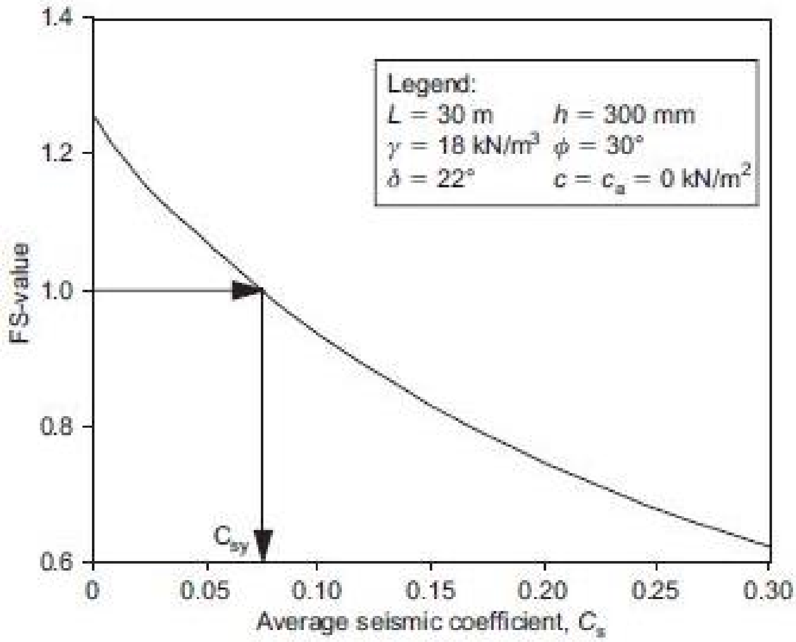


Fig. 36 – Relationship between the average seismic coefficient and the safety factor (Koerner and Soong, 2005)

5.3 Situations leading to slope stabilization

Having illustrated the possible situations that can cause instability in a slope with overburden on top of a geosynthetic reinforcement (geomembrane in this case), in this section we will present actions that allow the slope's stability to be increased. The categories to which these actions belong are:

- Modification of the cover soil and slope geometry
- The use of geosynthetic reinforcements within the cover soil (reinforcement of the overburden)

The choice of one method over another depends on the assessments made by the designer.

5.3.1 Toe berm (buttress)

The placement of a soil mass at the base of the slope is a widely used technique, for example in motorway construction and earthworks. This added piece of soil makes it possible to increase the volume of the passive layer, which exerts a resistant force with respect to the active wedge. The calculation of the safety factor is carried out by means of static analysis, as shown in the previous cases. Two situations in which this technique is applied are illustrated in the following figures.

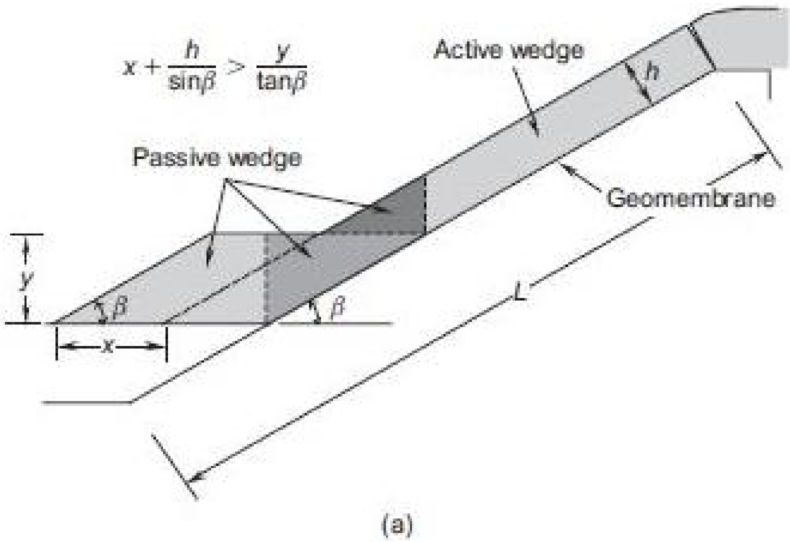


Fig. 37 – First scenario with toe berms used to increase stability (Koerner and Soong, 2005)

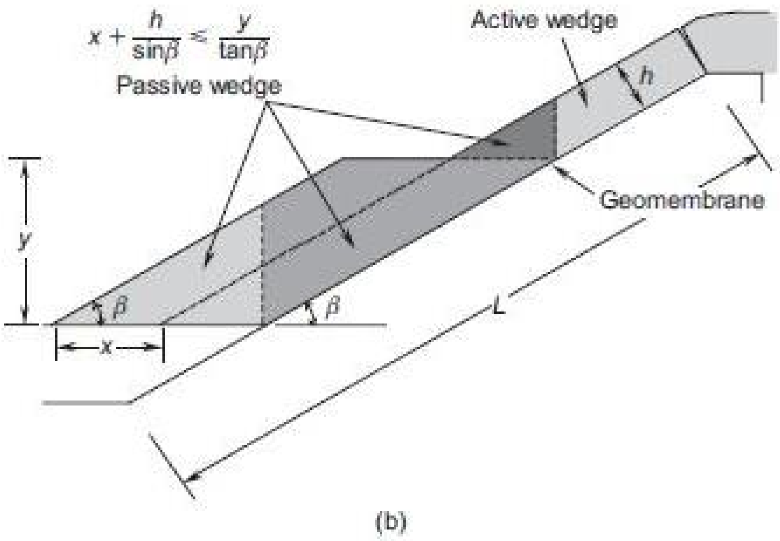


Fig. 38 - Second scenario with toe berms used to increase stability (Koerner and Soong, 2005)

5.3.2. Slopes with tapered thickness cover soil

An alternative method for increasing slope stability is the design of a cover soil with a tapered surface. In this configuration, the thickness of the cover soil increases from the crest of the slope to the foot of the slope in a manner proportional to the thickness at the base of the slope, maintaining a constant inclination equal to ω, with ω < β. The force diagram is depicted in the figure 39.

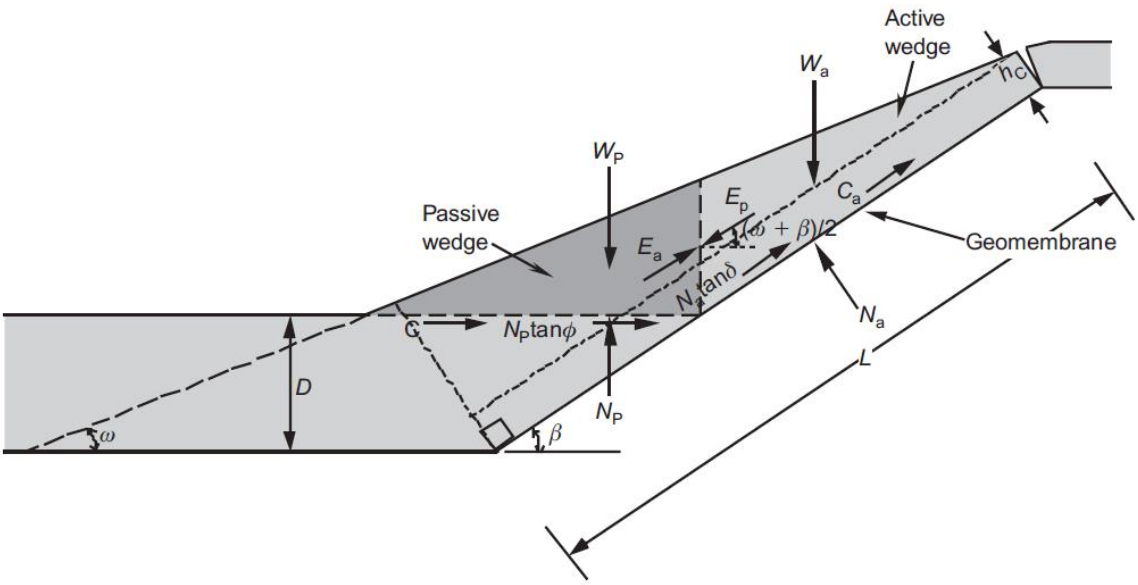


Fig. 39 – Influence of tapered thickness cover soil from toe to crest (Koerner and Soong, 2005)

The terms in this case are the same as in the previous cases, with the addition of:

- D is the thickness of the topsoil at the base of the slope
- H_c is the thickness of the topsoil at the crest of the slope
- ω is the angle of inclination of the top of the cover soil

The variable describing the slope of the top of the cover soil is y, expressed as follows

$$y = \left(L - \frac{D}{\sin\beta} - h_c \tan\beta\right)(\sin\beta - \cos\beta\tan\omega)$$

The forces acting on the active wedge are schematised as follows:

$$W_A = \gamma\left[\left(L - \frac{D}{\sin\beta} - h_c \tan\beta\right)\left(\frac{y\cos\beta}{2} + h_c\right) + \frac{h_c^2 \tan\beta}{2}\right]$$
$$N_A = W_A \cos\beta$$
$$C_a = c_a\left(L - \frac{D}{\sin\beta}\right)$$

The forces affecting the passive wedge, on the other hand, can be schematised as follows:

$$W_P = \frac{\gamma}{2\tan\omega}\left[\left(L - \frac{D}{\sin\beta} - h_c \tan\beta\right)(\sin\beta - \cos\beta\tan\omega) + \frac{h_c}{\cos\beta}\right]^2$$
$$N_P = W_P + E_P \sin\left(\frac{\beta + \omega}{2}\right)$$
$$C = \frac{\gamma}{\tan\omega}\left[\left(L - \frac{D}{\sin\beta} - h_c \tan\beta\right)(\sin\beta - \cos\beta\tan\omega) + \frac{h_c}{\cos\beta}\right]$$

Through the horizontal and vertical balance of forces, and using the same simplifying assumption as in the previous cases, the coefficients of the second-degree equation for calculating the factor of safety are expressed as:

$$a = (W_A - N_A \cos\beta)\cos\left(\frac{\beta+\omega}{2}\right)$$

$$b = -\left[(W_A - N_A \cos\beta)\sin\left(\frac{\beta+\omega}{2}\right)\tan\phi + (N_A \tan\delta + C_a)\sin\beta\cos\left(\frac{\beta+\omega}{2}\right) + \sin\left(\frac{\beta+\omega}{2}\right)(C + W_P \tan\phi)\right]$$

$$c = (N_A \tan\delta + C_a)\sin\beta\sin\left(\frac{\beta+\omega}{2}\right)\tan\phi$$

5.3.3. Reinforcement of the veneer: intentional

The other method of increasing the stability of a slope with overburden is to place a geosynthetic material within the soil itself. The reinforcement can be:

- Intentional: the use of a geosynthetic is linked to the intention of increasing the system's resistance against instability. The development of the factor of safety also depends on the type and material of the reinforcement.
- Unintentional: a very common situation in multi-layer lining systems (such as landfill covers). In this case, a geosynthetic can increase the overall strength of the system even if the interface with the lowest shear resistance is located directly under the geosynthetic itself and even if the geosynthetic is not directly designed for an increase in the strength of the system. The designer cannot predict this scenario a priori.

The stability analysis does not change in form. The difference between the two cases lies in the FS value, which increases substantially in the case of intentional reinforcement.

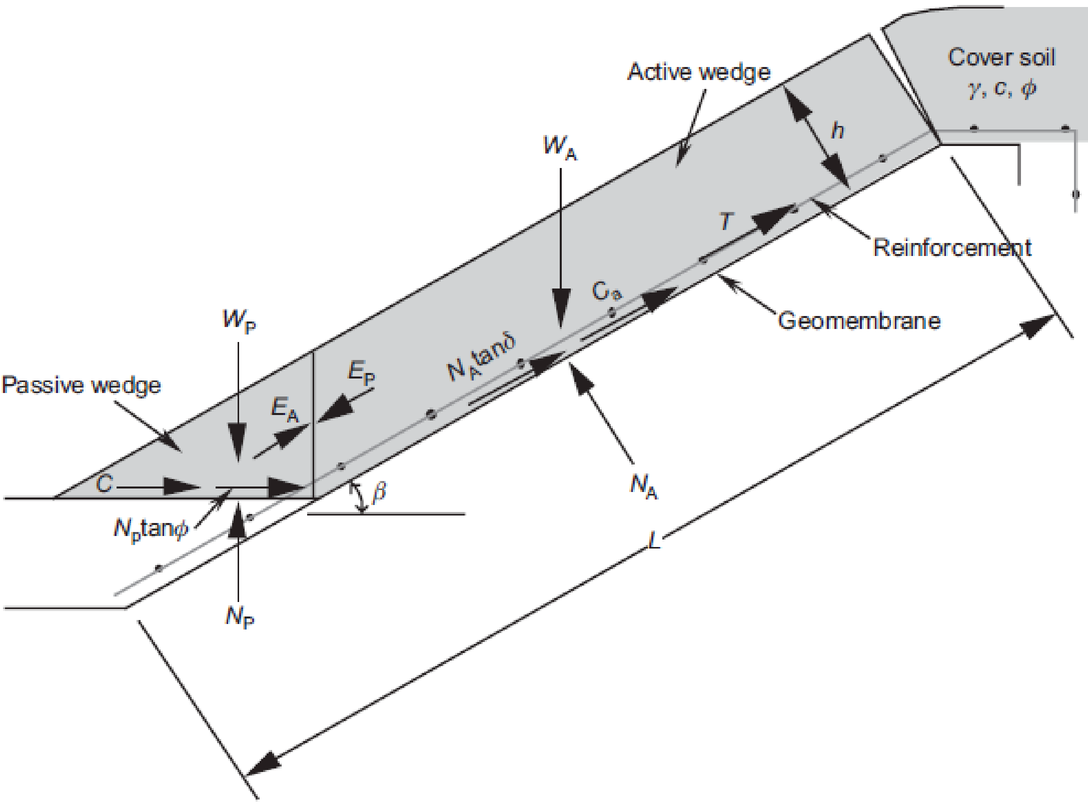


Fig. 40 – Influence of veneer reinforcement for a cover soil slope (Koerner and Soong, 2005)

Compared to the previous scenarios, in this situation there is an additional stabilising component (T) acting within the active wedge parallel to the slope. Through the horizontal and vertical balance of forces, and using the same simplifying assumption as in the previous cases, the coefficients of the second-degree equation for calculating the factor of safety are expressed as:

$$a = (W_A - N_A \cos\beta - T \sin\beta)\cos(\beta)$$
$$b = -[(W_A - N_A \cos\beta - T \sin\beta)\sin(\beta)\tan\phi + (N_A \tan\delta + C_a)\sin\beta\cos(\beta) + \sin(\beta)(C + W_P \tan\phi)]$$
$$c = (N_A \tan\delta + C_a)\sin\beta\sin(\beta)\sin(\beta)\tan\phi$$

The parameter T applied at the design stage takes into account possible damage mechanisms during installation, long-term degradation, creep and in some cases also takes into account the presence of seams in the geosynthetic. The value used can be obtained as follows.

$$T_{allow} = T_{ult}\left(\frac{1}{RF_{ID} \cdot RF_{CR} \cdot RF_{CBD}}\right)$$

Where:
- T_{allow} is the permissible value for ringing
- T_{ult} is the ultimate value of the material
- RF_{ID} is the reduction factor taking into account damage during installation
- RF_{CR} is the reduction factor taking into account creep reduction
- RF_{CBD} is the reduction factor for degradation of the reinforcement material

Intentional reinforcement of the backfill is often provided by high-strength geogrids or geotextiles placed over the top surface of the low-strength interface material. The reinforcement is usually placed directly above the geomembrane or other geosynthetic material.

5.3.4 Veneer reinforcement: unintentional

Unintentional veneer reinforcement is achieved through the action of a geosynthetic placed on an interface with a lower shear strength. The most common cases are:
- Presence of a geosynthetic placed on a geomembrane
- Presence of a geomembrane placed on a geotextile layer with a protective function
- Presence of a geosynthetic placed on a compacted clay layer with lining functions or GCL with lining functions
- Multilayer geosynthetics placed on a layer of compacted clay or clay Geosynthetics placed on a layer of compacted clay or GCL.

The cases described above are depicted graphically below.

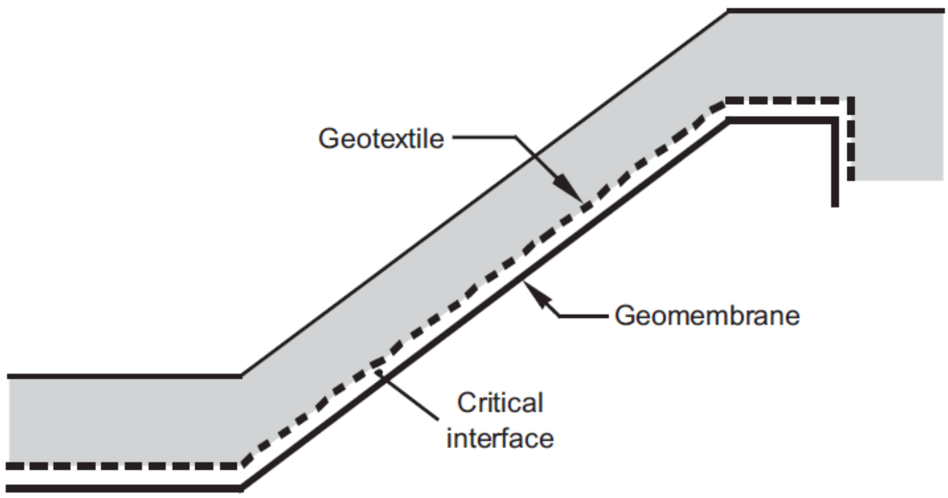


Fig. 41 – Veneer reinforcement with geotextile on a geomembrane (Koerner and Soong, 2005)

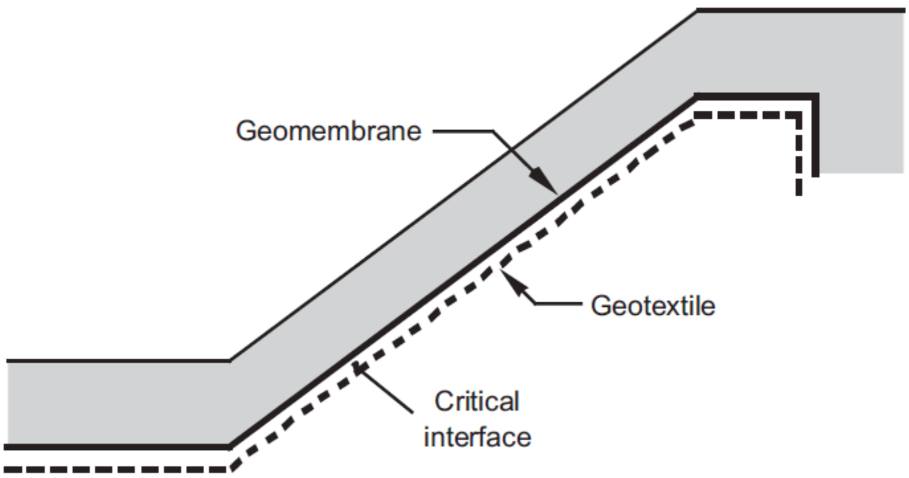


Fig. 42 - Veneer reinforcement with geomembrane on geotextile (Koerner and Soong, 2005)

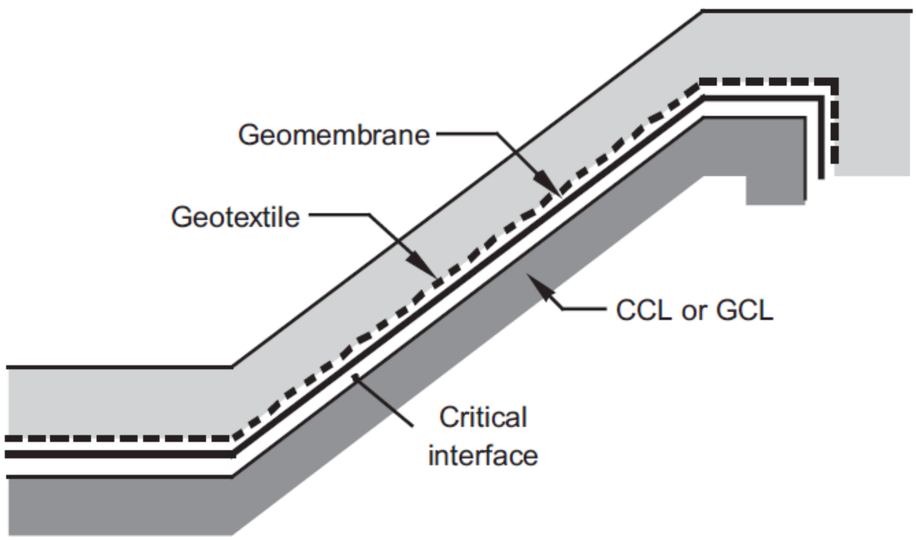


Fig. 43 - Veneer reinforcement with geotextile and geomembrane on GCL or CCL (Koerner and Soong, 2005)

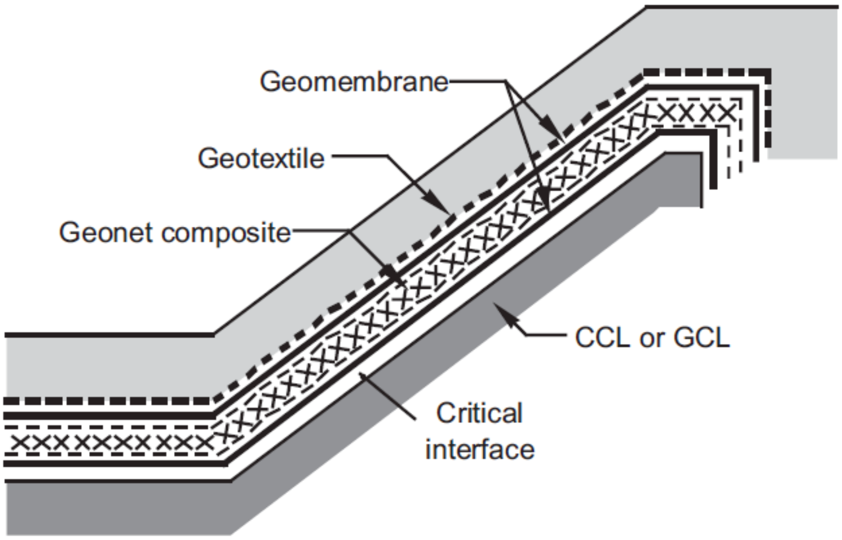


Fig. 44 - Veneer reinforcement with double liner system on GCL or CCL (Koerner and Soong, 2005)

5.4 Final considerations

The factor of safety (FS) for slopes with overburden soils is considered under global conditions and may vary according to site conditions, the service life of the structure, and the context. In all cases, it is the responsibility of the designer to choose the appropriate factor of safety value for the specific case, but recommendations on the use of the minimum global factor of safety can be found in the literature depending on the underlying waste. The table 1 summarises these considerations.

Table 1 – Safety factor for different type of waste

Ranking	**Type of waste**			
	Hazardous waste	Non-hazardous waste	Abandoned dumps	Waste piles/leach pads
Low	1.4	1.3	1.4	1.2
Moderate	1.5	1.4	1.5	1.3
High	1.6	1.5	1.6	1.4

6. Peak versus residual interface shear strength

The direct shear test allows the evaluation of the shear strength at the interface between two materials (soil-geosynthetic, granular-geosynthetic material). This value is fundamental in the dimensioning of the components of a landfill cover system and for the selection and application of the most suitable materials. It is very important to use shear strength values, offered in the literature, only in comparative terms, since every situation is different. This chapter will discuss the design of landfill slopes lined with geosynthetics and the instability mechanisms between the interfaces of the multilayer system.

As explained in Chapter 4, three tests are performed on the same sample, varying only the magnitude of the normal stress. The measured values of resistance and its trend are shown in the figure 45.

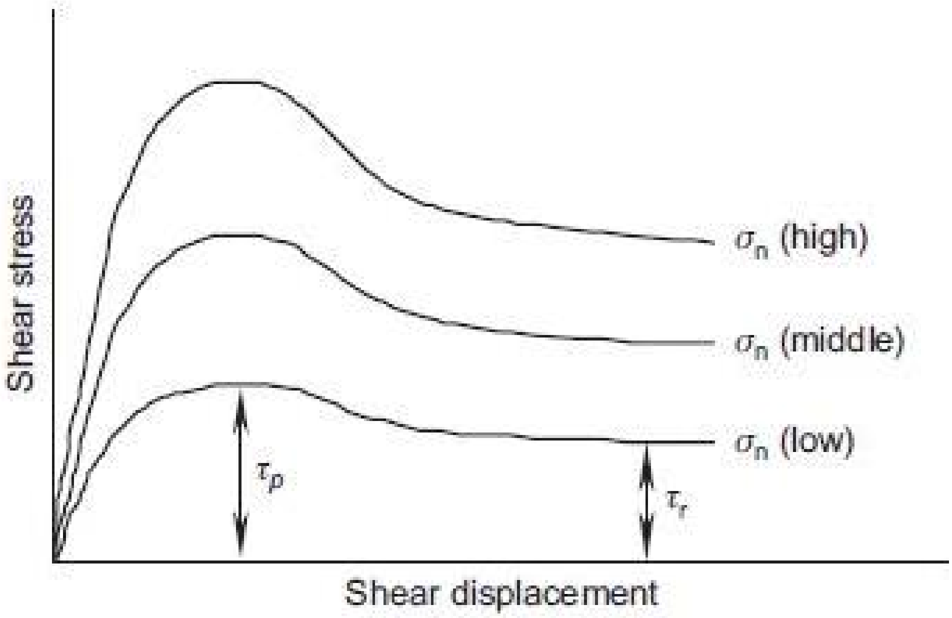


Fig. 45 – Relationship between shear stress and shear displacement

Two values can then be obtained for each test:

Peak shear strength: τ_p

Residual shear strength: τ_r

Next, the Mohr-Coulomb criterion is applied graphically to derive the peak and residual envelopes. As can be seen from the figure 46 and the respective equations, two fundamental values are obtained for each shear resistance:

- Interface friction angle:
  - Peak: δ_p
  - Residual: δ_r
- Cohesion:
  - Peak: c_{ap}
  - Residual: c_{ar}

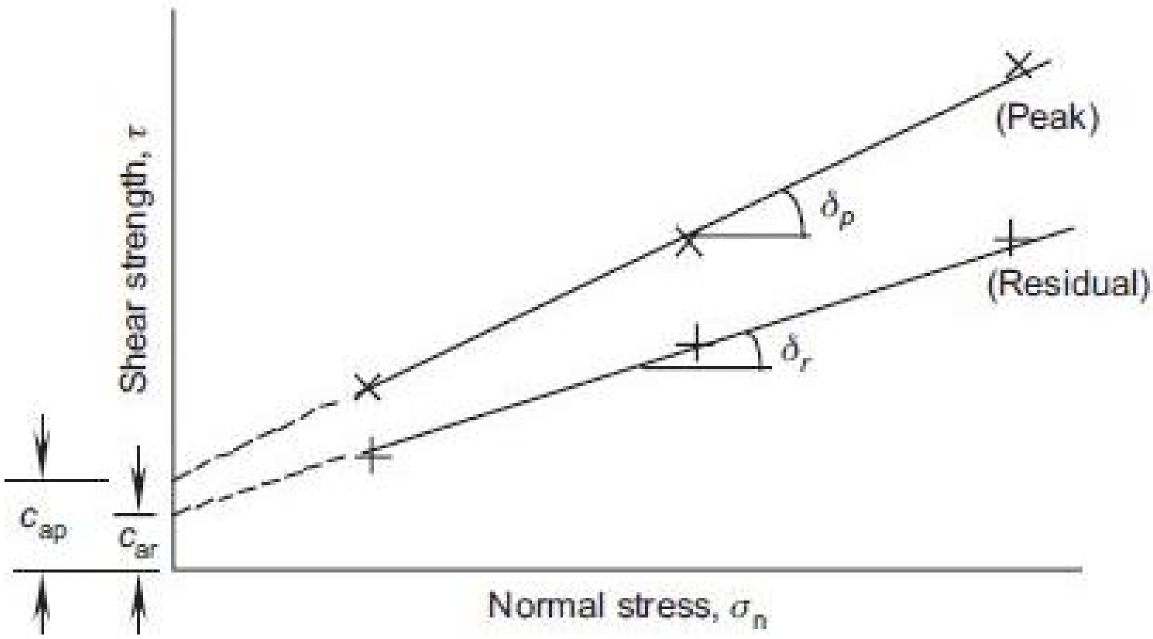


Fig. 46 – Mohr-Coulomb criterium

$$\tau_p = c_{ap} + \sigma_n * tan\delta_p$$

$$\tau_r = c_{ar} + \sigma_n * tan\delta_r$$

The δ value can have a maximum value equal to the soil shear strength angle value if the latter is involved as an interface. The maximum value of c_a, on the other hand, is the value of soil cohesion.
This theory, used in the geotechnical field and therefore in the presence of soils, granular materials, can be applied and adapted to the presence of geosynthetic materials and waste.
For example, in terms of cohesion, there must be a physical justification for the application of this concept. Geosynthetics such as textured geomembranes, or the bentonite component in a GCL are cases where cohesion is physically justified. In any case, analyses are often conducted neglecting the cohesion value as a conservative assumption.
As can be seen from the figure 46, the residual strength values are lower than the peak strength values. The difference between the two values depends on the geometric and mechanical characteristics of the material.

In stability analyses, the use of the peak strength value rather than the residual strength value is left to the designer as this is a material- and site-specific issue.
The choice of interface shear strength to be used for the design of the lining and cover system is important because it affects the disposal capacity of a waste containment plant. The usual goal for waste containment facilities is to maximise storage capacity. Thus, the side slopes are designed and constructed as steep as possible, and the height of the waste and the slope will be as high and steep as possible, respectively.
As stated earlier, residual strength values are lower than peak strength values, with the former being 50-60% lower than the latter.
The use of a residual value results in a landfill geometry with smaller sides, with reduced storage capacity and decreased landfill performance. The choice of a peak value leads to a higher storage capacity but requires greater caution to consider instability phenomena and to avoid high reclamation costs. Within a waste disposal site, it is possible to conduct stability studies considering different resistance values depending on the spatial location of the situation being analysed: it is possible to consider a peak value at the crest of a slope and residual values at the tip of the slope.

The residual resistance of an interface can be mobilised through various activities, such as the placement of a mass of waste, lateral movement or swelling of the waste, construction of the lining system, expansion or

contraction of the geosynthetic material, seismic events, stress transfer between waste and the base of the landfill or side slopes. All these events can generate shear displacements that mobilise the residual resistance of an interface resulting in progressive failure between the side slope and part of the base of a lower liner system.
The residual shear resistance of an interface only develops in the field if a damaging shear displacement occurs along the interface where the geosynthetic is placed. To study this condition correctly, two important factors must be considered:

- The 'detrimental' or 'damaging' shear displacement
- The interface along which this displacement occurs

The damaging shear displacement indicates that the shear resistance at the interface exceeds the peak value due to this displacement and is about to reach the residual value.

6.1 Design of landfill lining systems

The overall stability of the slope depends on the interface in the base lining system having the lowest peak resistance and waste resistance and is independent of the lateral slope.
The instabilising event, as mentioned above, is the force due to the triangle of waste lying within the landfill, which comes into contact with the side slope of the disposal site. The control of the stability of this volume of waste is related to the interface shear strength that is mobilised along the side slope and the base of the landfill. The greatest stress caused by the waste volume in the landfill concerns the base, and the surface of instability has a greater length given the assumption that the interfaces appear on both the base and the side slope of the landfill. The interface shear strength value in the base zone is given by:

$$\tau = \sigma_{0n} \tan(dp)$$

Where:

- dp is the peak strength angle of the weakest interface
- σ_{0n} is the effective normal stress acting on the base interface.

In order to ensure overall stability of the landfill, the volume of waste must mobilise some shear resistance along the base, due to the low shear resistance exhibited by the geosynthetic interfaces located along the slopes.
The resistance along the side slope is low due to the low σ_{0n} e dp along the slopes at the sides of the base. Therefore, damaging shear displacement may occur more simply along the side walls of the landfill, and this may mobilise the passive resistance of municipal solid waste along the base of the landfill.

One aspect to be considered in order to properly assess the stress state acting on the base is the compressible nature of the waste. The stress exerted by a mass of waste is indeed different from the stress generated, for example, by a block of concrete, in that there are no major changes in stress over time. The compressible nature of waste means that the shear displacement required to mobilise a shear resistance along the base is greater than in the case of concrete.

Slope failure occurs when the driving force exceeds the mobilised force of the weakest layer. An example of this is when the angle of inclination is greater than the friction angle of the weaker layer. The interface at which this condition occurs is called an overloaded interface. This local overload generates a shear displacement; the shear stresses applied to this interface are transferred to the immediately adjacent interface element, since the overloaded interface is in the post-peak phase and cannot withstand the stresses generated.

If the adjacent interface cannot withstand these stresses (shear stresses large enough to cause problems on this interface as well), the stresses will be transferred further. This process can continue until the slope collapses.
The collapse can be avoided if the interface with the weakest interface shear resistance (hence the lowest res) increases sufficiently to counteract the initial overload. This result is important as it explains how even if a limited portion of the interface reaches a post-peak condition, not all of the slope needs to be designed using the residual strength. One solution could be to transfer the stresses to the base of the landform.

Gilbert and Byrne (1996), Reddy et al. (1996) and Filz et al. (2001) also suggest the possibility of progressive failure occurring along a line interface, and thus residual or post-peak resistance, respectively, may be applicable. In summary, a residual resistance of the interface can be mobilised along a landform side slope, while a peak resistance of the interface is mobilised along the base.

6.1.1 Design of the Composite Rupture Casing for the Bottom Lining System

The interface where a damaging shear displacement can develop is the one with the lowest peak strength in the lower casing system, regardless of the residual shear strength value.
In fact, if the interface with the lowest peak resistance has the highest residual shear strength, damaging shear displacement may occur, but the resulting stability will be controlled by the residual resistance along this interface and not by the lowest residual resistance of the interface (e.g. a GCL). If the damaging shear displacement occurs on a surface with a lower peak resistance, that interface will govern stability and the interface with the lower residual resistance will not be considered.

As explained above, shear stresses can be transferred between interfaces, so if more than one surface is used to develop the failure envelope, it will be referred to as a composite failure envelope.

The procedure for constructing a peak composite rupture envelope uses the following three steps:

1. Determine the interface(s) or material(s) in the composite envelope system that have the lowest peak strength for the full range of normal stresses encountered along the lower envelope system.
2. Determine the maximum composite failure envelope for the weakest interface(s) or material(s) in the composite cladding system for the full range of effective normal stresses encountered along the cladding system.
3. Determine the residual composite failure envelope that corresponds to the composite failure peak in step 2.

For a better understanding of the procedure just explained, graphic examples are shown, where different interfaces and their envelopes are analysed under peak (fig. 2-3) and residual (fig.4-5) conditions obtained through a torsional ring shear device. The interfaces analysed are:

- Geotextile - geomembranes
- Clay - geomembranes
- Geonet - geomembranes

First considerations will be made on the failure envelopes under peak conditions (fig. 47,48)

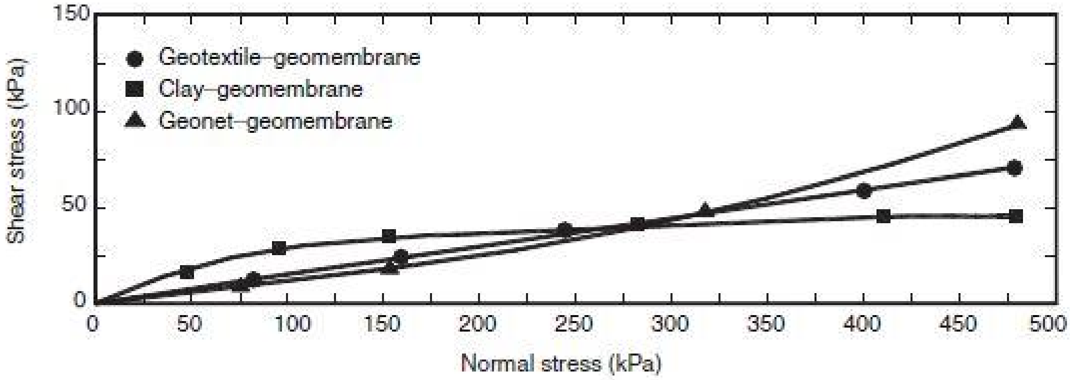


Fig. 47 – Peak failure envelopes (Stark and Poeppel, 1994)

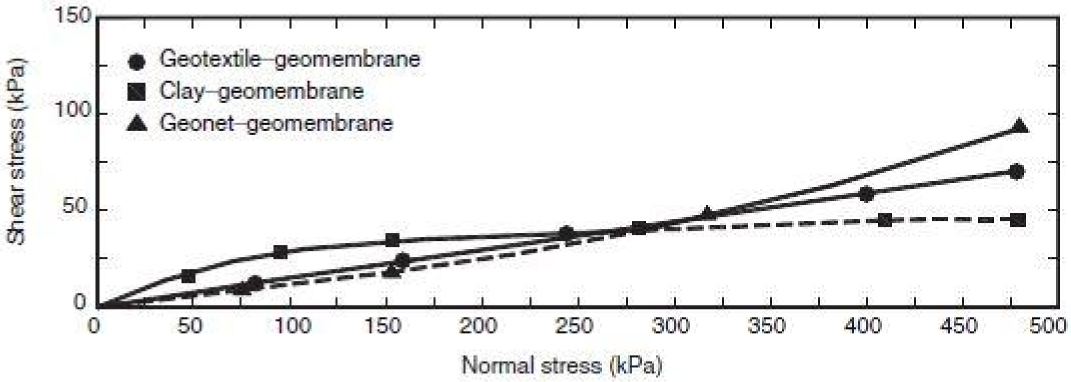


Fig. 48 - Peak composite envelope (Stark and Poeppel,1994)

For a normal interface stress value of σ_{0n} = 280 kPa, it can be seen from the graph in fig. 47 that the 'geonet-geomembrane' interface presents a lower peak resistance value, whereas for values of σ_{0n}> 280 kPa., the clay-geomembrane interface presents the lower values. According to the composite interface concept explained earlier, the composite failure envelope is depicted in fig. 48 with a dashed line. It depicts the weakest composite interface, along which residual strength mobilisation may develop after the peak strength has been exceeded.

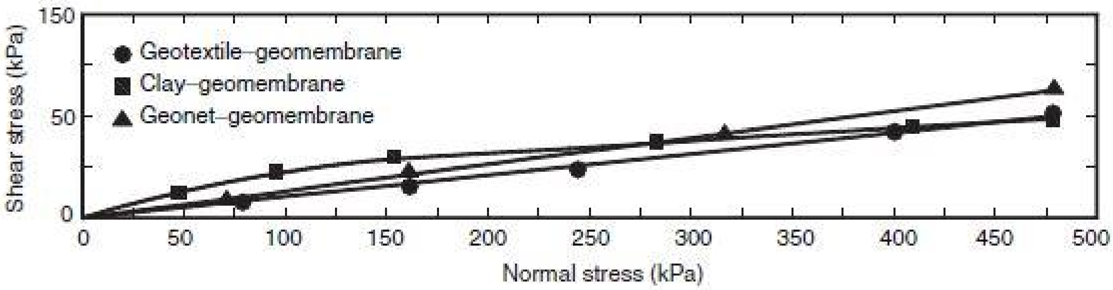


Fig. 49 – Residual failure envelopes (Stark and Poeppel,1994)

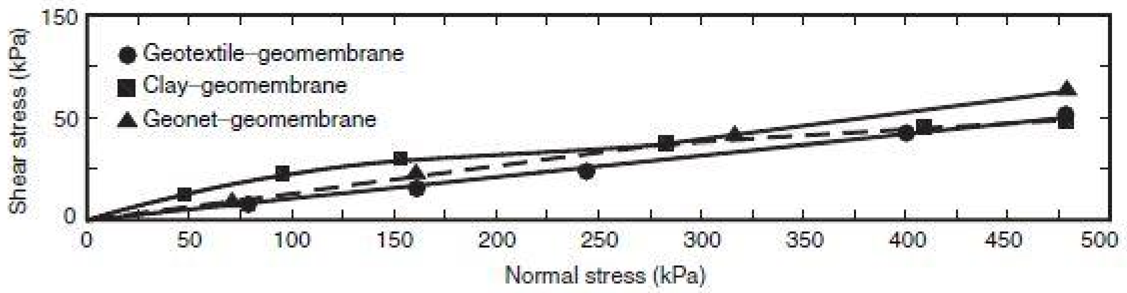


Fig. 50 – Residual composite envelope (Stark and Poeppel,1994)

With regard to the failure envelopes under residual conditions (fig. 49,50), for σ_{0n}= 280 kPa, the geotextile-geomembrane interface has the lowest residual strength, while for σ_{0n}> 450 kPa the clay-geomembrane interface has the lowest residual strength. In this case, the composite failure envelope does not relate to the two interfaces mentioned above, but always refers to what happens under peak conditions. Under peak conditions, the composite envelope concerned the geonet-geomembrane and clay-geomembrane interface. So again it will concern the same interfaces. It can be seen that the composite envelope of breakage under residual conditions is not the envelope with the lowest residual strength. In fact, the interfaces with lower residual strength are not affected under peak conditions, so their residual strength is not mobilised.
In cases where there is not a large deviation between the fracture envelopes, one can proceed in a precautionary manner by considering lower residual strength values (dotted line envelope in fig. 50), in order to increase the factor of safety for all interfaces.

6.2 Design of landfill cover systems

In the design of landfill cover systems, the conditions of instability and the causes that may lead to them are different from those for the lining system, so the considerations made for the latter cannot be applied to the cover system. A substantial difference concerns, for example, the stress system acting on the system. The stress system acting on the roofing system is significantly lower than that acting on the cladding system. Common recommendations for the design of the covering system of a landfill site are:

- use a maximum shear resistance of the weakest interface
- consider a shear resistance with a safety factor greater than 1.5 in the case of composite failure envelopes

As stated earlier, an unstable situation can arise when the destabilising force exceeds the resistance mobilised by the weak layer. This scenario can be realised when the slope angle exceeds the friction angle of the weak layer. To analyse this condition, laboratory tests such as ramp tests can be conducted, as illustrated in Chapter 4.
Ramp tests are very useful for studying the stability of landfill cover systems or for slope erosion control. Since, thanks to technological progress, the side walls of a landfill are composed of different layers of geosynthetic materials that perform different functions, it is important to study the interactions between the different layers in order to avoid the collapse of the structure, which would imply a further use of resources (material and economic) and time. This type of test can effectively simulate low stress levels at the interfaces, a situation very close to the real scenario that occurs in a landfill.
In order to reduce these traction forces, the cover soil can be reinforced. Placing the reinforcement directly on the geomembrane and not on the ground would be the easiest solution to implement but not the most efficient in terms of stability, as can be seen from the ramp test performed.
If there are large displacements due to the buckling events described above, the use of geosynthetic reinforcement in the roof system allows the peak shear strength values of the weakest interface to be used but considering a factor of safety greater than 1.5.
The use of a peak interface is recommended for the roofing system due to the lack or limited amount of

damaging shear displacement along the weaker interface in a roofing system compared to a lateral slope of the cladding.
If the slope of the roofing system is greater than the peak shear strength value of the weaker interface, progressive failure may occur, so a residual interface friction angle should be used for design.

7. Study case – Landfill "Chivasso 0"

The 'Chivasso 0' landfill project site of interest is located in the N - NW sector of the municipal territory of Chivasso, at Fornace SLET and Regione Pozzo, on the border with the Municipality of Montanaro, in the Province of Turin. The site is identifiable on I.G.M. maps at a scale of 1:25,000 in the I S-E "Chivasso' of F° 56 'Torino' at U.T.M. coordinates: 32T MR 120 081 (referring to the centre of gravity of the site). From a cadastral point of view, the area falls within Sheets no. 30 and 31 of the Municipality of Chivasso in the land parcels Nos. 51, 52, 52-01, 53 and 90 occupying a total area of approx. 90,200 square metres. The landfill was built pursuant to ex-article 12 of Presidential Decree 915/82 by constructing a controlled landfill for non-hazardous waste and annexed volume reduction plant.
The following figures (fig. 51,52,53,54,55) show the geographical framework of the analysed site.

Fig. 51 - Geographical framework: national level (Google Earth, 2021)

Fig. 52 - Geographical framework: regional level (Google Earth, 2021)

Fig. 53 - Geographical framework: municipal level (Google Earth, 2021)

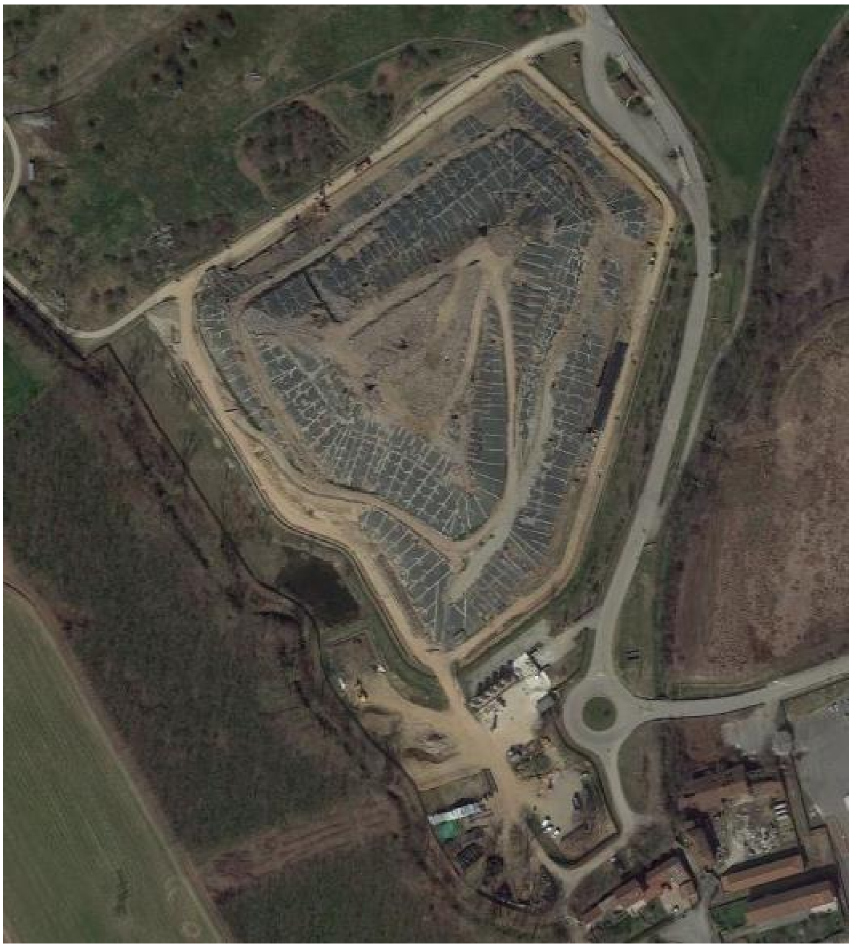


Fig. 54 – “Chivasso 0” Landfill (Google Earth, 2021)

Fig. 55 - "Chivasso 0" Landfill (Oggeri and Capozzo, 2022)

The average altitude at which the target area is located is about 200 m above sea level. The territorial context in which the site in question is located is characterised by the presence of a landfill complex (fig. 56) and agricultural areas consisting of medium-sized plots destined for arable cultivation with modest settlements and scattered farmsteads.

Fig. 56 - Adjacent landfill complex (Oggeri and Capozzo, 2022)

7.1 Geological and geomorphological framework

7.1.1 Geomorphological framework

The area under examination is located in the north-eastern sector of the Turin plain, on the right bank of the River Po; the average altitude of the area where the site under examination is located is just over 200 m above sea level. The area where the landfill is located is characterised by topographical irregularities due to natural phenomena and anthropic interventions, linked to previous brick production activities.

The site under examination can be framed within a broad 'quadrilateral delimited:
- to the north by the morainic relief of Ivrea
- to the east by the course of the Dora Baltea River
- to the west by the Orco stream
- to the south by the course of the river Po.

The area thus identified is overall sub-flat, with an altimetric gradient which is very nearly equal to 1%, with a topographical surface related to the shape of the fluvioglacial conoid present in the area concerned.

Reducing the scale of investigation, the situation is more articulated, as it is possible to observe morphological elements referable to fluvial terracing processes, linked to the presence of the Dora Baltea River and the Po River, and local streams such as the Orco torrent.

7.1.2 Geological framework

From a geological point of view, the area under examination is located near the southern edge of the wide conoid of fluvioglacial origin that departs from the Ivrea Morainic Amphitheatre, and that from the morainal reliefs progressively descends southwards, until it connects with the neighbouring Po River alluvial plain of the Po River, which flows at the bottom of the Collina di Torino.

The area where the site in question rises has been characterised by changes at a geological level, due to the presence and action of the various watercourses present, the Valle d'Aosta glacier and tectonic evolution.

From a cartographic point of view, the area investigated, indicated by a red circle, is included in Sheet no. 56 of the Geological Map of Italy, published at a scale of 1:100,000 (fig.57). The formations appearing in the surroundings of the site are defined as *"gravelly-sandy deposits, with orange-red palaeosols, prevalently terraced, corresponding to the fundamental level of the high plain, connecting with the rissian morainic circles"*.

Fig. 57 - Sheet no.56 of the Geological Map with the site of interest

As stated in the previous paragraph, the area was affected by terracing phenomena, which allowed the identification of two morphologically different areas:

- A more elevated area at an altitude level with respect to the plain below, corresponding to the summit surface of the ancient fluvioglacial conoid
- A second zone located at the bottom of the previous zone, which includes the alluvial plain generated by the sediments deposited by the Orco torrent.

Terracing phenomena have triggered and accelerated erosion phenomena, leading to a morphological discontinuity of the main escarpment, which is interrupted and marked by minor incisions.
The top of the terraced surface, on which the site under examination stands, therefore corresponds the most superficial morphological expression of the fluvioglacial palaeoconoid: the escarpment of the terrace is modelled in fluvioglacial deposits with a predominantly coarse grain size, referable to the predominantly coarse grain size, referable to the solid contributions of the 'glacial discharges' coming out of the Anfiteatro Morenico di Ivrea.
Regarding the analysis of the sediments present in this area, they are mainly gravelly-pebbly and sandy in nature. It must also be taken into consideration that, due to the distance of the site from the morainic

amphitheatre, there is a lack of larger blocks that are instead found in other areas of the quadrilateral considered.
The lithological nature of the clasts that make up the alluvial deposits therefore reflects the lithology of the formations outcropping in the mountainous portion of the catchment area of the watercourse of the river basin: clasts of crystalline lithotypes of both magmatic and metamorphic origin are often well recognisable; the granulometric and sedimentological characteristics of the the grain-size and sedimentological characteristics of the deposits under analysis can be traced back to the mode of transport and their subsequent deposition by the waters of the watercourse. deposition by the waters of the stream that originated from the glacier front.
The mode of transport also influenced the processes of mechanical disintegration and chemical alteration of the clasts. The softer and more easily disintegrated rocks were progressively removed, so it is difficult to identify the clasts belonging to these rocks.
The analysis of the harder and more resistant rocks, on the other hand, was possible thanks to the analysis of clasts from gneisses, granitoid rocks or serpentinites.

At a stratigraphic level, the deposits present in the area under examination can be traced back to a reddish-brown soil layer, with a silty-clayey texture, with a thickness of up to 6 ÷7 meters, which is identified as 'palaeosols'. There is also the presence of another layer, decimetric in thickness, of aeolian silts, also known as 'loess', which overlie the paleosols.

7.2 Hydrogeological framework

7.2.1 Surface waters: hydrography

The surface hydrography of the area under examination is characterised only by the presence of the artificial watercourse, originating from a derivation located on the Orco stream, “Gora della Campagna”, which flows in a position west of the area under examination. The main watercourses are distant from the area under examination, as the Orco stream flows about 2.5 km from the site, and the Po River about 4 km.

Since it is an artificial watercourse, the water flow rates of the "Gora della Campagna" are subject to natural seasonal variations, as well as to agricultural needs related to irrigation requirements and the actual availability of the water flow rate derivable from the Orco stream; based on literature data, the "Gora della Campagna" has a maximum flow rate of 1.75 m^3/s.

7.2.2 Groundwater: hydrogeology

The hydrogeological structure of the area under examination is linked to the lithographic structure described above, as the characterisation of the aquifers present is a function of the continuity of the lithostratigraphic levels present in the area considered.

The reconstruction of the general lithostratigraphic structure described above therefore allows the identification of the main aquifers and the determination of the depth and trend of the levels of impermeable fine material (aquicludes): in fact, the latter constitute the main hydrogeological constraints, as they condition the circulation of groundwater and determine the greater or lesser vulnerability of the water tables to possible pollution from the surface.
According to bibliographic documents, it is possible to identify three different hydrological complexes in the area under examination, different from a lithological point of view and characterised by a different permeability coefficient. The hydrogeological complexes are presented below, in order of depth.
The three identified hydrological complexes are listed below, from the shallowest to the deepest:

- The first hydrogeological complex, for simplicity's sake referred to below as Complex A, consists of alluvial and fluvioglacial deposits of the Quaternary age. It consists of an almost continuous succession of deposits with a gravelly-sandy grain size, with subordinate silty-clayey intercalations, generally of limited extension.

- The second hydrogeological complex, for simplicity's sake referred to below as Complex B, consists of the deposits in the Villafranchian facies. This is a complex of sediments from fluvial-lacustrine and marshy environment deposits that are commonly referred to in geological literature as the Villafranchian facies. commonly referred to in geological literature with the chronostratigraphic term of 'Villafranchian'. From a lithological point of view, it is an alternation of impermeable (silty-clayey) and levels with good permeability (gravelly-sandy): the latter host water tables that are more or less intercommunicating, depending on the areal continuity and thickness of the impermeable silty-clay septa, of the semi-confined to confined type.

- The third hydrogeological complex, for simplicity indicated as Complex C, consists of the tertiary marine deposits. These are generally fine-textured sediments, consisting of marls, clays and sandstones, which can be considered on the whole as impermeable (aquiclude), except for the local presence of more permeable sandy levels, within which confined (pressurised) aquifers and 'connected waters' may be hosted.

In the subsoil of the area under examination, an aquifer can therefore be recognised Superficial and an underlying deep aquifer, represented by the sediments of Complex B, within which several confined water aquifers may be present, fed by deep hydrogeological circuits that organise deep hydrogeological circuits that are organised on a regional scale.

In the large lowland area north of Chivasso, the presence on the surface of the deposits of Complex A, consisting of coarse-grained soils with a good degree of permeability generally allows effective infiltration of meteoric inputs: the surface water table is therefore fed by infiltration. water table is therefore fed by direct infiltration, from the surface, of rainwater meteoric waters from the surface, while the base of the surface aquifer in which it is contained consists of the silty-clayey levels located at the roof of the underlying Complex B, generally referred to the 'Villafranchian' complex.
The underground water circulation is conditioned, however, by the effective lateral continuity of the more permeable levels: in particular, the pedogenetic processes developed on the terraced surfaces in this area can lead to the presence, on the surface, of layers of variously powerful clayey vegetable soil, capable of hindering the infiltration of rainwater infiltration and direct groundwater supply.

7.2.3 Surface aquifer

The surface aquifer is represented by the sediments of Complex A, within which is contained the groundwater aquifer fed by infiltration water, which can also be referred to as the 'first aquifer' because it is the first one found as one descends to depth.
The general assessment of the characteristics of this aquifer was initially carried out through a regional analysis of the piezometric surface of the water table. From this analysis, the main direction of subsurface water flow and the relationship between surface water and groundwater in the area considered were deduced.
Based on the reconstruction examined, the natural direction of groundwater flow at the site under examination, appears to be directed approximately from north-north-west to south-south-east, with a large-scale average hydraulic gradient of approximately 0.4 - 0.5 %.
The main source of recharge of the aforementioned aquifer is to be found hydraulically upstream of the area

under consideration, as the presence of extensive areas characterised by clay soils makes the infiltration of precipitation difficult. Therefore, the main groundwater recharge zone seems to be found in correspondence with the lowland sectors located within the Ivrea Morainic Amphitheatre.

Following this regional analysis, studies and investigations were conducted to obtain a more detailed reconstruction of the piezometric trend of the surface water table, confirmed in turn by further studies at the provincial scale.
These studies show how in the eastern portion of the Turin plain, as well as in the neighbouring western portion of the Vercelli plain, the flow lines of the water table are directed essentially radially from the Ivrea Morainic Amphitheatre.

By examining the attached piezometric reconstruction of the north-eastern sector of the plain of Turin, taken from the cited publication and referring to the summer of 2002, it can be observed that the flow of the surface water stratum is very variable; from an initial north-west to southeast in the sector of the conoid of the Stura di Lanzo stream, it flexes eastwards in correspondence the Vaude plateau, to then assume, in the sector between the Malone stream and the Dora Baltea, a north-south course, i.e. towards the course of the Po River, which, in the entire area considered represents the general base level of the surface water table.

Throughout the Canavese area, the terracing phenomenon determines a considerable control of the morphology on the piezometry trend; the areas of high morphology coincide on a large scale with areas of underground watersheds and at the edges of the terraces there is almost always an inflection of the isopiezometric lines upstream. The incisions and alluvial belts between the terraces terraces always represent areas of convergence of the groundwater flow; all the incisions and alluvial belts between the morphological terraces always represent areas of groundwater flow convergence. surface water table, especially the Malone and Orco streams.

Among the most evident morphological elements of the piezometric surface is the subterranean watershed that, from outside the Ivrea Morainic Amphitheatre, continues initially towards the south-west, then bends southwards and roughly delimits the area of influence of the perifluvial belt of the Orco stream(which acts as a 'draining axis') from the remaining portion of the fluvioglacial conoid: this watershed thus separates the water flow directed south, towards the Po River, as at the site under study, from that directed south-west, towards the Orco stream.

7.2.4 Hydrodynamic characterisation of the surface aquifer and the deep aquifer

- **Surface aquifer**

Once the type of water table present at surface level had been identified, studies were conducted to identify the hydrodynamic characteristics of the aquifer, in order to verify that the site chosen for the location of the landfill was indeed suitable for the construction of a waste disposal site. Moreover, the assessment of these parameters is necessary to safeguard underground water resources, as well as for the design of accessory and functional works and plants to the plant (purging wells).

The determination of the hydrodynamic parameters of the aquifer can be obtained through the interpretation of experimental data obtained from various tests (such as pumping tests in transient regime with constant flow rate).
The term "permanent regime", on the other hand, refers to the type of regime that is reached when at any point in the aquifer the components of velocity, pressure and density are independent of time.

Moving on to examine the main hydrodynamic parameters of the aquifer, it can be said that the transmissivity T is defined, physically, by the volume of water that can flow through an aquifer section of unit width and height b, equal to the thickness of the aquifer, in the unit time, when there is a unit piezometric gradient. It is defined by the following expression:

$$T = K \cdot B$$

Where

- K is the hydraulic conductivity of the aquifer (m/s)
- B is the thickness of the aquifer (m)

The storage coefficient S is then used to indicate the volume of water released or stored per unit area of the aquifer, when there is a change in load piezometric unit: in groundwater aquifers this parameter can be considered as equal to the effective porosity, and usually between 10^{-1} and 10^{-3}.

The interpretation of the experimental data made it possible to quantitatively determine the hydrodynamic parameters of the aquifer, obtaining the following results:

- Aquifer transmissivity: $T = 5.9 \cdot 10^{-2}\, m^2/s$
- Storage coefficient: $S = 8.43 \cdot 10^{-2}$
- Drainage factor: 40 m.

Again in order to determine the hydrodynamic characteristics of the surface aquifer, as part of the investigations for the characterisation of the area, 5 slug tests were then carried out to proceed with the expeditious determination of the hydraulic conductivity: the test consists of inducing an instantaneous variation (raising) of the piezometric level in the monitoring well through the introduction of a solid of known volume and the subsequent measurement of the lowering of the water levels, at a pre-established rate, until the initial level in the well is restored (or at least 80% of it).
In the present case, the tests performed confirmed the high productivity of the aquifer, as as the variations induced in the piezometric levels were very small, of the centimetric order, and the restoration of the initial conditions of the water levels occurred very quickly.

- **Deep aquifer**

In the area under examination, the aquifers from which the Chivasso aqueduct draws through of the wells located to the south of the landfill site are therefore hydraulically separated from the surface water table by these impermeable silty-clayey septa, whose lateral continuity can be correlated over the entire extension of the aforementioned district, as shown by the interpretation of the stratigraphies of the deep soundings carried out in past years, and which protect them from any pollution phenomena that may affect the surface phreatic aquifer.
Through several emunition tests, the effective hydraulic separation between the surface aquifer and the deep aquifer was verified and the hydrodynamic characteristics of the latter were determined. Throughout the entire duration of the test, no significant piezometric variations were detected in the surface aquifer, demonstrating that the drawing from the deep aquifer did not trigger drainage phenomena from the one above, thus certifying the presence of an effective hydraulic separation between the two aquifers.

7.3 Meteoclimatic analysis

A meteoclimatic analysis is useful to predict the rainfall regime affecting the area under consideration, as rainfall is one of the main factors contributing to leachate formation. For this reason, to make the discussion comprehensive, this section is included within the general overview of the site under consideration.

For the purposes of the meteoclimatic analysis of the area in question, available data from historical series and/or hydrological annals and data recorded by the meteorological station in the landfill area were taken into consideration.

The latter, although the number of years surveyed is not sufficient to provide a statistical significant basis, they still provide precise and continuous monitoring of meteorological parameters that may, in some way, influence the potentially induced impacts from landfills on the external environment.

The area concerned falls within the Po Valley district, characterised, in terms of temperature, by a temperate continental climate with prolonged cold winters and long hot summers with high atmospheric humidity.
The average annual temperature is 13 °C: the average temperature of the coldest month (January) is of 0.76 °C, that of the warmest month (July) is 23.49 °C; the number of frost days per year is equal to 54.

The rainfall regime in the area is pre-Alpine (type 'A'), characterised by a weak water depression in the summer quarter without any dry periods atmospheric. The percentage distribution of precipitation sees a concentration of same in spring (31.6%) and autumn (26.8%). The average hydrometeoric supply year is 827.1 mm (75.4 rainy days per year).

For the study of intense, short-lasting precipitation, empirical relationships can be used to calculate climate possibility curves, which, as a function of return time, can be expressed as follows:

$$h = a \cdot (t')^n$$

where:

- h is the precipitation height (in mm)
- t is the duration of the precipitation itself (expressed in days)
- a and n are, on the other hand, characteristic parameters of the area considered, as a function of return time.

Taking into consideration the hydraulic verifications within the landfill site, a return time of 50 years was chosen as representative. Considering also that the rainfall zone in which the waste plant is located belongs to rainfall homogeneous zone 10, the climatic possibility curve was derived with the following values of the coefficients a and n:

$$a = 33.171 \cdot \ln(T_r) + 76.94 = 206.71$$

$$n = 0.016 \cdot \ln[\ln(T_r)] + 0.392 = 0.41 \ (for\ corrivation\ time\ lower\ than\ 1\ day)$$

so that the formula can be written into the expression:

$$h = a \cdot (t')^n = 206.71 \cdot (t')^{0.41} = 56.06 \ \cdot (t)^{0.41}$$

where:

- t' is the duration of precipitation expressed in days
- t is the duration of precipitation expressed in hours.

7.4 Technological and design features

The executive project consisted of the construction of a new waste disposal site with sealing in compliance with the requirements of Legislative Decree. 36/2003. The project consists in:

- the construction of a first waterproofed storage basin, where waste from the municipalities (new intakes) and waste extracted from the old landfill batch will be deposited
- The total removal of waste deposited in the area in the early 1980s, over a thickness of several meters and in the absence of any waterproofing (apart from that deriving from the silty matrix present within

the fluvioglacial soil after the removal of the palaeosols) and its deposition in an area parallel to that in which the first lot is being built

- In the area where the waste deposited in the early 1980s were removed, the second batch of the Chivasso 0 landfill is being built, with waterproofing that complies with the requirements of Legislative Decree 36/2003, where the new waste will be stored. The volume available for the intervention is 531.000 m^3, of which 86.000 $m^3$of waste from the reclamation.

Within storage area A, there was waste that had been deposited approximately 25 years before the construction of the new batch. This waste has already reached an advanced stage of mineralisation being in the methanogenic phase, so the content of contaminants in the leachate generated by it is significantly lower than the content of the same substances in the leachate generated by acidogenic waste.
The removal of the waste in Lot A was necessary to create a waterproofing layer compatible with the requirements of the regulations, as the waste previously rested on an area without waterproofing. The geological barrier on which the waste had been stored consisted of a fluvioglacial soil with a silty matrix, capable of retarding, thanks to its permeability, the dispersion of the pollutant into the water table.

The project carried out foresaw the realisation of a waterproofing intervention through the laying of a HDPE geomembrane in contact with a layer of compacted clay, purposely made and with a hydraulic conductivity of no more than $10^{-7} cm/s$ (approximately 6 orders of magnitude lower than that measured in the water table, and at least 4 or 5 orders of magnitude lower than that of the unsaturated soil present between the waste and the water table itself).
The clay substrate significantly reduces the dispersion rate of contaminants in the water table, by a factor of 1,000 to 10,000 times compared to the previous configuration of the plot. Added to this is the waterproofing effect shown by the applied geosynthetic.
This engineering configuration also allowed better management of the leachate, which is systematically removed and sent to the treatment plants. In the previous configuration, on the other hand, leachate and precipitation water infiltrating the landfill body flowed directly into the underground water table.

The essential technological components characterising a landfill can be identified in:

- waterproofing system of the storage area (also called basic containment system);
- final cover system;
- leachate drainage and extraction system;
- biogas collection system

7.4.1 Basic containment system

The sealing of the bottom and banks of the landfill is carried out separately, with similar characteristics of the system realised. As for the base of the landfill lots, proceeding from the bottom upwards, above the natural soil, it is composed as follows:

- natural clay layer with permeability K ≤ 10^{-9} m/s and thickness of 1.5 m
- HDPE geomembrane with thickness of 2 mm
- non-woven geotextile with with a weight greater than 300 g/m^2
- layer of shredded pneumatic tyres with thickness of 40 cm with the function of protection of the geotextile and drainage
- layer of aggregates with thickness of 10 cm with drainage function.

As far as the banks are concerned, proceeding from the bottom to the top, the components of the waterproofing system are:

- clay layer with permeability K ≤ 10^{-9} m/s and thickness of 1.5 m

- HDPE geomembrane with thickness of 2 mm

The thickness and permeability values of the clay layer comply with the parameters imposed by the standards, given in Chapter 1.

7.4.2 Final cover system

The final cover proposed in the project differs from the indications of the Legislative Decree No. 36 of 13 January 2003. In particular, the project proposes a multilayer structure made up, from top to bottom by the following layers:

- superficial cover layer (composed of vegetated soil) with a thickness greater than 1 m
- drainage geocomposite, consisting of an extruded geotextile core (HDPE) and a non-woven geotextile;
- natural compacted clay layer with a thickness of 60 cm and hydraulic conductivity less than or equal to 10-8 m/s;
- non-woven geotextile with a grammage greater than 300 g/m^2;
- layer of aggregates with a thickness of 10 cm
- gas drainage layer with shredded pneumatic tyres with a thickness of 40 cm.

The surface cover described above guarantees the isolation of the landfill from precipitation events, also taking into account the expected settlements; the equivalence for water drainage between the layer of granular material (provided for in Legislative Decree 36/2003) and the draining geocomposite (design proposal) was also verified.

7.4.3 Drainage system and leachate extraction

The bottom of the landfills will be constructed with slopes of 1-1.5% for the two landfill lots in order to achieve proper leachate drainage.
The bottom collection network for each lot will consist of

- a main collection conduit having DN 300, made of HDPE pipe suitably slotted on 2/3 of the surface
- a network of secondary conduits arranged in a herringbone pattern on the bottom of the landfill, having DN 200 and made of HDPE piping suitably slotted on 2/3 of the surface

The leachate collected flows through the main conduits, made of HDPE and having DN 500, at the base of the landfill lots to the respective storage and extraction pits.
The leachate collected is extracted and pumped towards the existing accumulation tanks by a pump, specifically for lifting turbid, muddy and aggressive water, housed inside the well; this pump, equipped with automatic level regulation, guarantees the control of the hydraulic head inside the landfill.

7.4.4. Biogas collection system

The plant constructed to capture the biogas has the following components:

- 19 vertical wells drilled in elevation, 200 mm in diameter, with a radius of influence of 25 meters for each well;
- 20 inclined wells built in elevation, 200 mm in diameter, with a radius of influence of 20 meters, located along the sealed banks of the landfill;
- Pipelines made of HDPE with DN 80 and PN 16, in order to realise the transport from the wells to the suction and combustion plant located near the service area at the entrance to the landfill.

For the Chivasso 0 landfill, it is also planned to place a dedicated extraction station near the service area.

7.5 Waste disposal planning

The waste delivered to the landfill comes from the 29 municipalities of the Consorzio di Bacino 16 managed by SETA SpA, from the municipality of San Mauro T.se, from the 57 municipalities of the Società Canavesana Servizi, from the Chivasso purification plant and from AMIAT SpA of Turin, for a value of about 4000 tons/month (until March 2013). The waste mainly conferred is indicated below, with the abbreviations given in the European Waste Catalogue (CER):

- *CER 20 03 01 unsorted municipal waste;*
- *CER 20 02 03 other non-biodegradable waste;*
- *CER 20 03 03 street cleaning residues;*
- *CER 20 03 07 bulky waste*
- *CER 19 08 01 screenings (limited to those from urban waste water treatment);*
- *CER 19 05 01 part of municipal and similar waste not composted.*
- *CER 19 12 04 plastic and rubber*
- *CER 19 12 12 Other wastes (including mixtures of materials) from mechanical treatment of waste other than CER category 19 12 11*
- *CER 15 01 06 mixed material packaging*
- *CER 07 02 13 waste plastic*
- *CER 07 02 99 wastes not otherwise specified*

The material used for engineering purposes, on the other hand, belongs to the following categories*:*

- *CER 16 01 03 end-of-life tyres*
- *CER 19 12 04 plastic and rubber*

The material used for the construction of temporary and permanent ramps and forecourts belongs to the following categories:

- *CER 19 12 09 minerals (sand and rocks)*
- *CER 17 05 04 soil and rocks not containing dangerous substances (as CER 17 05 03)*
- *CER 17 01 07 mixtures of concrete, bricks, tiles, ceramics, without dangerous substances (as CER 17 01 06)*
- *CER 17 09 04 mixed construction and demolition wastes (other than 17 09 01 - 02 - 03 containing certain substances)*

The waste compaction plant plays a crucial role in the waste management process. Prior to disposal at the landfill, the waste undergoes pre-treatment in this plant. The plant is housed in a prefabricated building with an industrial floor that is both waterproof and drivable. Within the building, there is a designated area for receiving and temporarily storing non-hazardous waste that enters the plant.
The central feature of the plant is the volumetric reduction line, which includes a baling press fed by a plate feeder. Bulky waste is also processed by being fed onto the line, where it is initially reduced in volume by a grinder. This process helps to reduce the amount of space that the waste will occupy in the landfill. By compacting the waste, it becomes more manageable and easier to handle. This pre-treatment is an essential step in the waste management process and helps to ensure the safe and effective disposal of waste.

7.5.1 Waste delivery and cultivation scheme

The conferment activities were carried out as follows:

- First activity started in September 2010

- Second activity from April to October 2011: waste from reclamation
- Third activity: construction of the new lot in November 2011, completed in May 2012.
- Fourth activity: transfers to the new lot starting in May 2012 with flows from the City of Turin as well as from the other municipalities.

The trends and quantities of waste sent to landfill are shown in the following graph.

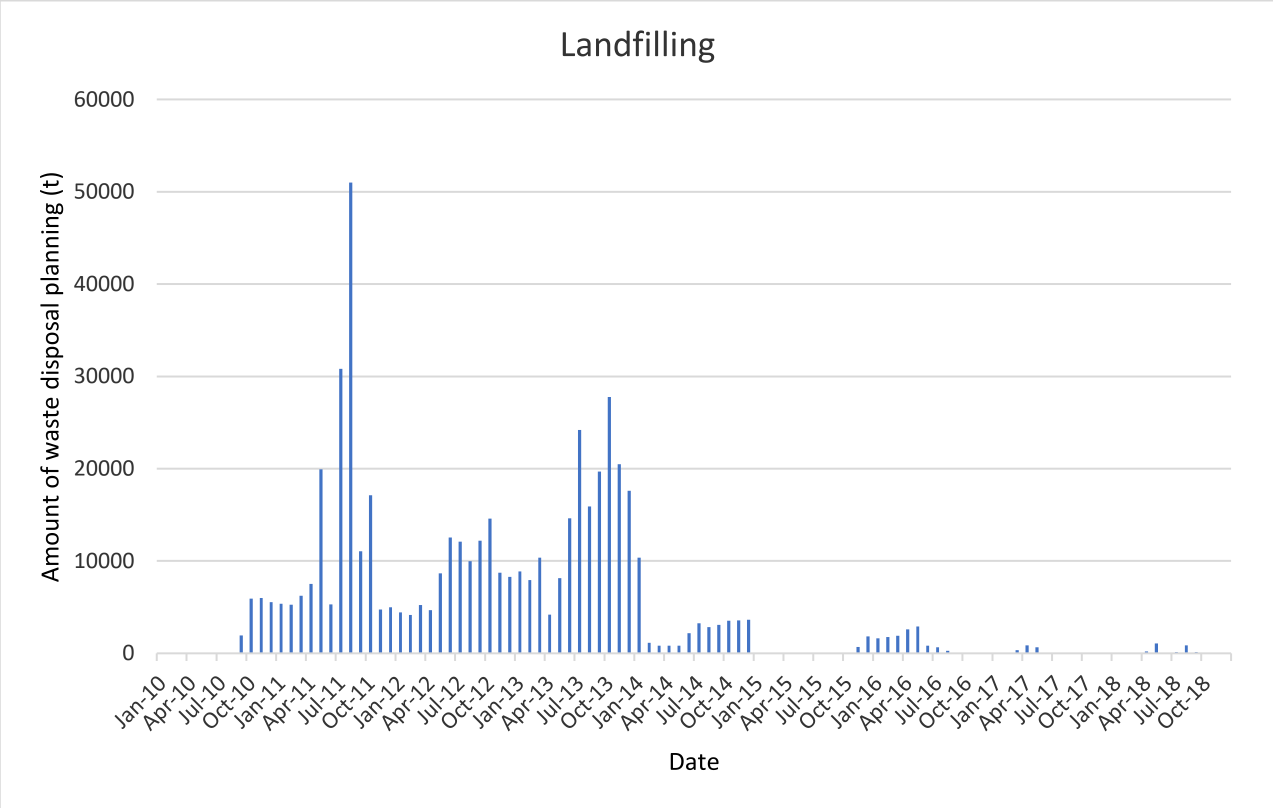


Fig. 58 - Schedule of landfill deliveries

The waste cultivation process involves the use of bales from volumetric reduction treatment of waste to reclaim an area. The process starts with cultivation on Batch 2, where the bales are placed. Once Batch 1 is prepared on the reclaimed area, the dumping of bales is extended to the second batch, leading to the simultaneous cultivation of the two basins.
In cultivating the lots, several general principles must be followed, including maintaining slopes with a 27° angle of inclination to ensure the natural runoff of uncontaminated rainwater away from the waste disposal area. The overflow of leachate outside the waterproofed storage area must be avoided, and the embankments of the cultivation plot should be protected by placing a layer of pneumatic tyres to prevent the waterproofing layer from being lacerated during cultivation operations. Bales should be deposited in layers and reach a predetermined height by overlapping them.
To guarantee stable conditions for the waste and the connected landfill structures, the waste must be unloaded and deposited in layers. Each layer should have a 40 cm layer of inert material to reshape the surface before laying the next layer. A self-propelled crane truck on crawler tracks will deposit the bales in the final storage area.

7.6 Equipment used in landfill

The equipment used in the landfill, which will be discussed in more detail in a later section, are:
- Excavator
- Dumper
- Crawler loader

- Compactor

All work vehicles used by in-house personnel will comply with current regulations and will be maintained in accordance with the provisions of the specific use and maintenance manuals. The management and maintenance activities of the fleet of vehicles and equipment are divided into:

- daily checks
- scheduled maintenance
- extraordinary maintenance
- inventory management.

Maintenance and checks on the means and equipment before each commissioning, in order to ensure their safety and normal operation, will be carried out by the operator, who will record the activities carried out on specific forms.
Specific maintenance programmes will be set up to ensure that the preventive maintenance indicated in the operation and maintenance manuals is carried out. The Landfill Manager will be the guarantor of compliance with these programmes; he will also check the work of suppliers and archive the records of all maintenance carried out.

7.6.1 Excavator

An excavator is a machine that consists of a chassis, a boom, a house and a bucket (shovel) and is mainly used for inconsistent ground handling operations. They can be used in mining (mines or quarries, especially open pits) in situations where the material is relatively easy to break up. Another application is in civil construction (for foundation excavation or trench construction) or in the construction of controlled landfills, dredging of rivers, ponds, lakes.
The choice of excavator (in terms of type, size, performance) always depends on the application area.
The machine is operated by a professional figure, the excavator operator, who sits in a cabin, called the house, from where he can control the movements of the excavator and its boom. The excavator is driven by three pumps, which are driven by a supercharged diesel engine:

- Two pumps drive the boom, bucket and track movements
- A pump drives the excavator controls

With regard to the chassis, the excavator can be mounted on a wheeled, tracked, skid-mounted or articulated chassis. The choice of chassis depends on the design and logistics choices of the site under consideration, in fact it depends on the load capacity, the speed and frequency of movement required, the operations to be performed, the size of the excavator, and the productivity to be achieved.
The crawler excavator is used in situations with soft, unstable soils. The crawler tracks distribute the excavator's own weight over a larger area than tyres, which allows for better load distribution and better overall stability conditions.
The bucket is a digging tool that can be classified into two categories: clamshell or dragline bucket. The clamshell bucket is used for digging deeper than the frame support level. It is lowered open at the point of excavation and is driven into the ground. The excavator then uses the controls in the cab to close the bucket to collect the material and withdraw it.
The drag bucket, on the other hand, is used when there is a need to reach greater horizontal distances. It collects the material during the dragging movement driven by a rope system and is widely used in dredging the seabed.
In addition to the bucket, other equipment such as rotary cutters, hydraulic hammers, can be attached as an end tool of the excavator arm, depending on the objective of the operation being performed.
Moving an excavator requires a truck and a trailer.

Fig. 59 - Use of an excavator on the construction site (Oggeri and Capozzo, 2022)

7.6.2 Dumper

The dumper is a motor vehicle equipped with a body that is used to transport materials, such as rubble, soil, debris, within the construction site area. The main components of this dumper are two: the body and the position from which the driver drives the vehicle.
The choice of dumper (in terms of weight, size, capacity and body size) depends on the site under consideration (road conditions, type of terrain) and the size and volume of the material to be transported. The criteria for selecting a dumper are:

- Versatility: A versatile dumper can allow for easy adaptation and transport in different terrains. If it is also equipped with a wide range of attachments, the convenience and ease of operation increases significantly.
- Handling: It is important to choose a dumper that can move smoothly over rough, uneven terrain and has smooth handling capabilities.
- Safety: In difficult conditions, it is important to ensure the stability and safety of the driver. It is mandatory to select a dumper that meets all the safety standards of the European directives.

Fig. 60 - Use of a dumper on the construction site

7.6.3 Crawler loader

The truck is an articulated lorry consisting of a driver's cab and a tipper body. It is used to transport bulk material required for the progress of work on the construction site. The choice of truck depends on the size and type of roads and tracks, and the volume and size of the material to be transported.

Fig. 61 - Crawler loader

7.6.4 Compactor

For a landfill to be profitable, it must utilise its capacity to the maximum, which means compacting the waste in the best possible way. This is why a compactor is used.
Compactors are heavy machinery that are used to reduce the volume of waste in a landfill by compacting it into smaller and denser bales or blocks. Compactors are typically used to compress and compact waste materials such as household trash, construction debris, and industrial waste. There are several types of compactors that are used in landfills, including stationary compactors, self-propelled compactors, and transfer station compactors.
The specific characteristics of a compactor for waste in a landfill will depend on the type of compactor and its intended use. Some common characteristics of compactors for waste in landfills include:

- Size and weight: Compactors are typically large and heavy machines, with weights ranging from several tons to over 100 tons.
- Compaction force: The compaction force is the amount of force that a compactor can apply to waste materials to compress and compact them. This is typically measured in pounds per square inch (psi).
- Compaction ratio: The compaction ratio is the ratio of the volume of waste before and after it is compacted. A higher compaction ratio means that the waste has been more effectively compacted and occupies less space in the landfill.
- Capacity: The capacity of a compactor refers to the amount of waste that it can hold and compact at one time.
- Fuel efficiency: Some compactors are designed to be more fuel efficient, which can help reduce the environmental impact of waste management and reduce operating costs.
- Safety features: Compactors may be equipped with safety features such as alarms, guards, and interlocks to help protect operators and prevent accidents. These characteristics help determine the efficiency and effectiveness of a compactor in a landfill setting.

Fig. 62 - Use of a waste compactor on the construction site

7.7 Environmental recovery

The environmental reclamation of a landfill aims to mitigate and eliminate the impacts caused by the establishment of the landfill and restore the landscape to its original state. The impacts caused by a landfill include soil occupation, release of leachate, and the formation of biogas.
This paragraph focuses on the management of greening the landfill, which is an essential part of environmental reclamation (fig. 63). The operations planned for the greening of the landfill are divided into

two time phases: the rooting period and the post-rooting period. During the rooting period, viable species will be irrigated as required, lawns will be mown monthly, and fertilized once a year during the vegetation period. Dry branches of tree species will be removed once a year, and shrubs will be pruned once a year during the vegetation period.
If some plants die, they will be replaced with native plants that are compatible with the climatic conditions of the site. If herbaceous plants fail to take root, sowing will be repeated where necessary. During the post-rooting period, interventions will consist of shearing meadow areas at least every three months during the vegetative period, fertilization, and fertilization if necessary. Dry branches of tree species will be removed once during the vegetative period, and shrubs will be pruned once during the vegetative period.
In conclusion, the management of the greening of the landfill is an essential part of environmental reclamation, and the operations must be carefully planned and executed in two time phases, namely the rooting period and the post-rooting period. The maintenance of the vegetation layer is critical to ensure that the landfill is properly restored to its original state.

Fig. 63 - Use of geosynthetics to promote slope greening (Oggeri and Capozzo, 2022)

8. Waste properties and settlements

8.1 Waste properties

In order to assess the stability of the landfill in general, through boundary-state analysis, and the integrity and proper functioning of the individual components (geosynthetics, geotextiles, geocomposites, etc.), the knowledge of the properties of the waste interacting with the components is necessary.
Recently, the collapse mechanisms affecting various landfills around the world have been studied, and the need to study the waste-landfill interaction has emerged. This aspect is very often analysed by means of numerical models, which, however, require the characteristics of the waste mass as input data. The purpose of these models is to evaluate, for example, potential post-peak instability surfaces along the interface between the linear (geosynthetic) component and the waste mass.
The behaviour of waste is very difficult to assess due to the great variability of materials that can be delivered within the same disposal site.
The heterogeneity of the waste delivered can be seen in the different sizes of the materials, which can vary from granular material such as soil, gravel to commonly used items made of plastic, rubber, paper. The amount of waste and the different sizes of materials that reach the landfill depend on a larger scale, on the habits and lifestyles of the residents of the areas from which the waste originates, the time of year, the geographical context, and population density. All this creates important differences in the inputs and subsequently in the stability scenarios within the landfill; therefore, it is very difficult to create a model that can be considered valid for all landfills regardless of geographical location. Given also the heterogeneity of the materials, it is not possible to obtain values with a very low variance of the properties of the materials of interest, but it is common to obtain ranges within which these values may vary. For each case under consideration, the evaluation of other characteristics will be very important in order to choose an appropriate value for the stability study.
The waste characteristics to be considered for an initial classification are:

- Shape of the components, which is useful for predicting the potential influence on the mechanical behaviour of the landfill complex
- Size of the components
- Constituent materials
- Deformability of materials
- Degradability and biodegradability of organic and non-organic waste components

To assess the mechanical behaviour of waste, tests must be carried out to evaluate material properties. Tests should be conducted considering the material under real, undisturbed conditions, but this is not possible as laboratory tests are conducted under rearranged conditions within the equipment.
In general, within a landfill site, waste may be present, simultaneously, in the three different states of matter: solid, liquid, gaseous. This also makes the study of waste subsidence within the landfill very complicated as different mechanical behaviours come into play.
The most important characteristics to consider are:

- Unit volume weight
- Compressibility
- Lateral stiffness
- Shear resistance
- Hydraulic properties
- Void index
- Horizontal in situ stress

8.1.1 Unit volume weight

The volumic weight of waste within the landfill has a very high variability. Given the heterogeneity of waste, it is very common to encounter situations in which the volumic weight has an important spatial variability within the same disposal site and a temporal variability, which will be discussed in more detail below. Furthermore, within a landfill site, waste may be present, simultaneously, in the three different states of matter: solid, liquid, gaseous. This also makes estimating the volumetric weight of the waste inside the landfill very complicated, as different mechanical and degradation behaviours get involved, resulting in weight variation.
Due to the presence of moisture in most of the waste, it is possible to define the density weight through three different values: dry unit weight, bulk unit weight (partially saturated) and saturated unit weight. The moisture content of waste depends on climatic conditions, operating conditions, the composition of the waste (organic waste has a higher water content than plastic) and the presence of organic matter in the waste. Under saturated conditions, each component capable of absorbing water will increase its own weight, thus increasing the specific weight of the waste. The absorbed water must be considered as it will be one of the main components of the leachate. Older waste, after having undergone degradation processes resulting in the expulsion of interstitial water, will have a higher bulk unit weight than poorly compacted, fresh waste. In most cases, the bulk unit weight is considered, which can be estimated through laboratory tests or field tests. The two tests are different and depending on the case study and the available resources, the most appropriate type of test is evaluated in each situation.

The weight by volume depends on the composition of the waste, the daily cover (if present, made of granular material), the degree of compaction during delivery, the climatic conditions, the degree of decomposition (biological or chemical) and the depth at which the waste is located.
This property, as stated earlier, has a temporal variability that depends on the degree of decomposition, climatic conditions and depth. In fact, as the age of the waste increases, there is the increase of:

- the degree of compaction: due to the vertical tension generated by other deposits, by the final covering, and by the passage of mechanical equipment. The compression generated by these processes leads to a progressive reduction of the voids index, which varies according to the type of load generated by the contributions, the type of compaction means used and their frequency of passage
- the degree of degradation: due to the reduction in mass, change in shape and mechanical parameters as a result of chemical and biological degradation: as degradation increases, the void index decreases, generating a reduction in volume, which leads to an increase in the volume weight.

The volume weight is used for the calculation of vertical and horizontal stresses, which are necessary in the calculation of the subsidence of the waste within the landfill body.

8.1.2 Compressibility

The total subsidence of the waste mass can be calculated by means of topographical surveys, successive in time, of the same points. The individual contributions of total subsidence, on the other hand, are more complicated to calculate, as each waste layer would have to be calculated:

- The contribution of initial compression
- The contribution of primary compression (failure due to own weight)
- The contribution of secondary compression (subsidence due to chemical-biological degradation)

- The contribution of the daily cover, the laying of additional waste layers and the laying of the final cover

To simplify the discussion, only the primary and secondary compression terms have been considered for the calculation of the total compression.

9.1.3 Shear resistance

The shear resistance of a mass of waste is calculated by applying Coulomb's method. To do this, it is therefore important to know certain mechanical parameters of the waste such as:

- Cohesion
- Shear resistance angle

As far as cohesion is concerned, it is very important to be careful when estimating the value as an unreal cohesion could underestimate the risk of collapse of the structure. Cohesion can be considered apparent or real and can change spatially and temporally within a landfill. For this reason, it is necessary to assess the stability conditions in the short, medium and long term.
Coulomb's method, moreover, as can be seen from its formulation, shows that there is an increase in shear strength because of an increase in the tensional state, and thus also in the depth at which the waste is placed.

There are two main methods for calculating shear strength: triaxial compression tests and direct shear tests. The latter is more reliable than the triaxial compression test, but there is still an underlying problem due to the impossibility of testing a sample under real, undisturbed conditions. In order to obtain optimal values and then incorporate them into a numerical model for stability assessment, it is also possible to consider values obtained from field tests and back analysis of landfill slope failures.
Through direct shear tests, it is possible to obtain a wide range of possible shear res values, which also depend on the tensional state. possible shear resistance values, which also depend on the tensional state. The shear resistance value is used to assess the stability of linear components of the landfill (geosynthetics). Failure mechanisms within landfills are associated with shear surfaces within the waste mass, soil and along the interfaces of lining and/or cover materials.
Some typical values of shear strength parameters are reported in the following table for various types of waste, referring to different studies:

Table 2 – Resistance parameters for different type of waste

Waste types	**Shear strength**		**Bybliografical references**
	Friction angle (°)	**Cohesion (kPa)**	
Fresh MSW non compacted	38-42	< 5	*Schmutz B. and Morzier C. (1986)*
Fresh MSW	38-40	30-50	*Jessberger, H.L. and Kockel (1991)*
Decomposed MSW	19-24	16-32	
Decomposed MSW	17-23	0-10	
Decomposed MSW	23-27	5--15	*Zoino, W.S. (1974)*
Bales of MSW	15-25	70	*Fang, H.Y. (1977)*
Ashes of MSW	37-40	0	*Schmutz B. and Morzier C. (1986)*
Decomposed sludges from water purification	15-25	0	*Jessberger, H.L. and Kockel (1991)*

Undrained sludges from water purification	21	8	
Drained sludges from water purification	35	0	

8.1.4 Hydraulic properties

Hydraulic conductivity is very important for correctly sizing geosynthetics and for understanding the influence of leachate circulation within the landfill waste body. This parameter is anisotropic within a landfill for two reasons:

- Use of daily cover
- Arrangement of the waste inside a landfill

The horizontal hydraulic conductivity (K_h) is much higher than the vertical component (K_v). It also depends on the level of stress applied to the waste: it has been shown by studies that the K value can vary by up to 3 orders of magnitude depending on the age of the waste and its state of compression.
Hydraulic conductivity governs the uptake of leachate, which usually accumulates along low-permeability surfaces. It is an important parameter for sizing geosynthetics for filtration, drainage and impermeability.
K is measured by means of direct tests on fresh waste samples through the following relation (Fang et al., 1977)

$$K = 10^{-\frac{\rho+0.2}{0.425}}$$

Where ρ is the bulk density (t/m^3), K is the coefficient of permeability (cm/s), 0.2 and 0.425 are correlation parameters.

8.1.5 Lateral stiffness

Knowledge of this parameter is necessary to assess the interaction between waste and the lining system along the side slopes of the landfill. In order to assess this parameter, tests were carried out at different depths and on municipal solid waste (MSW) with different ages. Elastic parameters such as Shear Modulus (G), Young's modulus (E) and Poisson's ratio (ν) were used to evaluate how the material responds following the application of a tensional state.

The parameters are related, as the following equation shows:

$$G = \frac{E}{2(1+\nu)}$$

To evaluate this parameter, field tests are required as different delivery methods, waste type and waste depth influence this value. Laboratory tests on reworked samples are not representative as they do not evaluate physical properties such as density and stress state; instead, a laboratory test such as the pressure test is a standard technique used for soils and rocks.

8.1.6 Void index

A waste mass can be likened to a porous mass in which materials in the 3 different states of matter coexist. As far as the solid fraction is concerned, the materials delivered to landfills are analysed in the paragraph 7.5. As stated earlier, the fluid and gas phase are the most complicated components to assess. Their presence and movement depend on the porosity of the mass, which depends mainly on the grain distribution. It is possible to measure the porosity of the mass of waste such as:

$$n = \frac{V_v}{V_t}$$

Where V_v is the volume of the voids and V_t is the total volume of the mass.
It is much more common to characterise a waste mass not by porosity, but by the voids index:

$$e = \frac{V_v}{V_s}$$

Dove V_s represents the solid volume of the mass.

8.1.7 Horizontal in situ stress

The horizontal in situ stress is mandatory to evaluate the stability of both shallow and slope side lining system components and the performance of components, as biogas and leachate capture tubes, present in the landfill. The horizontal in situ stress can be defined as:

$$\sigma_h' = K_0 \sigma_v'$$

Where σ_v' is the vertical stress and K_0 is the coefficient of each pressure at rest.
This parameter can be evaluated through laboratory measurements, which can give an indication of possible field behaviour considering the difficulties of replicating field conditions.
From past studies (Landva et. Al., 2000), it has been obtained the typical value of K_0 according to the type of waste: for fresh MSW K_0 ranges between 0.35 and 0.4, while if there is less reinforcing material K_0 increase towards values close to 0.5.
Furthermore, if the reinforcing materials is damaged because of degradation processes, the horizontal in situ stresses will increase with time but field measurements are missing to confirm this behaviour.

8.2 Settlements

The assessment of settlements is also important because it can affect the design and the stability of landfill in general and/or in single components (drains, covers, barriers). The capacity of storage, costs and feasibility will be influenced too. Excessive settlement, for example, leads to ponding, fractures of components. This means that there can be an increase of leachate production, which can affect the stability of the landfill.
In order to correctly assess the settlement of these materials, the following factors must be considered

- initial density or void index of the waste
- amount of degradable and biodegradable material
- compaction method
- landfill sequence and methodology
- stress history and overburden pressure (relevant in the case of a landfill trestle expansion)
- leachate quantity and fluctuation
- environmental factors (water content, temperature, oxygen content, presence of biogas)

The main failure mechanisms that can affect waste are:

- mechanical compression: densification, distortion, reorientation, due to self-weight. It is a phase similar to soil consolidation
- Relocation of fine particles into voids and cavities within the waste mass. Contribution very difficult to assess and analyse

- Chemical-physical changes in the waste, involving deterioration and volumetric reduction due to redox reactions or combustion
- Biochemical changes, due to biodegradation by microorganisms through aerobic and anaerobic mechanisms.

The extent of subsidence is difficult to predict, but the trend is generally standard. There is considerable initial subsidence in the first 2-4 months after waste placement, followed by subsidence over a longer period of time. This second contribution is related to the secondary compression of the waste but is smaller in magnitude. In spatial terms, however, it increases with increasing depth.
In general, the settlement of inert solid waste in landfills is a slow process that can take many years to complete. As the waste settles, the weight of the waste causes it to compress and the volume of the waste decreases. This process is known as settlement. The rate of settlement depends on a variety of factors, including the type and composition of the waste, the density of the waste, and the moisture content of the waste.
It is important to monitor the settlement of waste in a landfill to ensure that the landfill is being used efficiently and that the waste is not causing any environmental issues. This can be done using monitoring wells, settlement plates, and other monitoring technologies.

On a temporal level, the trend of vertical subsidence over time of municipal solid waste or assimilable waste can be broken down into its different components, related to the phenomena at its origin, similar to what is conducted in the geotechnical analysis of general soil subsidence. They are listed in chronological order of occurrence:
- Initial subsidence (S_i)*:* purely mechanical, due to self-weight, rearrangement of the structure, overpressures due to the use of compacting rollers. In general, condition in which the effective stresses vary. From literature, it can be estimated that due to self-weight, the final failure of the waste mass is 10-40% of the thickness of the landfill. The range depends on the type of waste placed in the landfill and the degree of compaction of the deposited materials.
- Primary subsidence (S_p)*:* subsidence that occurs at the end of the initial subsidence, also due to the expulsion of water and liquids from the deposited materials
- Secondary subsidence (S_s)*:* subsidence that sets in on larger time scales than the first two, and considers the biochemical and biological degradation of the waste. While the first two are exhausted over relatively short time scales, secondary subsidence is exhausted over longer time scales but its magnitude is smaller than the first two.

$$S_{tot} = S_i + S_p + S_s$$

The total subsidence S_{tot} can also be expressed according to the following expression:

$$\Delta H = H_0 * (\frac{\Delta\sigma}{E_R} + C'R * Log_{10}\left(\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right) + C'_\alpha * Log_{10}\left(\frac{t_2}{t_i}\right))$$

Where:
- ΔH is the total subsidence of the waste;
- H is the initial thickness of the waste layer;
- σ_0 is the average vertical tension of the waste at the initial moment;
- $\Delta\sigma$ is the overload-induced increase in vertical tension;
- E_R is the modulus of deformation of the waste;
- $C'R$ is the primary compression coefficient;
- C'_α is the secondary compression coefficient;

- t_i -is the time (in days) for exhaustion of the initial phase failure (90 days in the case study);
- t_2 is the time (in days) against which subsidence is to be calculated.

The contribution of primary (ΔHp) and secondary subsidence (ΔHs) are presented below:

$$\Delta Hp = C'R \cdot \frac{H_0}{1+e_0} \cdot \log\left(\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right) = C'R \cdot H_0 \cdot \log\left(\frac{\sigma_0 + \Delta\sigma}{\sigma_0}\right)$$

$$\Delta Hs = C_\alpha \cdot \frac{H_0}{1+e_0} \cdot \log\left(\frac{t_2}{t_i}\right) = C'_\alpha \cdot H_p \cdot \log\left(\frac{t_2}{t_i}\right)$$

Where H_p is the height of the layer considered at the end of primary consolidation.
Because a standard consolidation test method for solid waste has not yet been developed, the selection of waste compression indices is mainly based on experience and back analysis.
Generally, the initial void ratio of municipal solid waste placed in a landfill after compaction is quite difficult to determine, and so the value of $C'R$ and C'_α can't be estimated readily for settlement analysis. To calculate them, an alternative process could be using modified coefficients: $C'R$ and C'_α

- $C'R$ ranges from 0.180 to 0.325
- C'_α ranges from 0.060 to 0.094

The variability of these parameters depends on the age of the waste: in the case of a layer that has just been landfilled, the void index will be considered with the highest value; in the case of a layer that has been landfilled earlier, for example in the lower parts of the waste heap, a lower value will be considered.

8.3 Study case application

For the determination of the compressibility coefficients, there is currently no univocal model used for calculating the settlements of municipal solid waste; therefore, a model used for soils will be adapted and the material parameters will be modified to include parameters that better describe the properties of municipal solid waste.
In this study, the model of Sowers (1973), a model based on soil mechanics, was taken as a reference. In this method, only vertical deformation is considered, which is why the verticals obtained from the study of the landfill under consideration can be properly analysed. In this model, subsidence is subdivided into primary and secondary subsidence by separating the instantaneous subsidence due to the laying of waste.

The study of subsidence was conducted by making some initial simplifying assumptions:

1) The vertical considered was constructed from topographical surveys and sections, taking into account the difference between topographical height above sea level and height above sea level of the base of the landfill, and considering the validity of the datum within a 5m circle of the chosen point (based on the size of the compactors used in the landfill)
2) The instantaneous subsidence and the biodegradation-related component of subsidence were not considered, as only primary and secondary subsidence were focused on.
3) The role of moisture balance and hydrodynamic effects in the compression of partially saturated materials is not very clear, so it is neglected.
4) The bale thickness, in general, was entered considering the landfill delivery document to realistically represent the situation described.

5) The analytical model took into account the placement of daily cover, assumed by 40 cm of inert material needed to reshape the surface before the conferment of the next layer.
6) The analytical model does not take into account operations such as compaction through rollers and changes in geometry.
7) The pumping of biogas and leachate, components acting on primary and secondary subsidence, was not considered. The continuous pumping of biogas and leachate promote the biodegradation of waste and thus tend to increase the rate of secondary subsidence.
8) The exhaustion of the primary failure mechanism was considered after 90 days;

To show the validity of the assumptions made and the ranges of primary, secondary compressibility and volumetric weight of waste, the evolution of landfill subsidence at two different points for the two different batches is shown by studying the evolution of the vertical behaviour.

The points taken as reference refer to the positions shown in the figure 64.

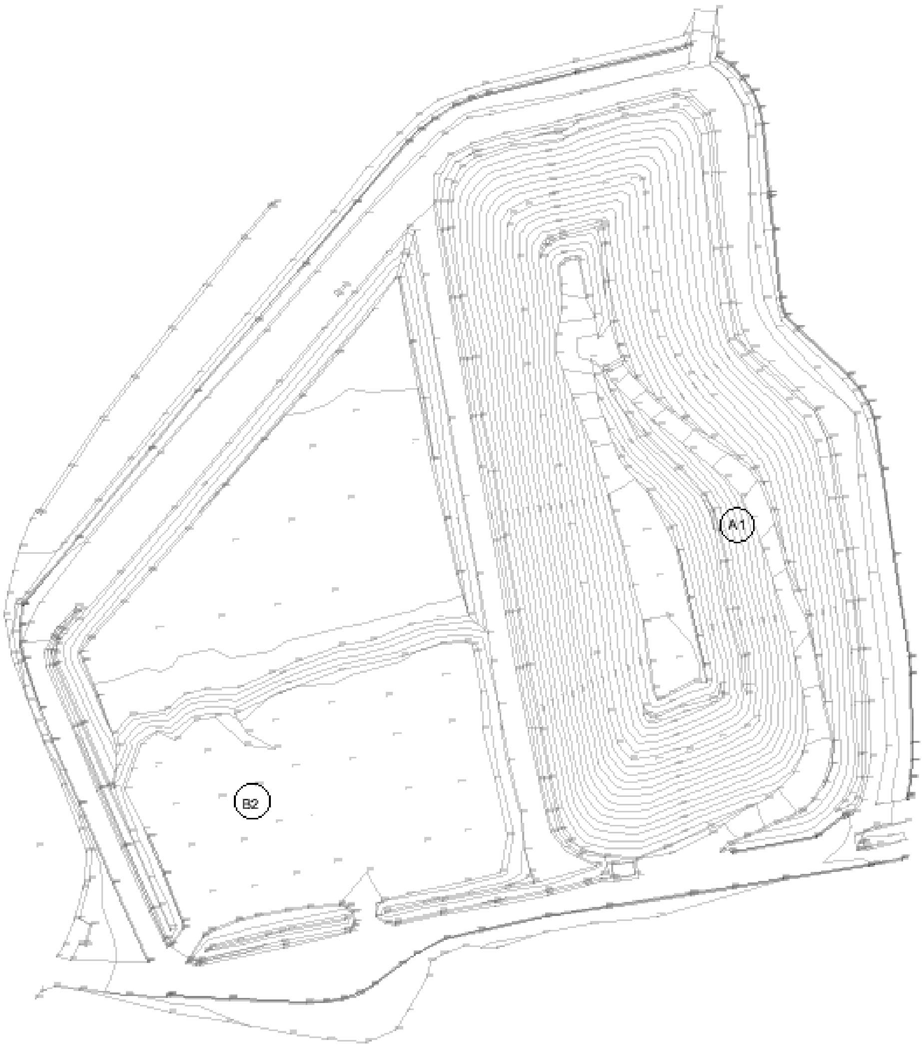


Fig. 64 - Location of the examined verticals

The code assigned to the analysed verticals consists of a letter and a number, with the latter representing the batch in which the vertical is analysed.

8.3.1 Results

For the calculation of the subsidence values, an Excel sheet was created in which the values of specific weight of the waste, primary compressibility coefficient and secondary compressibility coefficient were varied in relation to the age of the waste and the evolution of the landfill. The ranges used are shown below, following consultations from literature (specific weight of waste) and back analysis on the case study analysed (compressibility coefficients)

Table 3 – Values of the parameters of waste obtained

	Range
C'R	0.180 - 0. 325
C'α	0.060 - 0.094
γ_{RSU} (kN/mc)	7.7 - 10.5

The specific weight of the waste used within the analytical model was considered with reference to the table below:

Table 4 – Values of unit volume weight of waste

γ_{RSU}	Poor compaction	Moderate compaction	Good compaction
Range (kN/mc)	3.0 - 9.0	5.0 - 7.8	8.8 - 10.5
Average (kN/mc)	5.3	7.0	9.6

The primary and secondary compressibility coefficient of waste used within the analytical model was considered with reference to the table below:

Table 5 – Values of primary and secondary consolidation coefficients

	Range
C'R	0.180 - 0.325
C'α	0.060 - 0.094

Within the proposed ranges, intermediate values were also used to better discriminate the situation in relation to the age of waste. Locally, different values of primary and secondary consolidation coefficients were considered in the two batches.
For the study of this vertical, the intermediate values of the ranges for the primary and secondary compressibility coefficients were taken by considering 2.5% and 1.5% of the minimum value respectively as the difference between two consecutive values.

The graphics of the study conducted will be shown below with their values. The values represented in the graph concern:

- H_{top}: quota obtained from topographical surveys (often at six-monthly intervals), referring to the difference between quota obtained from topographical beats and average quota of the landfill bottom
- H_{mod}: quota obtained from the analytical model created using Excel software
- $H_{mod,max}$: Value equal to the topographic quota plus 5% of the topographic quota.

- $H_{mod,min}$: Value equal to the topographic quota from which 5% of the topographic quota is subtracted.

The maximum and minimum limits were introduced to assess the reliability of the constructed model.
The results of the two verticals analysed are presented below.

Vertical “B2”

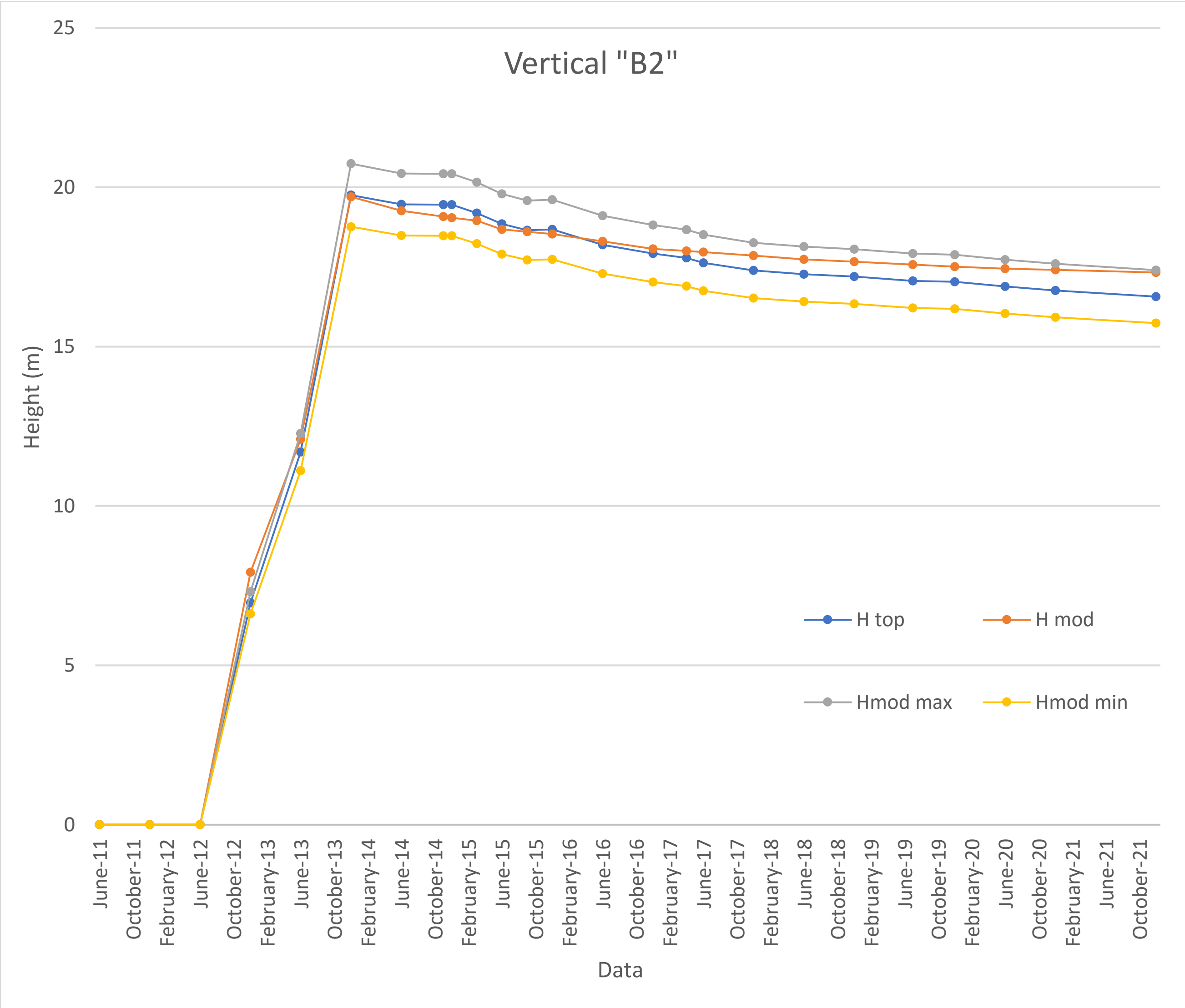


Fig. 65 - Variation in the elevation of waste within the landfill body for the vertical B2

For the study of this vertical, the intermediate values of the ranges for the primary and secondary compressibility coefficients were taken by considering 3% and 2.2% of the minimum value respectively as the difference between two consecutive values.

Table 6 - Values of the parameters of waste obtained for vertical B2

	Range
C'R	0.180 - 0.250
C'α	0.060 - 0.077
γ_{RSU} (kN/mc)	7.7 - 10.5

The extent of subsidence is shown in the table:

Table 7 – Subsidence of vertical B2

Subsidence B2	Value
S_{tot} (m)	6.9262
S_p (m)	4.4148
S_s (m)	2.0302

Vertical "A1"

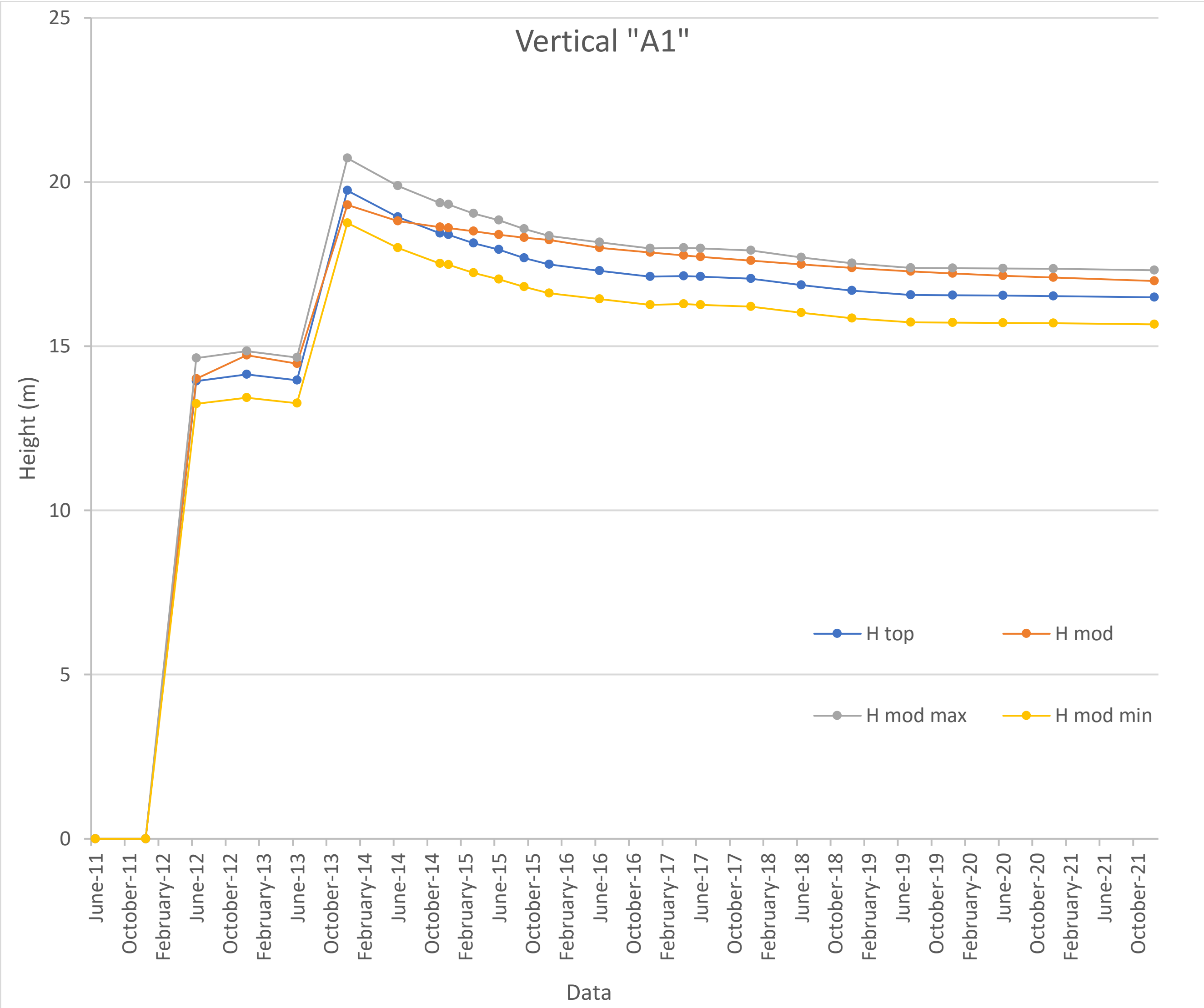


Fig. 66 - Variation in the elevation of waste within the landfill body for the vertical A1

For the study of this vertical, the intermediate values of the ranges for the primary and secondary compressibility coefficients were taken by considering 3% and 1.5% of the minimum value respectively as the difference between two consecutive values.

Table 8 – Parameters of waste for vertical A1

Parameters A1	Range
C'R	0.250 - 0.340
C'α	0.080 - 0.094
γ_{RSU} (kN/mc)	7.7 - 10.5

The extent of subsidence is shown in the table:

Table 9 – Subsidence of vertical A1

Subsidence A1	Value
S_{tot} (m)	8.2856
S_p (m)	6.0362
S_s (m)	2.2845

8.3.2 Discussion of the results

As we can see from the graphs, both curves obtained from the processed analytical model fall within the range of accuracy achieved, between the two extremes represented by the topographic elevation plus or minus 5% of it.

In addition, it can be noticed that after the end of the last contributions, the trends of the model share and the topographic share maintain a very similar long-term trend but deviate by a certain value. This deviation can be justified by the presence of mechanical creep mechanisms and organic matter biodegradation during secondary subsidence, as can be seen in the figure 67, and the failure to analyse the contribution due to the drainage of biogas and leachate.

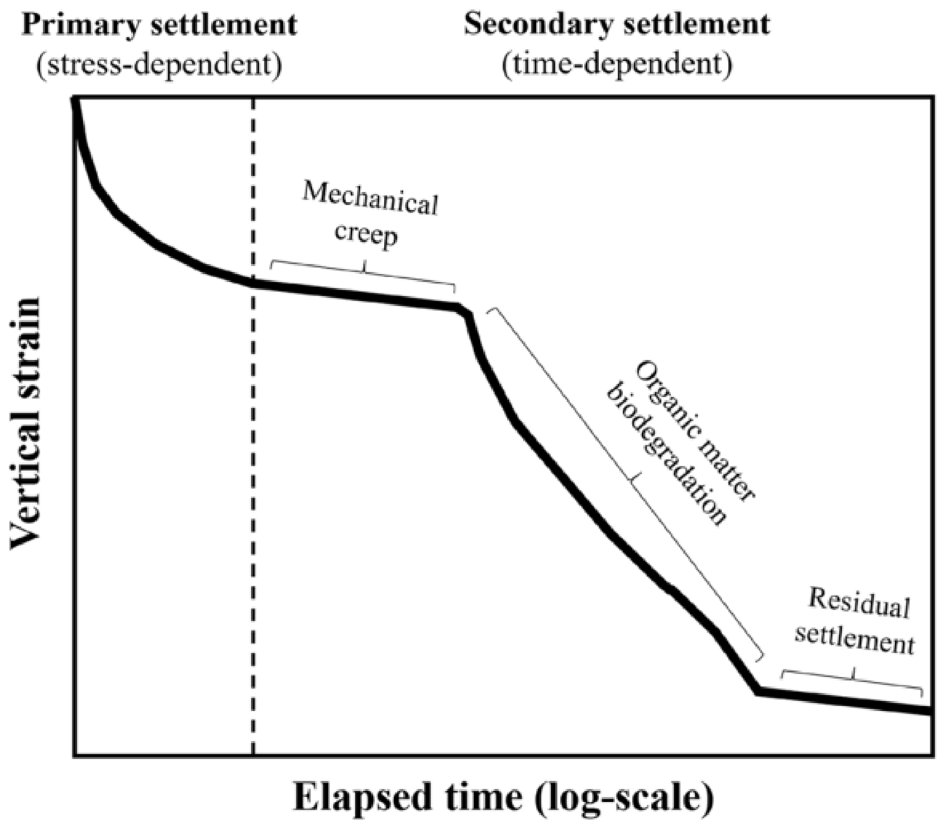


Fig. 67 – Principal components of settlements (Seok and Soo, 2022)

On the other hand, the two curves are not completely overlapping at the end of the inputs because the contribution of the immediate subsidence and the contribution of mechanical creep to the remodelling of the waste profile was not taken into account.

8.3.4 Sensitivity analysis

A sensitivity analysis was also carried out to study the variability of the results when varying the primary consolidation coefficient (C'R) and secondary consolidation coefficient C'_{α} of the waste.
The developed scenarios are presented below:

1° Scenario
In this scenario, the minimum value of the primary consolidation coefficient was increased by 3% and as a result all values increased. The ranges for the two verticals in this scenario are reported below:

Table 10 – Range of values used in the Scenario 1 for the vertical B2

B2	Range
C'R	0.185 - 0.257
C'α	0.060 - 0.077
γ_{RSU} (kN/mc)	7.7 - 10.5

Table 11 - Range of values used in the Scenario 1 for the vertical A1

A1	Range
C'R	0.25 - 0.350
C'α	0.080 - 0.094
γ_{RSU} (kN/mc)	7.7 - 10.5

The graphical results are shown below.

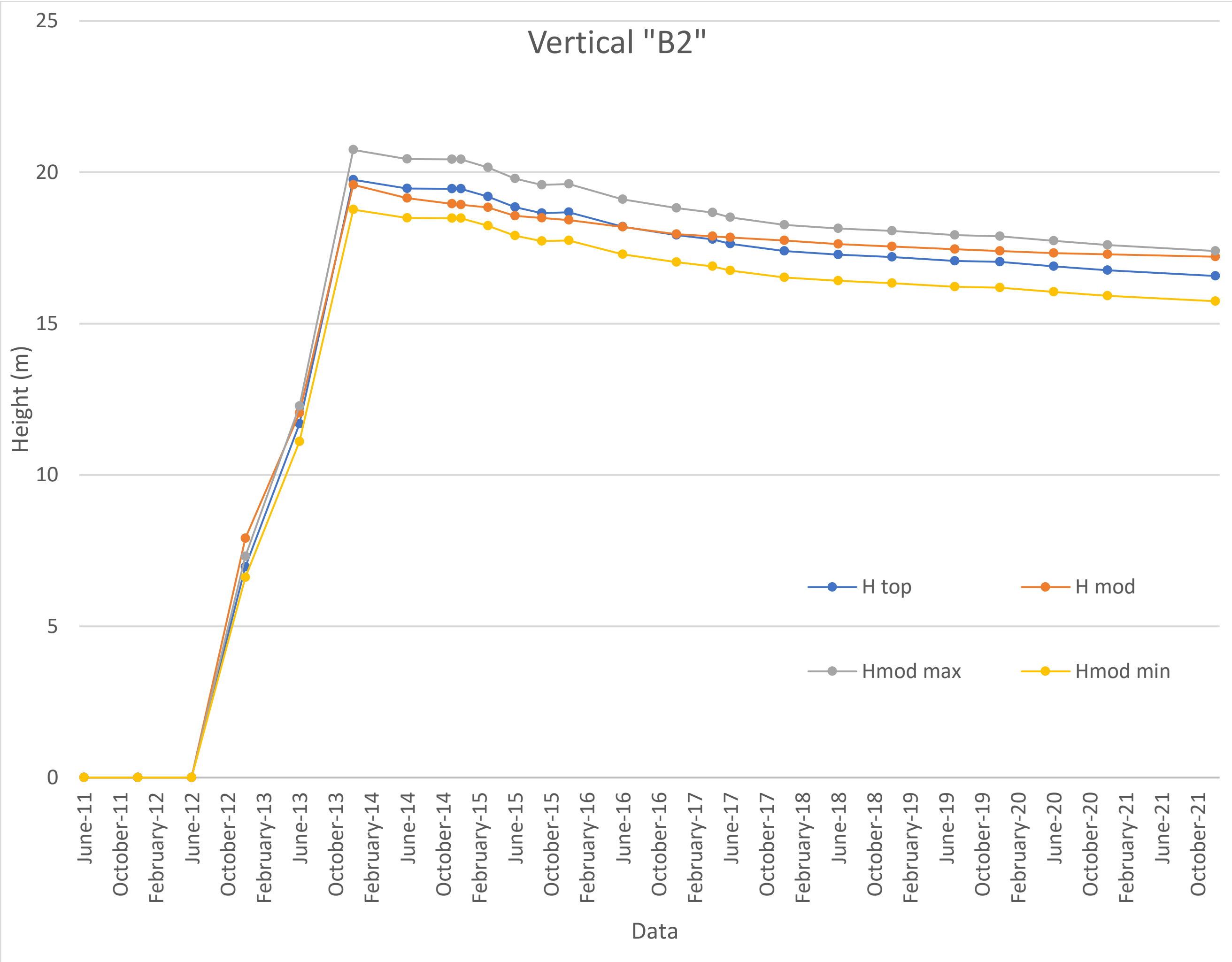


Fig. 68 - Variation in the elevation of waste within the landfill body for the vertical B2 in Scenario 1

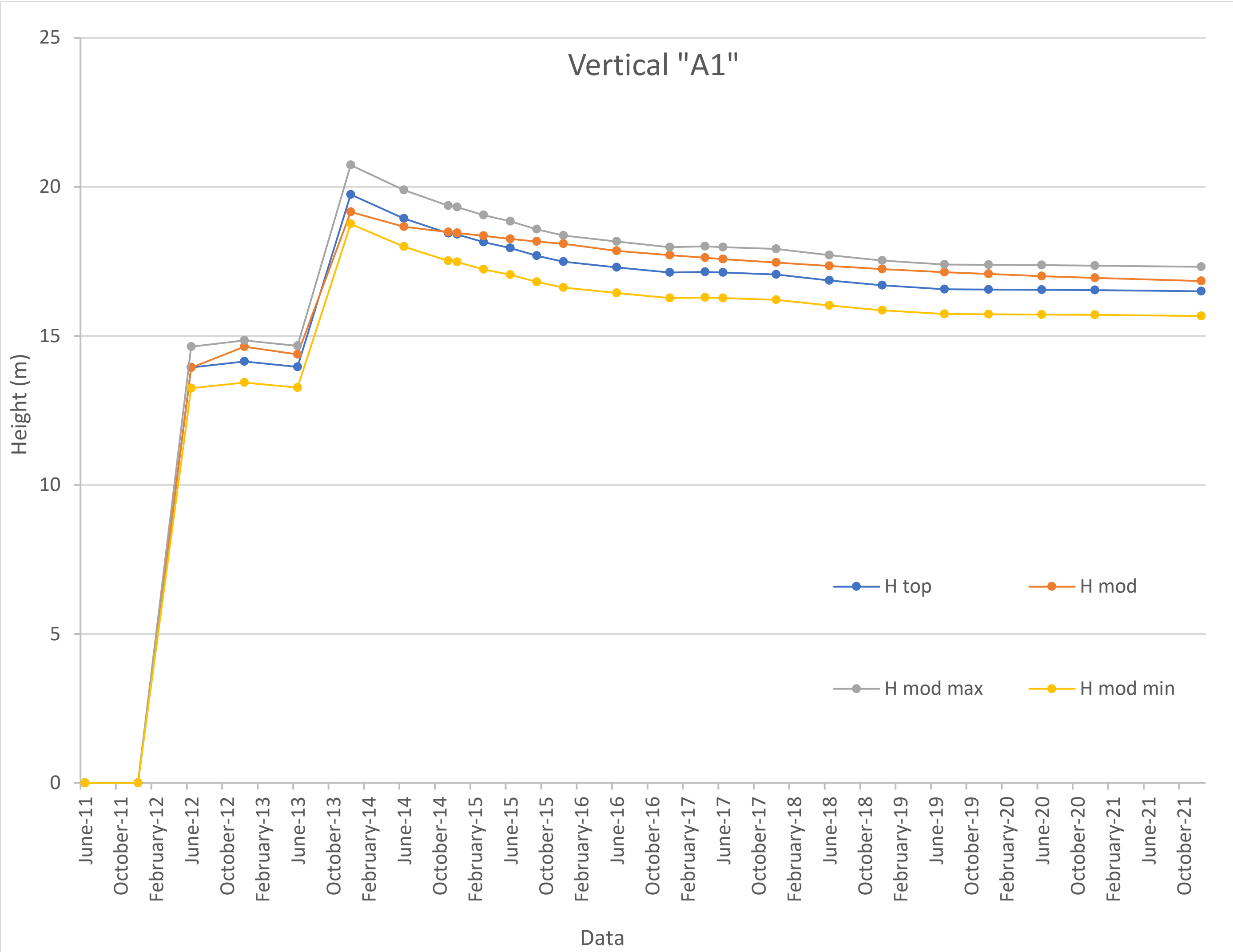


Fig. 69 - Variation in the elevation of waste within the landfill body for the vertical A1 in Scenario 1

As can be seen, the validity of the module is confirmed by the fact that the curve for the dimensions obtained from the model are within the validity interval plotted.

2° Scenario

In this scenario, the minimum value of the primary consolidation coefficient was decreased by 3% and as a result all values increased. The ranges for the two verticals in this scenario are reported below:

Table 12 - Range of values used in the Scenario 2 for the vertical B2

B2	Range
C'R	0.175 - 0.243
C'α	0.060 - 0.077
γ_{RSU} (kN/mc)	7.7 - 10.5

Table 13 - Range of values used in the Scenario 2 for the vertical A1

A1	Range
C'R	0.243 - 0.330
C'α	0.080 - 0.094
γ_{RSU} (kN/mc)	7.7 - 10.5

The graphical results are shown below.

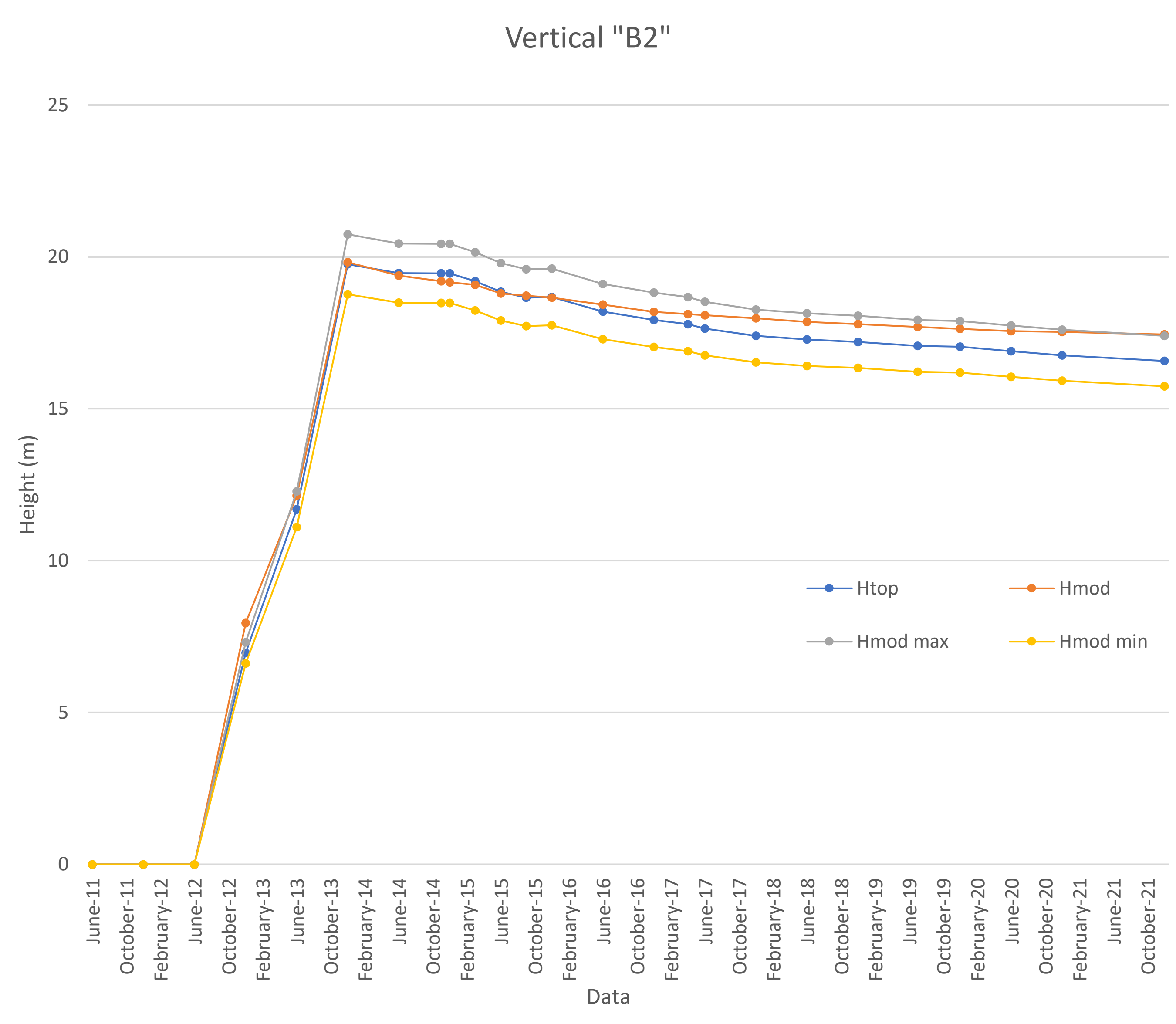


Fig. 70 - Variation in the elevation of waste within the landfill body for the vertical B2 in Scenario 2

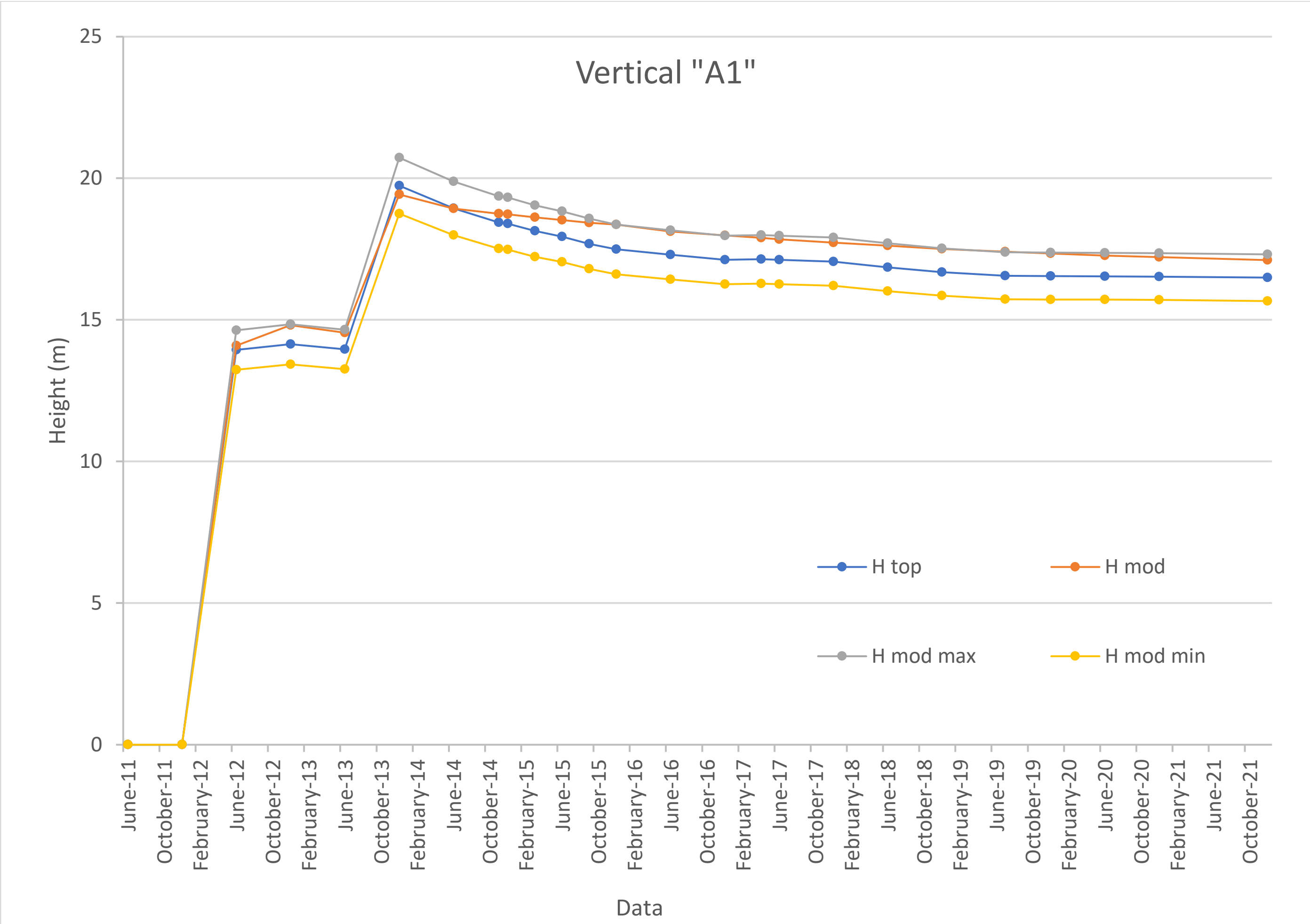


Fig. 71 - Variation in the elevation of waste within the landfill body for the vertical A1 in Scenario 2

As can be seen, the validity of the module is confirmed by the fact that the curve for the dimensions obtained from the model are within the validity interval plotted.

3° Scenario

In this scenario, the minimum value of the secondary consolidation coefficient was increased by 5% and as a result all values increased. The ranges for the two verticals in this scenario are reported below:

Table 14 - Range of values used in the Scenario 3 for the vertical B2

B2	**Range**
C'R	0.180 - 0.250
C'α	0.063 - 0.081
γ_{RSU} (kN/mc)	7.7 - 10.5

Table 15 - Range of values used in the Scenario 3 for the vertical A1

A1	**Range**
C'R	0.25 - 0. 325
C'α	0.084 - 0.099
γ_{RSU} (kN/mc)	7.7 - 10.5

The graphical results are shown below.

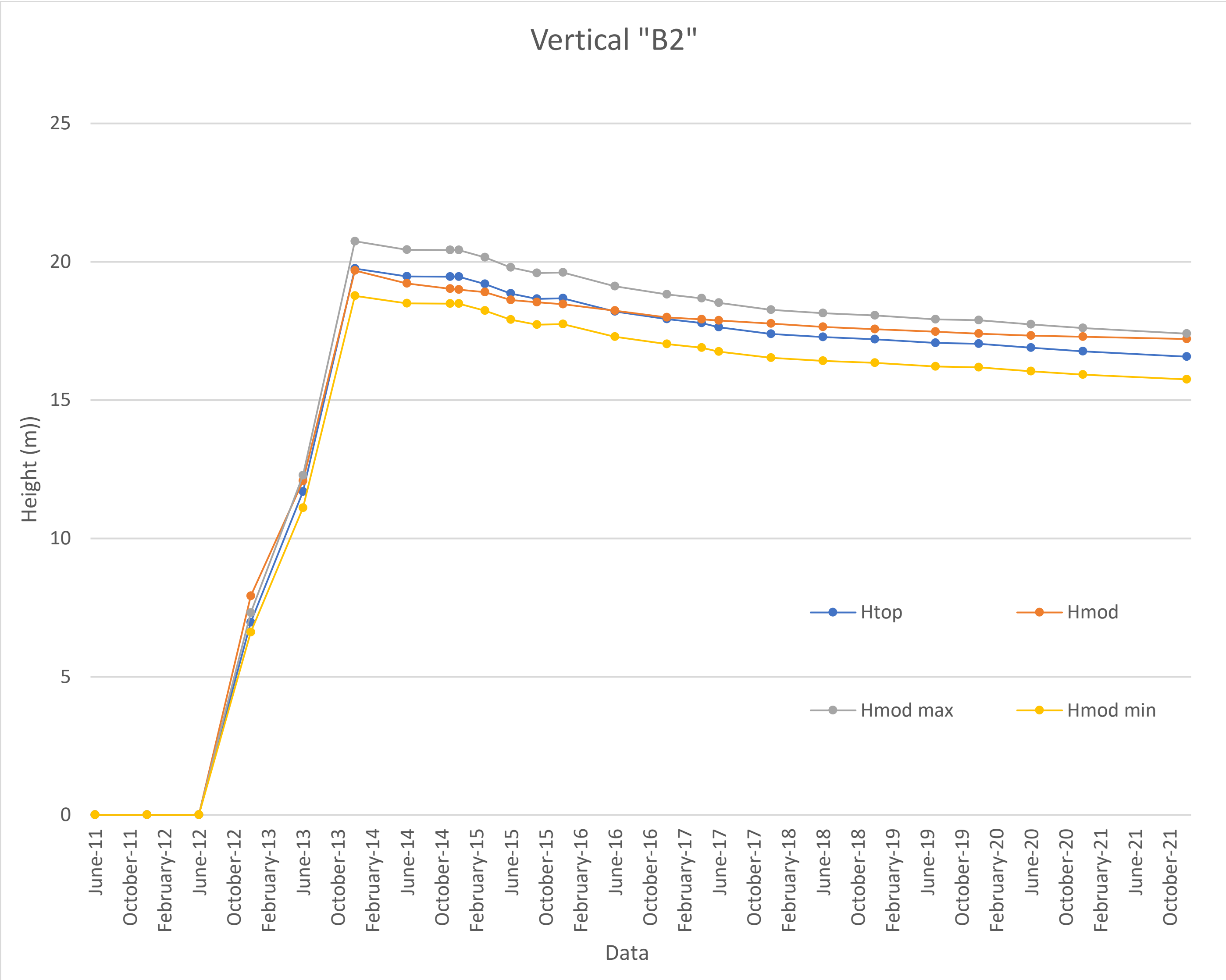


Fig. 72 - Variation in the elevation of waste within the landfill body for the vertical B2 in Scenario 3

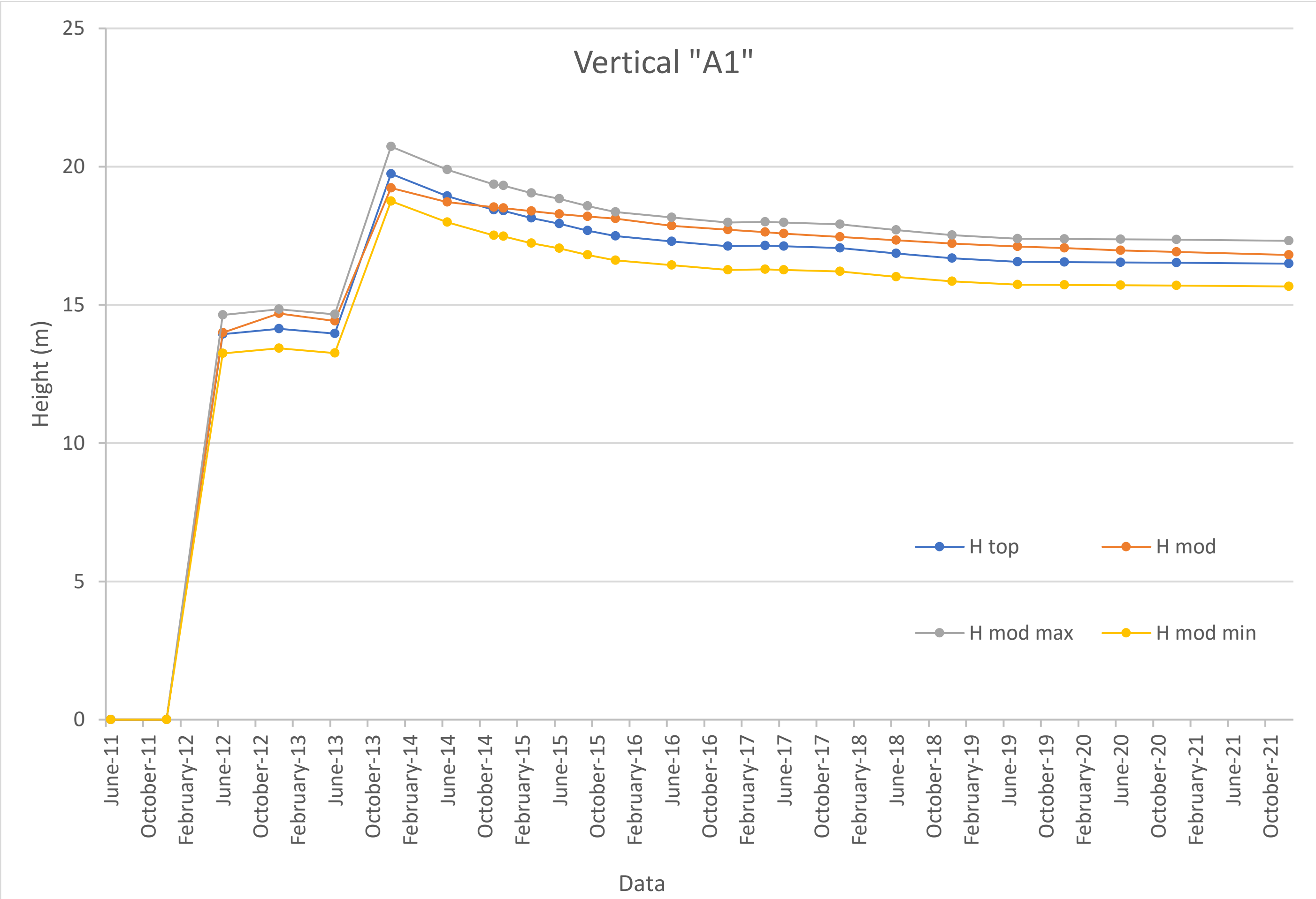


Fig. 73 - Variation in the elevation of waste within the landfill body for the vertical A1 in Scenario 3

As can be seen, the validity of the module is confirmed by the fact that the curve for the dimensions obtained from the model are within the validity interval plotted.

4° Scenario

In this scenario, the minimum value of the secondary consolidation coefficient was decreased by 5% and as a result all values increased. The ranges for the two verticals in this scenario are reported below:

Table 16 - Range of values used in the Scenario 4 for the vertical B2

B2	**Range**
C'R	0.180 - 0.250
C'α	0.057 - 0.073
γ_{RSU} (kN/mc)	7.7 - 10.5

Table 17 - Range of values used in the Scenario 4 for the vertical A1

A1	**Range**
C'R	0.25 - 0. 325
C'α	0.076 - 0.090
γ_{RSU} (kN/mc)	7.7 - 10.5

The graphical results are shown below.

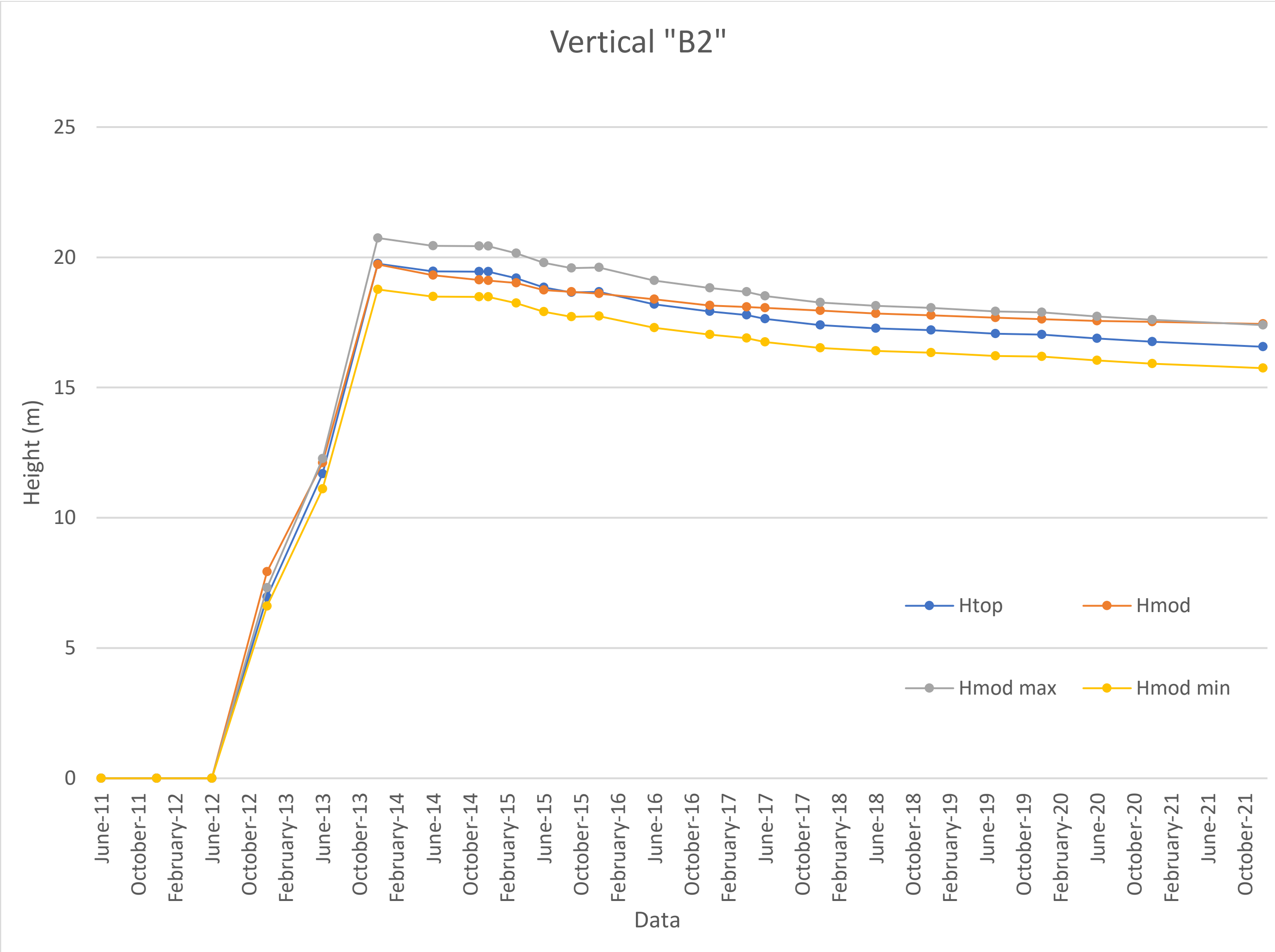


Fig. 74 - Variation in the elevation of waste within the landfill body for the vertical B2 in Scenario 4

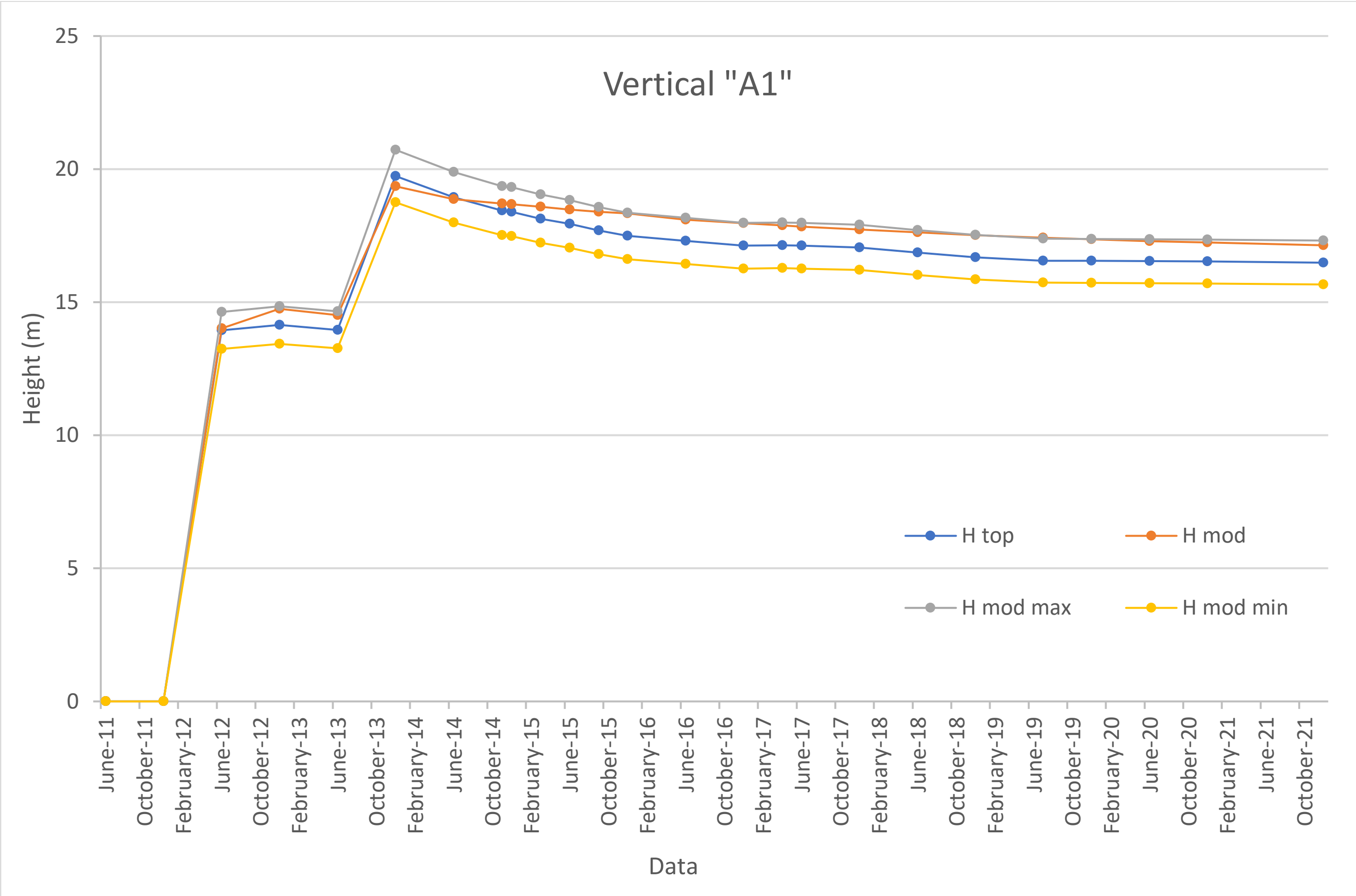


Fig. 75 - Variation in the elevation of waste within the landfill body for the vertical A1 in Scenario 4

It can be seen that on a graphical level, the result of the change in the primary compression coefficient leads to a vertical shift in the curve, while changing the secondary compression coefficient leads to a change in the inclination of the curve.
In general, with a variation of the 5% of the external parameters of the range there's not an important difference in the model. This means that the model, with the hypothesis explained before, proves to be accurate and also robust.
Other parameters that can be changed are the height of the single layers and the interval in which the unit weight ranges. These two aspects have not been evaluated in this study.

9. Possible new applications - vertical expansion of a landfill

The current environmental and legislative constraints, which mean that it is almost impossible to open new waste disposal sites, bring with them the need to find new engineering solutions to cope with the still considerable waste production. As we have seen, the use of geosynthetic reinforcements within the base lining and the topsoil makes it possible to correlate stability and maintenance of performance of large (in terms of volume) stockpiles. To cope with the constraints described above, geosynthetic reinforcements can also be used as a separation between an old landfill and a new waste disposal site, thus allowing a vertical expansion of the landfill while maintaining the stability and waterproofing of the site. The vertical expansion of a landfill is also called 'trestle expansion'.
A case study where this solution is applied is the landfill in Champigny-sur-Yonne (France). In order to correctly size, in geometrical and mechanical terms, the separation geosynthetic, the designers of the proposed case study landfill used the RAFAEL calculation method (from the French Renforcement des Assises Ferroviaire et Autoroutière con tre les Effondrements Localisés or reinforcement of road and railroad

foundations against localised sinkholes) in order to derive design parameters, tensile strength that the geosynthetic must meet and the correct installation procedure to avoid soil subsidence.

9.1 Example considered

The area of the site in question is 12,000 square metres and the height of the new delivery cell is approximately 22 metres. The installed geotextile reinforcement allows a maximum deformation of 3%. The new configuration allows the storage of 1.1 mln cubic metres of non-hazardous waste arranged in 4 cells (A, B, C, D).

The first operation to be carried out to allow the vertical expansion of the landfill is the preparation of the surface where the new mass of waste will rest. This consists of removing the topsoil until the top layer of waste is reached. After spreading granular material to level the surface, the geosynthetic liner was installed in the following sequence

- Geocomposite clay liner
- Smooth HDPE geomembrane with a thickness of 2 mm
- Geocomposite with drainage and protection functions

The waterproofing system is then topped with a geosynthetic to retain loads.

The subsidence of old waste can be associated with soil subsidence. The calculation method used to estimate the tensile strength of the geotextile reinforcement is based on Terzaghi's theory for calculating the vertical stress acting on the geotextile. This method, called RAFAEL, has been validated by experimental tests and is subject to continuous improvement and optimisation through subsequent research work.

The subsidence zone taken into account is defined by considering the differential settlement in a maximum space of two metres, depending on the type of waste delivered or the long-term degradation process of the waste delivered; therefore, the subsidence zone will be considered in the calculation as 2 m.

9.2 Calculation of parameters

The input parameters used in the software to define the waste are:

- density = 9 kN /m^3
- angle of friction = 18°
- cohesion = 0 kPa.

With regard to cohesion, the choice of a zero value is dictated by the risk of long-term loss of cohesion. The actual value of this parameter is 22 kPa.

With regard to geosynthetic reinforcement, the most important parameter to be entered is geosynthetic deflection. It is also essential to include the condition that this parameter, corresponding to the permissible deformation of the geosynthetic at the serviceability limit state, does not exceed 3%, since the deformation at the ultimate limit state of an HDPE geomembrane with a thickness of 2 mm is less than 6%.

The service life of the geosynthetic is 120 years, and in the calculation of the required tensile strength, the mechanism of long-term creep, damage during installation, and chemical degradation are taken into account through special factors. In addition, a safety factor on the tensile strength of the reinforcement based on ISO TR 20432 is also considered.

The other parameters relating to the geometry of the structure are given in the previous chapter in the description of the case study.

9.3 Calculation of the required tensile strength

The short-term tensile strength at maximum deformation is calculated using the RAFAEL method as follows:

$$R_t = R_{t;d} \cdot RF_{creep} \cdot RF_{damage} \cdot RF_{chemical} \cdot \gamma_{m;t}$$

Where

- RF_{creep} is the reduction factor to limit the deformation during the service life of the assembly. Its value may vary according to the type of analysis to be conducted: to investigate the ultimate limit state, a scenario in which there is physical failure of the product is considered; to investigate the serviceability limit state, the maximum creep elongation evaluated between the end of construction and service life is considered.
- RF_{damage} is the reduction factor that evaluates damage during installation due to the type of geosynthetics and site-related conditions, such as delivery methodology, anchoring, characteristics of backfill material and compacted layer.
- $RF_{chemical}$ s the reduction factor related to the chemical degradation of the geosynthetic and depends on the type of geosynthetic material and the site conditions in terms of temperature, pH, etc..
- $\gamma_{m;t}$ is the partial safety factor applied to the geosynthetic tensile strength. The value depends on the national regulations of the country in which it is applied.
- $R_{t;d}$ is the long-term value of the tensile strength of the geosynthetic, calculated after determining the vertical stress acting on the geosynthetic layer (σ_v) and considering the condition shown above regarding the allowable deformation (ε_{max}).
- D is the sinkhole diameter

This last parameter used can be determined through the equation:

$$R_{t;d} = \frac{\sigma_v D}{2}\sqrt{1 + \frac{1}{6\varepsilon_{max}}}$$

The choice of the geosynthetic to be used is based on compliance with the following condition:

$$R_{t;k} \geq R_t$$

9.4 Calculation of the anchoring and overlapping of the geotextile

Dimensionally, the anchoring and overlapping of the geotextile must be correctly defined in order to maintain a stable condition within the landfill at all times.
With regard to anchoring, the friction between soil and geosynthetic on the one hand, and between geosynthetic and geosynthetic on the other, are taken into account for the calculation.

It is necessary to determine the longitudinal overlap between the geotextiles and the anchoring at the edges of the geotextile. For the anchorage, soil/geosynthetic friction is taken into account and for the overlap, soil/geosynthetic friction on one side and geosynthetic/geosynthetic friction on the other side are considered. The calculation principle is the same for both cases:

$$L_L = \frac{T_{max} \cdot \gamma_{mf}}{\left[(\gamma \cdot H + Q) \cdot \left(C_{i\phi 1} \cdot \tan\phi\right)\right] + \left[(\gamma \cdot H + Q) \cdot \left(C_{i\phi 2} \cdot \tan\phi\right)\right]}$$

Where:
- T_{max} is the tensile force on the anchorage $R_{t;d}$
- γ_{mf} is the partial factor applied on the interface shear strength
- Q is the permanent overload
- ϕ is the internal friction angle of the confinement materials
- H is the materials height above the geosynthetic
- γ is the volumic weight of confinement material
- $C_{i\phi j}$ is the interaction coefficient at the soil/geosynthetic interface or geosynthetic/geosynthetic interface (it depends on the type of geosynthetic and confinement material).

As far as overlapping in the transverse direction is concerned, the continuity of the reinforcement must be ensured in order for it to perform its functions correctly. In the case of unidirectional reinforcement, the overlap length (x) of the plates must be within the following range.
The RAFAEL model was subsequently corrected and improved through subsequent studies, in that the elongation of the geosynthetic material in the anchorage zone was studied in depth and the consequences were analysed, which consist of an increase in the vertical displacement of the geosynthetic material and a change in the orientation of the reinforcement at the edge of the cavity. Failure to assess these aspects results in a higher tensile strength value of the product, so in order to study the possible scenarios more realistically, these corrections were made (Briancon and Villard - 2008).
In conclusion, once the conditions that the reinforcement must fulfil have been defined, the geosynthetic is also selected according to the nature of the constituent polymer and the method of manufacture.

9.5 Results of the study

The use of geosynthetic reinforcements to protect the geomembrane and waterproofing system of a landfill site in Champigny-sur-Yonne is an interesting solution from a technical and economic point of view. The RAFAEL method was used to estimate the geosynthetic tensile strength required to retain the loads of the new overlying waste, taking into account the subsidence or collapse of the old waste during compaction and consolidation.
To date, this calculation method is the most appropriate method for landfill projects. In fact, this method was developed for the case of granular soil collapse over sinkholes. However, waste behaves slightly differently to plant materials (Dixon and Jones, 2005). Therefore, specific waste studies and tests should be carried out to model the behaviour of these materials.

10. Conclusions

This chapter summarises the conclusions drawn from this study:

- As stated in the previous chapters, the variability of the mechanical and geotechnical parameters of the waste, due to the environmental and site conditions under consideration, makes the calculation of subsidence within a waste facility complex.
- The model developed is accurate and robust. After showing the constitutive assumptions of the developed model, sensitivity analyses were carried out in Chapter 9 to study the robustness of the model. The primary consolidation coefficient was decreased and increased by 3% without obtaining a significant change in the subsidence trend. In fact, the latter always remains confined within the range drawn to assess the model's goodness. On the other hand, the secondary consolidation coefficient was increased and decreased by 5%, while the primary consolidation coefficient was always considered with the standard value. This scenario also did not lead to a significant change in the subsidence trend.
  Other 2 scenarios were presented in order to investigate the robustness of the model, without showing any particular anomalies. This demonstrates how small variations in the parameters influencing the magnitude of subsidence do not lead to substantial changes in the behaviour performed.
- The subsidence mechanism explains how a correct evaluation of subsidence is essential in order to maximise the efficiency of the landfill, as a greater volume of waste can be delivered than the available volume due to subsidence phenomena. Waste compaction allows the storage of a larger volume of waste, which increases the efficiency of waste management. Furthermore, it is necessary to study the subsidence mechanism not only for volumetric issues, but for stability issues.

- In this study, the focus was on the analysis of the primary and secondary components of subsidence as the starting data consisted of topographic measurements taken every six months. With the aforementioned cadence, it was very difficult and unreliable to conduct an accurate study of each individual subsidence component. In order to be able to discriminate between them, topographical measurements would have to be carried out at shorter time intervals through the use of drones or total stations placed at two extremes of the landfill body .A shorter time frame would allow for a more accurate study, as the change in the parameters influencing the study, such as the height of each individual waste layer considered, the age of the waste, the primary and secondary consolidation coefficients, the volume weight, the void index, etc., can be continuously studied. It is important to remember that many of these parameters are site-specific, as they depend on environmental, climatic and social factors, so a site-specific study is necessary.
- Following legislation at European level, it is necessary to try to increase the useful life of a product as much as possible and to choose products that can be recycled at the end of their useful life and not sent to landfill. The correct application of these principles also concerns every citizen of the world as the nature and quantity of waste sent to landfill depends on the lifestyle, habits, and sensitivity to environmental issues of each of us.

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