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in

CONSOLIDAMENTO DEI TERRENI LS

FLEXIBLE FACING FOR SOIL NAILING RETAINING SYSTEMS

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a mio zio Gianfranco…

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ABSTRACT

La chiodatura dei terreni è una tecnica di consolidamento del terreno che consiste nell’inclusione di elementi di rinforzo (solitamente barre d’acciaio) in un terreno di riempimento, successivamente rivestiti con uno strato di malta cementizia. L’unione di questi elementi con il terreno e il paramento, costituisce la formazione di una struttura omogenea con funzione di sostegno e rinforzo di pendii instabili.

La tecnologia del *soil nailing* è largamente utilizzata in tutto il mondo per il sostegno di muri in terra verticali o, più generalmente, per il sostegno di pendii con un’inclinazione che in condizioni normali non permetterebbe il raggiungimento di uno stato di equilibrio. Una quota parte significativa dei costi realizzativi di una struttura costruita con la tecnica del *soil nailing* è da attribuire alla scelta costruttiva di realizzare una parete rigida in calcestruzzo come paramento.

Le potenzialità, in termini costruttivi, economici e ambientali nell’utilizzo di un paramento flessibile, realizzato con una maglia esagonale in acciaio, solitamente utilizzata per le opere di ritenuta di massi, in alternativa alla scelta costruttiva di realizzare un paramento rigido in calcestruzzo, sono studiate in questa ricerca.

E’ importante precisare che muri in terra a forte inclinazione, che solitamente sono caratterizzati da una mobilitazione del terreno di notevole entità, nei quali entrano in gioco forze di elevato ordine di grandezza, richiedono comunque la realizzazione di un paramento rigido, che assicura stabilità alla struttura stessa. L’utilizzo di paramento flessibile per pendii con forte inclinazione è comunque possibile se le deformazioni che nascono non interferiscono con strutture limitrofe.

Questa tecnica presenta anche un minor impatto ambientale rispetto ad una struttura con paramento rigido, in quanto la crescita di vegetazione è concessa

e nella maggior parte dei casi voluta con la costruzione dei cosiddetti muri verdi. L'uso di maglie in acciaio rappresenta una soluzione economica anche per questo aspetto, perché con la crescita di vegetazione si elimina la necessità di progettare un sistema di drenaggio per eliminare le pressioni interstiziali nel terreno.

Sebbene l'utilizzo di questa tecnologia è in continuo aumento, non esistono metodi di progetto specifici per la loro costruzione. Sono presenti solo approcci empirici che forniscono solamente indicazioni generali.

Per questo motivo e con l'intento di studiare e analizzare il comportamento di strutture realizzate con la tecnica del *soil nailing* a paramento flessibile, modelli numerici alle differenze finite realizzati col software di modellazione geotecnica *FLAC*^{3D} e modelli numerici agli elementi finiti sviluppati con il software per il calcolo strutturale, *Straus7*, sono stati implementati e studiati in questa ricerca.

Con il software di modellazione geotecnica sono stati studiati e analizzati sette differenti modelli di struttura, differenti tra loro principalmente per la diversa tecnologia costruttiva del paramento (rigido, flessibile, deformabile) e per le caratteristiche del pendio e degli elementi utilizzati. Da questi modelli, dapprima, si è analizzato il differente comportamento tra le diverse strutture (deformazioni, stress nei chiodi e stress sulla facciata) comparandole tra di loro e in seguito le strutture realizzate con paramento flessibile sono state prese in considerazione per la seconda parte della modellazione, che è stata effettuata utilizzando il software agli elementi finiti *Straus7*. Con questo secondo software si è modellizzata numericamente la maglia esagonale di acciaio utilizzata come paramento strutturale, considerando diverse dimensioni delle aperture (60 x 80 mm, 80 x 100 mm, 100 x 120 mm), applicando ad esse lo sforzo membranale calcolato nei precedenti modelli alle differenze finite comparandoli con la tensione nominale della maglia, che in questo studio è stata considerata di un valore di circa 350-500 N/mm^2 (*Maccaferri Rocknetfall*).

Si è realizzato quindi un modello multi-scala della maglia in acciaio esagonale, comparando lo sforzo membranale agente sull'intera maglia in acciaio (studiata come elemento unico e omogeneo) allo sforzo totale di fibra agente nel singolo elemento componente la stessa. In questo modo si è fatta l'ipotesi che il modello in macro-scala non sia reagente a sforzi di momento flettente ma solamente a sforzi membranali, mentre nel modello in micro-scala, i vincoli tra i singoli elementi sono considerati rigidi (ipotesi derivante dalle proprietà dell'acciaio) e quindi reagenti anche a sforzi di momento.
Con queste semplificazioni è possibile dimostrare come la maglia esagonale di acciaio abbia funzione di controllo dell’erosione nel caso di bassi valori dell’angolo di inclinazione del pendio mentre all’aumentare della pendenza, l’elemento espleti anche funzione strutturale.
Inoltre l'utilizzo di una maglia con aperture larghe con funzione strutturale, accoppiata ad un geotessile non tessuto con funzione di controllo dell'erosione, può permettere un risparmio in termini di consumo di materiale e quindi di costi senza compromettere la stabilità strutturale della facciata.

Riassumendo, la tecnica di realizzazione di strutture con la tecnica del *soil nailing* con paramento flessibile rappresenta una valida alternativa alla tecnica che prevede un paramento rigido, in quanto la maglia esagonale espleta, se pur in maniera ridotta, una funzione strutturale soprattutto di contenimento delle deformazioni e, caratteristica fondamentale di questa tecnica, con la possibilità di crescita della vegetazione sulla facciata con conseguente impatto ambientale nettamente inferiore rispetto al caso di paramento rigido.

Questa ricerca getta le basi per la realizzazione di linee guida per la progettazione di una tipologia strutturale in continuo sviluppo che presenta vantaggi notevoli rispetto ad altre tecniche di consolidamento presenti.

INTRODUCTION

Soil nails are more or less rigid bars driven into soil or pushed into boreholes which are subsequently filled completely with grout. Together with the in situ soil, they are intended to form a coherent structural entity supporting an excavation or arresting the movement of an unstable slope.

Soil nail walls are a widely used technology for retaining vertical cuts, nearly vertical cuts in soil and any slope which is at an angle steeper than the soil parameters would normally permit. A significant portion of the cost of soil nail wall construction is related to the construction of a reinforced concrete face. The potential for use of a flexible facing design for soil nail walls to replace reinforced concrete facing was studied in this research, studying the literature data and using three-dimensional finite difference modelling. It is important to say that vertical walls will always require concrete facing due to the forces involved. However, steep slopes can use flexible facings instead. This approach represents also an environmental benefit because of its peculiarity to allow the growth of vegetation (green walls).

Soil nails are structural reinforcing elements installed to stabilize steep slopes and vertical faces created during excavations. Commonly used soil nails are made of steel bars covered with cement grout. The grout is applied to protect the steel bars from corrosion and to transfer the load efficiently to nearest stable ground. Some form of support, usually wire mesh-reinforced *shotcrete*, is provided at the construction face to support the face between the nails and to serve as a bearing surface for the nail head plates. The use of wire mesh-reinforced *shotcrete* facing can require the mobilization of a specialty contractor and increase the cost of a project. Use of flexible facing material such as geosynthetic, steel wire mesh, or chain link without *shotcrete* could provide significant savings. In recent years, alternative forms of facing support for soil nail supported slopes have been used, including steel wire mesh which has been successfully applied in Europe. The use of high strength steel wire mesh is economical, eliminates the need of drainage, and facilitates the greening of the slopes.

The weight and friction of the mesh material provides stability, and allows controlled downward movement of material. More advanced installations provide deeper stabilization by holding the mesh to the surface with anchors or soil nails throughout. These designs are largely dependent on the ability of the system to transfer forces from the facing material to the anchor points. The low tensile strength of conventional wire mesh has led to the use of steel wire rope nets, but these nets tend to be relatively expensive. These limitations have been overcome by the development of a cost-effective diagonal wire mesh manufactured from high tensile strength, highly corrosion- resistant wire.

When the wire mesh is used as a facing material, the mesh and nails act together as a system to provide stability to the slope, preventing deformations in the top layers and restricting movement along planes of weakness. With the high strength of the mesh, it is possible (not so common) to pre-tension the system against the slope, and the pre-tensioning enables the mesh to provide active pressure against the slope, preventing break-outs between the nails.

Although the use of flexible facings are increasingly common, there is no design method for their construction. Only few empirical methods exist and they only provide approximate indications.

Thesis layout

The aim of this research is to understand the behavior of a soil nailed structure with a flexible facing, hence to find when it represents a cost effective solution and which limits it could shows. This aim was discussed in two different points:

The first was to investigate the load transfer mechanisms occurring in the structure and how they are relevant to the mesh; the second point was to determine whether commonly used mesh/nail arrangements have a suitable factor of safety.

This will be carried out through a review of the data found in the literature and through analytical models developed with the finite differences software *FLAC3D* and the finite elements software *Straus7*.

The following chapters examine in further detail the contents of the introduction and an explanation of each chapter is outlined below:

Chapter 1: *Fundamentals of soil nailing*. The review covers the concept of soil nailing and its historical application and development throughout the world. The review includes the description of different application fields in which soil nailing system is used and a brief description of the characteristics of the soils in which it can be installed.

Chapter 2: *Key mechanisms of behavior*. The fundamentals of soil nail design are examined. In particular the behavior and the design of the nails focused on the description of how they act in the transfer of loads and how they can provide equilibrium in the structure in a limit state condition against failure, are investigated.

Chapter 3: *Facing*. The facing system can modify the internal failure mechanisms. Three types of facing can be used in a soil nailing design. These are soft, flexible and hard. Every type has different characteristics of element composing the structure, differences in the design and in the construction. In particular, this chapter, is focused on a detailed design of flexible facing systems.

Chapter 4: *Numerical analysis of soil nailed walls with flexible facing*. A series of *FLAC3D* finite differences models were constructed to simulate the performance of different soil nail walls with steel wire mesh as facing system. The aim of this chapter is to simulate and understand the real behavior of flexible facing, hence how the stresses develop in the different elements by their own stiffness and, in particular it is focused, on the stress developed in the facing and how it could be design to be cost effective and safe at the same time, compared with the other types of facing.

Chapter 5: *Numerical analysis of the stress acting in steel wire meshes*. In this chapter the coupling stress acting in the *geogrid* element output with *FLAC3D* models is converted into a total fiber stress acting in a single element composing the steel wire mesh to compare it with the nominal tension strength of the wire mesh itself. The aim of this chapter is to understand whether the flexible facing system is acting with a structural function or it only can provide the function of erosion control.

Chapter 6： Conclusions and recommendations. The research is concluded and further recommended research is presented.

1. FUNDAMENTALS OF SOIL NAILING

1.1 Generals

Soil nailing is a form of soil reinforcement in which the reinforcement is installed into the slope face creating a mass stabilised of ground either natural soil or an existing fill. Soil nails are made of metallic or polymeric material and may be: installed into a pre-drilled hole and then grouted, drilled and grouted simultaneously, or inserted using a displacement technique. Most common soil nails are installed at a sub-horizontal inclination.

Nails used in soil nailing method are usually steel bars or polymeric fibres (FRP) elements, which resist to tensile, shear and bending forces. It is possible to identify two different categories:

- *driven nail*, which are small diameter elements (14 to 45 mm) inserted into the ground with a small spacing (0.5-4 nails per square meter of wall) with a vibro-hammer (pneumatic or hydraulic); steel nails with ductile behaviour to avoid brittle fracture mechanisms are preferable. This type of installation is quick and cheap, even if it is limited for the maximum nail length and for their ineffectiveness in heterogeneous soils.
- *grouted nails*, which vary in size from 15 to 46 mm and placed in pre-drilled holes, large 10 to 15 cm in diameter, with a vertical and horizontal spacing that varies depending on the type of land (0.25 - 1 nail per square meter of wall). The hardening (usually cementation) takes place at atmospheric pressure (gravity force) or at low pressures.

The facing is the final element of the work in a soil nailing technique and it is produced not only in function of the spacing of the nails, but it depends also on the type of the structure, temporary or permanent. The main distinction is between hard and soft facings. The first type is preferred for steep slopes and for permanent works, instead soft type is preferred for shallow slopes and/or aesthetic finish .

The technique of reinforcement is considered very useful and cost-effective for soils with the capacity to sustain itself (stand up time) in an excavation 1-2 meters deep for a period of 1-2 days. Highly weathered rocks are preferred, as well as cemented sands and gravels, and uniform sand from medium to fine size and soils with water capillary cohesion (with a water content of the order of 5%, Byrne et al.1993). However, the method is also applicable to silt soils which are located above the aquifer, as well as in cohesive materials and clays with low plasticity index. A hard, flexible or soft facing may be used at the surface of the slope. These topics will be described in the chapters below.

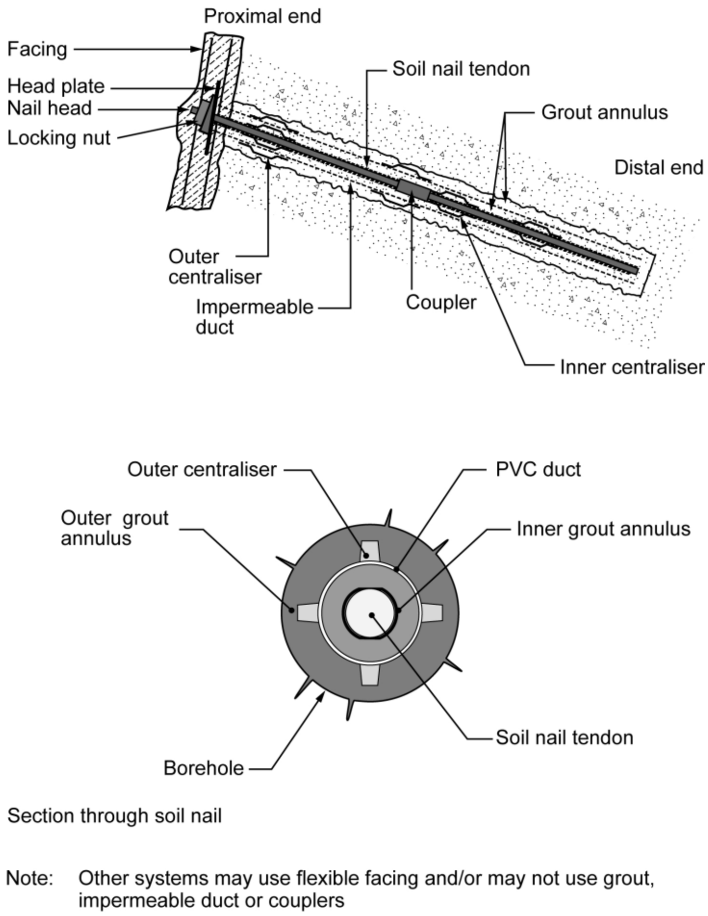


Fig.1.0 Main components of soil nailing system

1.1.1 Basic mechanisms

The basic mechanism of soil reinforcement relies on tensile forces developing in the reinforcement to resist those developing in the soil. To be efficient, the orientation of the soil nails needs to correspond closely to the principal tensile strain field of the soil. In addition, the resistance of the nails to tensile rupture should allow, whether a reduction of properties incomes during the design life (*i.e.* an allowance for corrosion), the maintenance of the structural stability without failure.

The mechanism by which soil nails develop tensile resistance requires some relative movements between the soil and the nails. Because of this, soil nailing is considered a *passive* system. The magnitude and distribution of movements will depend on the type of structure, the type of construction and the spacing between the nails.

It is important to say that much research has focused on the behaviour of the nails and mechanisms relating to the behaviour of the facing are not well understood.

1.1.2 Advantages of Soil Nailing

Soil nailing presents the following advantages that has contributed to the widespread of this technique in several countries (Abramson *et al.* 1995):

- *Economy*: economical evaluation of a few projects has led to the conclusion that soil nailing is definitely a cost - effective technique as compared with a tieback wall.
- *Rate of construction*: fast rates of construction can be achieved if adequate drilling equipment is employed.

- *Facing inclination*: in rigid facings, the use of *shotcrete* easily accommodates an inclined facing, with benefits to overall stability. Backwards inclination of the facing also reduces *shotcrete* losses due to rebound.
- *Deformation behaviour*: observation of actual nailed structures demonstrated that horizontal deformation at the top of the wall ranges from 0.1 to 0.3% of the wall height for well-designed walls (Clouterre 1991, Juran and Elias 1991).
- *Light construction equipment*: soil nailing can be done using conventional drilling and grouting equipment. Thus the techinique is of particular interest on sites with difficult access and limited space constraints
- *Adaptability to different soil conditions*: in heterogeneous ground where boulders or hard rocks may be encountered in softer layers soil nailing generally is more feasible than other technique such as soldier piles.
- *Flexibility*: nailed soil retaining structures are more flexible than classical cast-in-place reinforced concrete retaining structures. Consequently, these structures can conform to deformation of surrounding ground and can withstand larger total and differential settlements. This characteristic of soil nailing can provide economical support for excavations on unstable slopes.
- *Reinforcement redundancy*: if one nail becomes overstressed for any reason, it will not cause failure of the entire wall system. Rather, it will redistribute its overstress to the adjoining nails.
- *Environmental benefits*: the use of a flexible facing system permits the construction of green-walls.

1.1.3 Limitations

Soil nailing technique mobilises soil strength and the soil mass deforms, leading to displacements in the surroundings of the wall. This can bring unacceptable deformation to a sensitive structure in the vicinity of the wall. This effect is higher if the soil nailed structure presents flexible facing because displacements are bigger than which occurred with hard facing system. Placements of the *shotcrete* requires that the excavated face be free standing for a period of time. Corrosion protection requires careful attention in aggressive environments. For a flexible facing system the main limitation is to achieve steeper inclination, because it is not possible to guarantee the stability and the erosion control as in a rigid facing system. Due to this fact, the use of flexible facing is not suggested where important displacements could develop.

1.2 Principal differences between soil nailing and other type of geotechnical structures

Although soil nailing technique shares certain features with the older and more widely known technique of reinforced earth for retaining wall construction, there are also some fundamental differences which are important to note.

Soil nailing structures are realized by a “top-down” technique, which consists in free staged excavations usually 1-2 meters deep (the height must be smaller than the critical height of cut), followed by the inclusion of nails and by the realization of the facing with light cover. This is applied to the achievement of the fixed depth (fig.1.1).

Reinforced earths, on the other hand, are realized with a “bottom-down” technique: the soil is made dense and levelled with rollers and then the plain reinforcement are laid out and covered by other soil subsequently made dense. Also in this case this technique is applied to the achievement of the fixed height.

Another difference is while in the soil nailing the resistant elements are chosen to reinforce a slope or an excavation, for a reinforced earth structures the choice must be taken both for reinforcement and soil, opting to the combination that offers the best result. In addition the reinforcements in soil-nailing are mainly bars (items that can be considered one-dimensional), while in reinforced earth structures strengthening components are continuous elements along the horizontal.

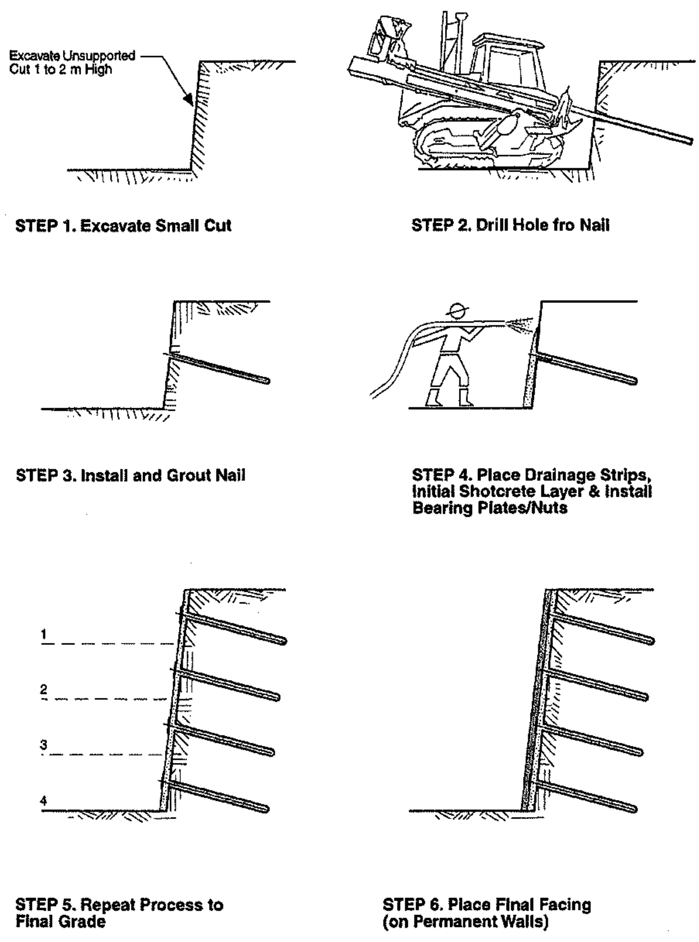


Fig 1.1 Typical nail wall construction sequence (Byrne et al. 1998)

The main difference between these two kinds of structures, however, concerns the forces' distribution and deformations along the walls and sure enough in soil nailing technique the maximum deformation involves the top of the structure unlike the reinforced soil technique where the deformation is bigger at the toe.

The soil nailing system is also used as alternative of soil anchored system. Although the used methods are similar, substantial differences exist between these two techniques of reinforcement (fig.1.2).

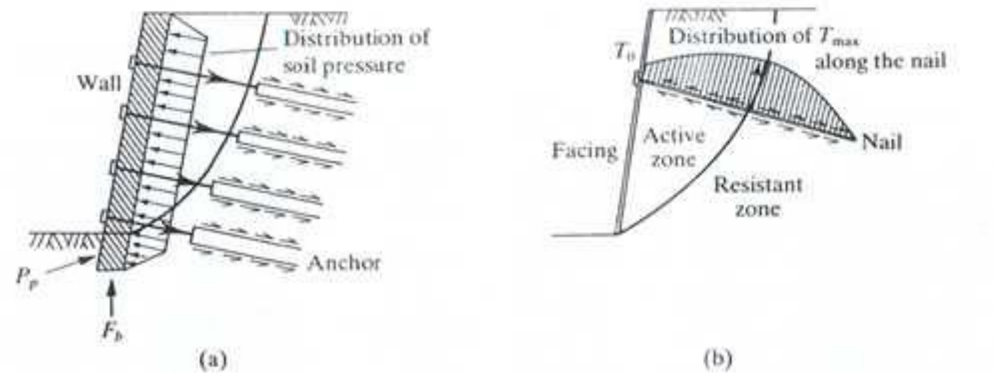


Fig 1.2 Differences between anchors and soil nails

While the anchors are connected to the ground only in a limited zone hence with a limited area where frictional resisting forces are developed, nails are completely connected to the ground and the resistance generated by soil/nail friction is developed along the entire element. For that reason, inclusions are also called “uniform” (Schlosser et al.1983) as the interaction between soil and reinforcement can occur in every part of the inclusion; also the capacity of the nails to develop friction resistance even in the "active zone" makes smaller the forces acting on the facing which, in this manner, could have no bearing capacity. Anchors can be pre-tensioned after installation and that is not possible for nails, therefore they always require soil deformation to develop resistance.

1.3 Development of soil nailing

This section speaks about the history and the development of soil nailing techniques in the 20th century. First techniques were developed partly for rock-bolting and multi-anchorage systems and partly for the reinforcement of soils.

In Austria was developed a method of tunnelling design, between 1957 and 1965, the (New) Austrian Tunnelling Method (NATM). That is a technique for supporting underground galleries and tunnels. The NATM integrates the principles of the behavior of rock masses under load and monitoring the performance of underground construction during construction (fig. 1.3)

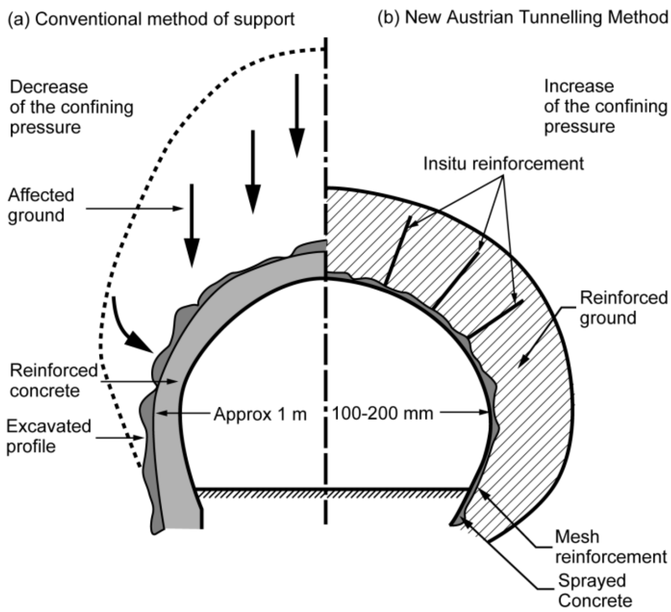


Fig 1.3 Schematic comparison of the New Austrian Tunnelling Method and a traditional method of supporting an underground gallery (from Bruce and Jewell, 1986)

The design's idea is to excavate the tunnel and immediately after that, put steels bar to reinforce the ground and then grout them to achieve a perfect anchorage and meanwhile spray concrete reinforced with steel wire meshes provides primary support realizing a perfect rigid facing (where used). That minimizes the lining deformations and creates a resisting ring-like and in firsts designs it was possible to monitor the deformation and the stresses developing in the structure, hence to improving the knowledge on the structure's behavior and its design.

Using this method is possible to achieve to a better result, building a tunnel lining thinner than those built with conventional method of support.
Soon were built other structures composed by the inclusion of steel bars in rocks, and due to its good behavior in different kind of rocks, this technique was started to be used with soil reinforcement.
Some trials were made up to the first (reported) structure in 1972, built at Versailles. It is an application of passive inclusions in a soil cut which used closely spaced short grouted nails 4m or 6m long (fig. 1.4). This was an 18 meter high wall, with a 70 degrees slope, in Fontainebleau dense sand. It could be considered the first soil nailing structure.

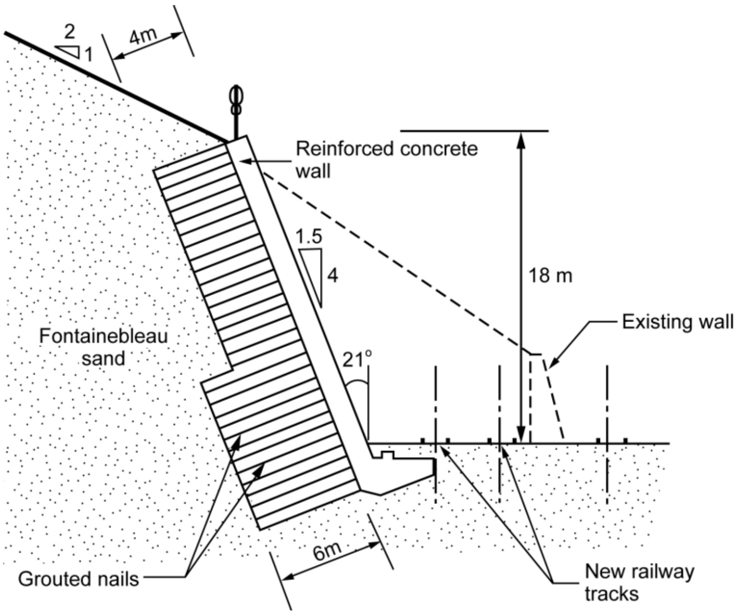


Fig. 1.4 Section through the first soil-nailed wall in the world, built at Versailles, France, in 1972/73 (from Clouterre, 1991)

The development of these techniques involved many countries: the first orderly/systematic research about soil nailing was developed in Germany, concerning the "*Bodenvernagelung Programme*" (1975 – 1979) made by Gässler and Gudheus, who made a lot of studies on this topic; after that, similar programmes were made in the US and in France (*Programme Clouterre*): this research programme included experiments in different soils, with different type of nails and different techniques of nail inclusion, to study the real behaviour of this kind of structures. The results of this research were published in 1991 and form the basis of the soil-nailing design approach used in France and adopted in other countries.

It is possible to say that soil nailing technique is widespread used all over the world. Studies and researches, from Germany to France, from Japan to the USA and the UK, have contributed to the development of this technique, which due to its low impact on the environment and cost-effectiveness, is achieving a fundamental role in the geotechnical applications.

1.4 Development of soil nailing in the UK

The development of soil nailing in the UK has been relatively slow. The main reason was a concern about the long term durability of the nails and about the role of shear and bending in the stability in this kind of structures.

Many studies were carried out by UK's universities and researches institutions like TRL (Transport Research Laboratory) up to 1993, to discovered and study the principles of the behaviour of these structures to find and improve new analytical methods.

So, the development of this technique was totally different than in France and Germany. That is due to the use of soil nailing in existing slope rather than new constructions. This reflects the typical UK's way of thinking that consists in more effort given to remedial works and maintenance than to new construction.

Flexible and soft facing have been used much more extensively in the UK than in France or Germany, this is due to its involvement with sustainable remediation existing slopes. With the aim of this types of facing it is possible to guarantee the growth of vegetation that could be an important factor for both the facing's stability and its visual appearance.

The use of this technique looks to increase in the future and it has become widespread all over the UK, often as part of new infrastructure work or as remedial works to existing infrastructure. The recent works for the Channel Tunnel Rail Link, the M6 Toll Road and the A3 Hindhead Tunnel Scheme have included several soil nailing applications.

1.5 Application of soil nailing

This technique of soil improvement can be used in:

- Stabilisation of existing retaining walls;
- Stabilisation of existing (unstable) slopes *e.g.*:
    - *Natural slopes*: soil nailing can be used to stabilise natural slopes. For example at Dolywern, north Wales, in 1986 a 10 m-high slope was stabilised using seven rows of soil nails (Barley, 1992);

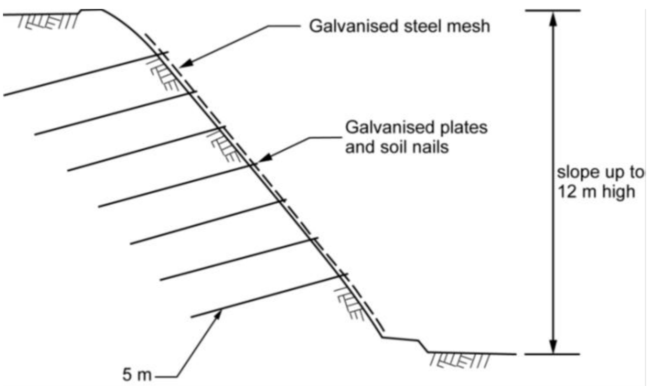
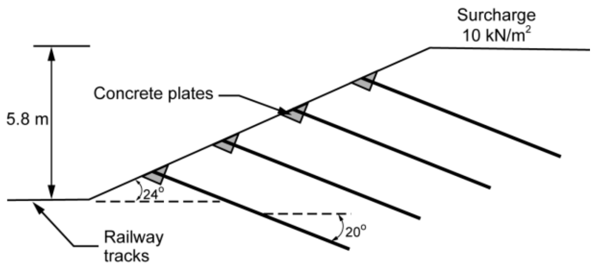


Fig. 1.5 Natural slopes (Barley, 1992)

- *Railway embankments*: stabilisation of the side slope of existing railway embankments is currently the largest single application of soil nailing in the UK. Soil nailing can be a good solution where access is difficult, because drilling equipment mounted on long reach excavators can sit at the toe of the embankment slope and temporary works are minimal. Self-drilled nails are commonly used on the majority of railway earthwork stabilisation sites in the UK for the following reason:
  - easy access;
  - rapid installation.

- Railway cuttings;



Number of rows:	4
Nail length:	8.0 m
Nail diameter:	200 mm
Bar diameter:	25 mm steel
Nail type:	drilled and grouted
Horizontal spacing:	2.0 m
Vertical spacing:	1.2 m

Fig. 1.6 Railway cuttings

- Highway embankments and cuttings;
- Embankment dams;

1.6 Aspects of ground conditions relevant to soil nailing

Not all soils are suitable for nailing. It is important to know and understand the characteristics of the soil that will be managed and worked, therefore a well define site investigation is required.

There are many problems that could affect the behaviour of the soil, particularly during construction and in the long term.

It is possible to divide soil in three main categories:

- Cohesive soils;
- Granular soils;
- Soft/weak rocks.

The aim of designers, after the study of geological and hydrogeological soil condition is to identify the specific risk and the best suitability of soil nailing. It is also important to study the ground condition that governs the application of soil nailing. Soil nailing is considered a cost-effective solution if the shear strength developed by the soil is sufficient and it works as solution if the bond generated between the nail and the soil into which is installed is adequate.

Soil nailing technique gradually progresses down the slope: a row of nails in installed in a bench cut with a height of 1-2 metres and it needs to stay unsupported until the nail have been installed. So it is necessary that the bench cut hence the soil, has sufficient apparent cohesion, hence shear strength.

1.6.1 Cohesive soils

Soil nailing is unsuitable for soft clays and silt. That is due to the low shear strength developed in this type of soil hence it provides low bond strength as well as the impossibility to maintain temporary stability. Deformations are also difficult to control, they could be really excessive and the maintenance of the stability of the structures would become unsustainable in economical and time terms.

Differently, firm to stiff clays are suitable for soil nailing because their un-drained shear strength is greater than 50 kPa and provides sufficient bond strength, particularly in slope where tension forces are quite low.

In dry condition they also provide enough stand-up time for excavation of benches and they also provide good bore stability for drilled and grouted nails without the need for casing.

There are some particular problems that would occur in cohesive soils. These are shrinking and swelling and they usually affect high plasticity and over consolidated clays subjected to change their volume very easily because of their mineral composition.

Cohesive soil structures suffer deterioration at the crest and the face because the repeated seasonal wetting and drying causes shrinking and swelling. High plasticity clays are more inclined to suffer these problems than low plasticity ones.

Particular care is required to ensure that nail spacing, head plate dimension and facing stiffness are all sufficient to avoid gradual degradation of the nail system which could lead to progressive failure.

1.6.2 Granular soils

Because of their high angle of shearing resisting resulting in a strong bond, granular soils are well suited to soil nail applications. It is required to pay attention to the short term stability because it is possible that granular soil do not provide sufficient cutting stability in the short term causing unstability in the benches. In many cases, where soils are unsaturated, suctions will provide temporary stability thanks to their cohesion properties.

Particular attention must be given to the presence of ground water that can destroy the apparent cohesion.

1.6.3 Weak rocks

Weak rock are well suited for soil nailing because they provide good shear resistance hence good bond strength. They provide also good stand-up time of excavated forces. It is important to pay attention with the joints because they could contain low strength fine material and the structure could occur to an unstable state, both in short and long term.

1.6.5 Groundwater

For most soil types, soil nails are not suited to applications below the water table and should be installed from a dry excavation.

Groundwater can have adverse effect on:

- Bond;
- Durability of the nail and the integrity of the grout;
- Stability and durability of the facing;

- Stability of temporary excavations;
- Overall stability of slopes;

Seepage of groundwater through the unsupported cut face can lead to instability of temporary excavations, particularly in predominantly granular soils or cohesive soils (fig. 1.7).

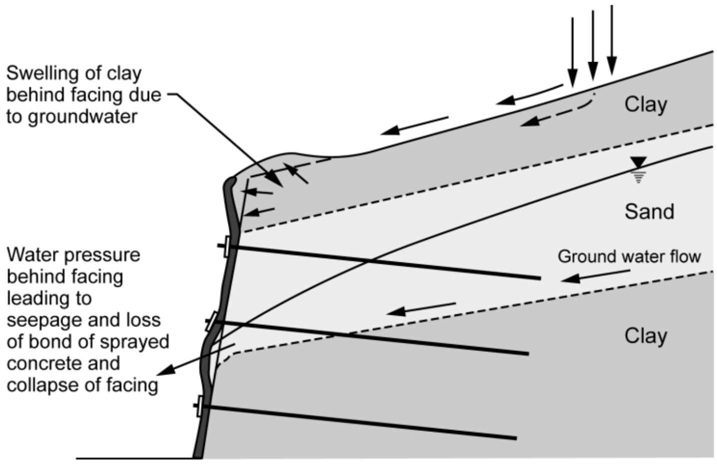


Fig. 1.7 Effect of ground water on wall facing

Groundwater at depth can still introduce difficulties, both in terms of installation and in the long term performance of the nails. Groundwater flow through the soil can lead to instability of nail bores, unless casings are employed.

Effects of long term change in ground water conditions on the soil nailing scheme. This may include climate charge, different rates of abstraction from aquifers or changes in groundwater regime.

A rise in groundwater is often associated with old mining areas where groundwater pumping of the mines has ceased an equilibrium of the natural water table is gradually being restored.

2. KEY MECHANISMS OF BEHAVIOUR

The reinforcements used in reinforced earth systems have the primary function to develop tensile strength to collaborate in the whole structure behaviour, so their maximum efficiency is achieved by placing them in the same direction of principal strains axes.

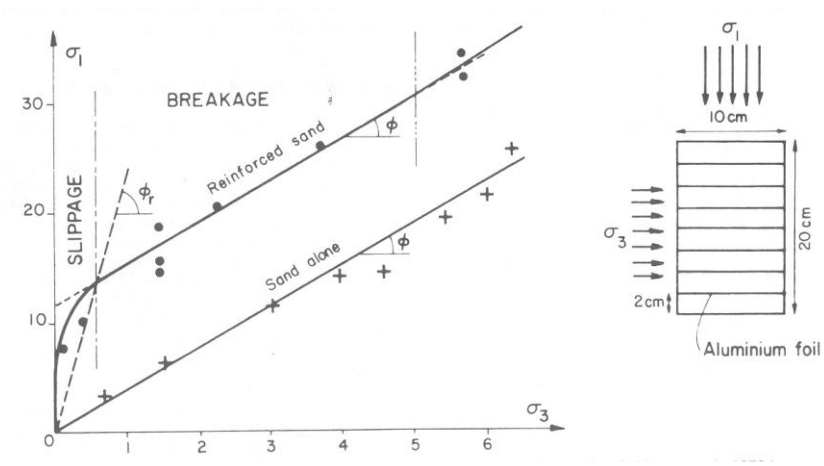
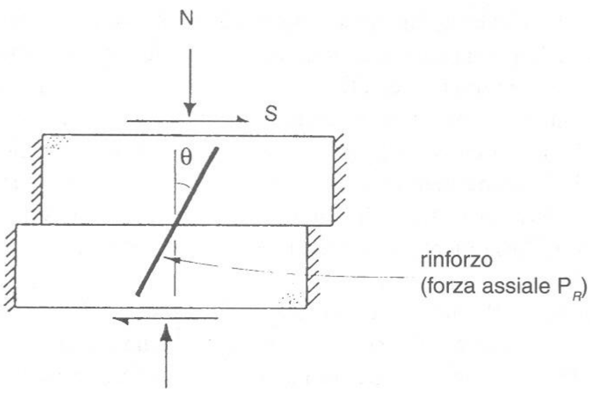


Figure 2.1: Effects of reinforcement of a sample of soil in triaxial conditions (from Schloesser et al., 1972).

The usefulness of reinforcing element can also be evaluated with reference to a simple scheme in which the reinforcement intercepts a failure surface (fig. 2.1). The beneficial effects of the presence of a tensile stressed element are:

- the component of stress in the reinforcement (P_R) normal to the surface scroll ($P_R$senθ) contributes positively to the shear strength to increase the normal strain forces;
- the component of stress in the reinforcement (P_R) parallel to the surface scroll ($P_R$cosθ) contributes positively to the efforts for reduce shear forces.



$$\sigma_n = \frac{1}{A}(N + P_R \sin\theta)$$

$$\tau = \frac{1}{A}(S - P_R \cos\theta)$$

Figure 2.2: Effects of reinforcement on both sides of a fracture surface of the ground.

The behaviour of a reinforced soil mass is typical of composite materials whose mutual interactions are developed by friction. So, it depends primarily by two variables that govern the shear behaviour of the uncemented bodies: friction angle (in this case the interface between reinforcement and soil) and normal stress acting on the interface’s surface (for reinforcement plans is usually the vertical tension or a component).

The stress states induced by these mechanisms of reinforcement interaction are: a tension or compression state due to longitudinal interaction, and a shear and bending state due to transverse interaction.

The interaction between soil and inclusions has two beneficial effects: reduced deformability and an increase in shear strength. Stability is satisfied by the mobilisation of shear stresses at the soil/nail interface.

2.1 Transfer of loads

Soil nailing technique improves the stability of an excavation or a slope mainly through the mobilization of tensile stresses in the inclusions that is developed through the friction interaction between soil and reinforcement and for the reaction of the head of the nail and the facing.

As a result of small deformation occurring in the facing, the nail is subject to displacements both in the axial and transverse direction to its axis that induce stress. The axial displacements are generated by tensile stresses occur in the nail, which can reach the maximum limit value equal to the maximum frictional resistance that can occur in the soil-reinforcement interface, which is called *pullout* resistance. The tensile stresses contribute to increase the soil resistance, whether absorbing part of the shear stress or causing an increase of normal stress along the potential failure surface.

Lateral displacements develop transverse forces on the nail; these forces achieve the maximum limit equal to the bearing capacity of the soil determined in the same way of piles loaded by horizontal forces. Shear and bending forces are influenced by the inclination and the stiffness of the nail itself. That is due to lateral displacements arising in the nail.

Because of its thinness that characterizes nail, reinforcing actions related to shear and bending are limited by low bending resistance and are usually not considered (FHWA 1998). They do not occur under deformations of less than 0.3 – 0.4 per cent of the wall or slope height.

The tensile stress generated in the nails has parabolic development and its peak, which coincides with the hypothetical failure surface, separates the soil-nailing system in two areas: it is greater than the stress transferred to the facing (as shown in figure 2.3):

- *ACTIVE ZONE*: Zone of potential failure where the friction forces along the nail are directed towards the facing and act to remove the reinforcement;
- *RESISTANT ZONE*: passive zone, where friction forces are directed toward the interior of the slope, preventing outward movement of the nail and, therefore they minimize displacements even in the active zone.

Often we consider the soil nails as the elements that bind in a certain way the active region with the passive and the concept of two distinct and separated areas, however, is only an idealization to simplify the model. There is actually a complex failure zone subject to shear distortion, and also the failure surface is influenced by the presence of joints where it is evident a beginning of detachment.

Soil nail head together with the facing perform primarily a confinement function minimizing possible deformations of the soil, with a consequent growth of the effective tension and the shear strength of the soil behind the nail head, and help to avoid preventive local ruptures near the surface of the wall. As evident from the forces distribution the strain hanging on the facing and on the nail heads is less than the maximum achievable value because of the interaction between nail and soil even in the active region, hence the forces at the nail head will never be as high as the maximum resistance developed further down the nail. In this way, the facing may have not bearing functions but only aims for protection and containment.

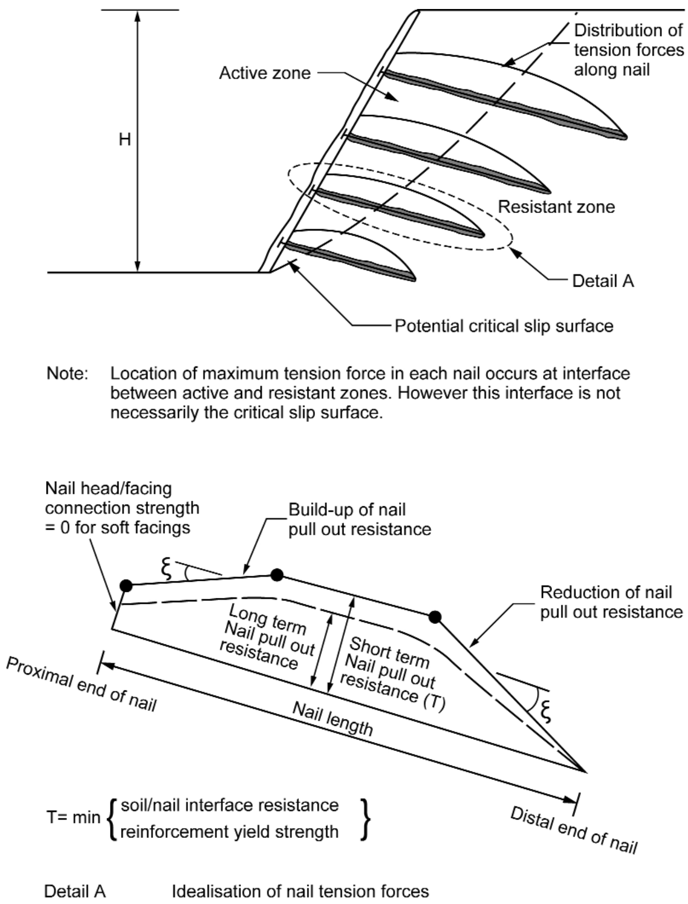


Figure 2.3: Active zone and resistant zone, and stress distribution in nails

The *pullout* resistance is provided by the part of the nail located in the passive zone and the mobilization of its resistance depends on several factors. Theoretically, the shear strength acting between the soil nail and the surrounding soil depends on the strain contact and the factor of friction. When a soil nail is drilled and filled with grout, the drilling process reduces significantly the radial tension on drilling perforation, the hole remains stable due to arching.

2.2 Failure domain of the nail

Although the soil nails are mainly stimulated by tensile stresses, as a result of big displacements and because they cross the failure surface, they are also subjected to shear and bending stresses that deform the element into a "S" shape. The position on the nail where the stresses are greater are two points: "A" (Fig 2.4) where the value of the bending is maximum and where they are at the same distance (symmetrically) from the failure surface, and point "B" where the nail intersects the failure surface, which is precisely the point where the maximum shear stress is acting (and therefore with none bending stresses). The shear and bending stresses are uniquely related to each other, once defined load conditions, and the tensile stress generated in the nail is totally independent.

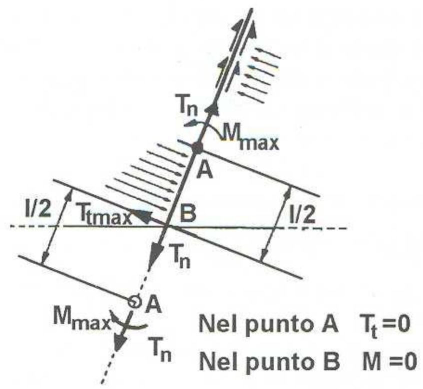


Figure 2.4: Loads and stresses subjected by the nails on the failure surface (Jewell, 1990 edited by Evangelista 1995), where T_n is the tensile stress along the nail, T_t is the maximum shear stress, M_{max} is the maximum bending stress.

These shear and bending forces, although they are usually ignored in design phase, affect the maximum resistance which can be provided by the soil nail and depends on the rupture of the nail itself. The reinforcement ability to support these other types of stress as well, can increases the shear strength of the soil. However, ignoring development of shear and bending in design is conservative. The value of the maximum force the nail is required to carry can be obtained from analysis on possible states of stress that can be developed simultaneously. The domain of possible states of stress in the plan T_n-T_t for a nail was defined by Schlosser (1982) by using four failure criteria. Based on the Mohr's circle, Jewell et al. (1987) proposed a relationship for the calculation of the strain forces developed on the nail as a function of its angle of inclination compared to the failure surface.

This report, confirmed by experimental results of shear tests on sand reinforced with bar embedded in different inclinations, is indicative of the relationship between the maximum tensile stress in the nail and its angle of inclination β (Fig. 2.5), the soil shear strength increases with the angle β until it reaches up to 30° and then decreases. The results showed that the presence of reinforcement produces a re-orientation of the principal directions of deformation of the soil. The deformation of the soil in the vicinity of the reinforcement is less than the deformation occurring in a unreinforced soil, because the presence of the reinforcement inhibits the formation of failure surfaces. The shear strength of the soil increases if the reinforcement has the same orientation of the main directions of strain forces, but it decreases when it follows the orientation of the compression forces.

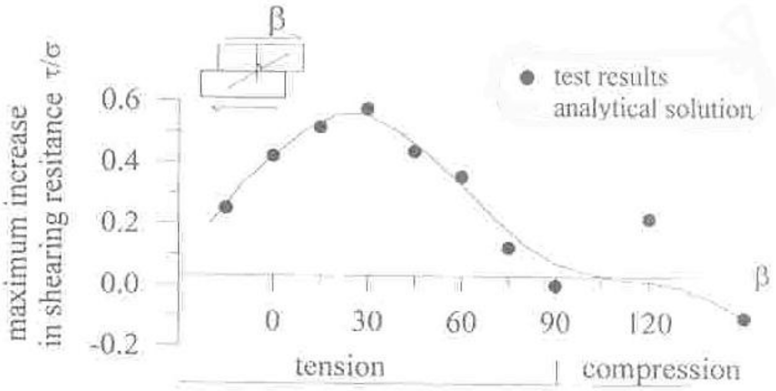


Figure 2.5: Variation of shear strength as a function of the angle of inclination β of the nail, with a rough reinforcement (Jewell et al., 1987)

As written above, the effect of reinforcement is influenced by the angle of installation, Gassler identified three groups that can divided the soil nail types (Gassler, 1992): those that are installed horizontally, working mainly under the action of tensile; soil nailing structures with a small bending stiffness that are installed with a slight inclination compared to the failure surface, and that work, however, mainly under the action of tensile forces; reinforcements with high bending resistance that are installed almost perpendicular to the slipping surface and that develop shear strength if important displacements are generated.

The displacements required to mobilize the shear and bending forces in nails are bigger than those that allow to achieve the maximum tensile strength and ultimate shear strength in the soil. For this reason, during the structure serviceability, due to the reduced movements, their contribution in the total resistance generated by the reinforcement is insignificant. Therefore, for shear and bending to be taken in account the designer needs to verify that the deformations will be sufficient to mobilise these resistances.

2.3 Mechanisms of failure and design methods for soil nailing

The detailed design of soil nailed slopes or walls is based on information about the soil, groundwater conditions, loads, geometry and type of soil nail to be used. Detailed design is undertaken to fulfil the following main requirements:

- to satisfy equilibrium of forces and moments (strength and stability)
- to limit displacements (serviceability)
- to maintain this performance criteria throughout the specified design life (durability)

The design of this kind of structures follows steps that depend each other like every other geotechnical structure. For this technique they consist in:

1) define geometry and design cross-sections;

2) define surcharges and loads;

3) define ground model;

4) define groundwater and design of drainage;

5) design codes and design methods;

6) define characteristic soil strengths;

7) determination of design soil parameters and design loads;

8) internal stability and *pullout* resistance.

9) nail tendon design;

10) ground aggressivity and corrosion protection;

11) internal and external stability checks;

12) design of facing and head plates;

13) prediction of deformation.

The firsts six steps are well defined above. This chapter treated the following steps.

In stability analysis of an excavation or a slope of primary importance is the identification of all possible sliding surfaces, in those surfaces is exceed the soil capacity to resist shear forces. The sliding surface, as seen, divides the system into an active and a passive zone, it is usually identified taking into account the mechanical properties of the soil and possible overload. The shear strength of the soil is mobilized along this surface, which can be expressed in terms of effective stress, such as $\tau= c'+ \sigma' \tan\phi'$ according to the Mohr Coulomb failure criterion. If the shear strength available is less than that required to prevent that the active zone is subjected to a relative displacements in relation with the passive zone, the soil breaks along the failure surface; conversely the excavation or a slope is stable.

Soil nailing structures, because of the presence of reinforcement, may be affected by failure mechanisms both internal and external, the firsts regards the failure which may subject the individual nail; in the seconds soil nail reinforcement and soil are considered as a single monolithic system that can be affected by a slipping surface. When they occur at the same time a mixed failure is happening.

The main types of internal failure that may affect the soil nail, both in active and passive zone, are: the *pullout* failure of reinforcement for loss of friction between the soil nail and the surrounding soil. Exceeding the maximum load bearing capacity of the soil due to excessive movement of the soil nail, the nail rupture due to excess tensile stresses or to the combined action of shear and bending forces; structural failures due to the rupture of the soil nail head or due to the rupture of the facing.

External failure of the soil nailing system occur when sliding mechanisms along the slipping surfaces, together with rotation and translation mechanisms occur in the soil-reinforcement complex.

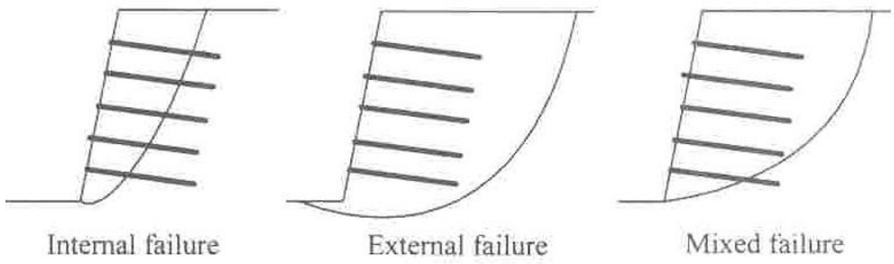


Fig 2.6 Possible mechanisms of failure of an excavation with the technique of reinforced soil nailing

There are several methods proposed for the soil-nailing system design based on the analysis of stability with the limit equilibrium method and on the use of partial safety factors. In the analysis of equilibrium stability limit, different assumptions about the potential slipping surface can be made and as well as forces to break the nail. Failure surfaces are variable from method to another one in a wide range: those that are linear methods that assume the formation of a slipping wedge or those bi – linear, logarithmic spiral and circular type; much discussed is the shear and bending forces influence in the soil nail system stability, because it’s of minor importance compared to tensile stresses (<10%), as already mentioned. In many cases they also can be overlooked.

The methods as well as the shape of the failure surface and for the stress state in the nail, differ among themselves for the factors of safety adopted: some methods, oldest ones and already outdated, referring to a single factor of global security calculated with the available resistance and acting forces ratio.

Other methods, more sophisticated but also more reliable, are based on several factors that take into account the possible failure type which may be locally subject a soil nailed slope and the various factors that may affect in some way the system stability.

In the latter approach is verified that the destabilising forces are less than the resistances, $S \leq R$, where S = τ_{mob} is equal to the resistance acting along the slipping surface and R = τ^{s} + $\Delta\tau_{NL}$ + $\Delta\tau_{NT}$ is the sum of the resistance respectively with the absence of strong nails and contributions due to longitudinal and transverse components of the strain force developed by the nails to the failure surface.

When the structures, may be affected by deformation to a lesser extent than those that would result in the collapse of the system, they are in a state called serviceability condition and, however, must also be verified the rupture condition. The deformations which can occur in the system cannot be catastrophic, but they can cause nevertheless a loss of structure's functionality as well as damage to surrounding structures or infrastructures: it must be ensured that the weaknesses developed in the excavation are acceptable as well as the reinforcements' deformation. The structure's functionality can be a problem in long-term. The displacements of the excavation shall be such as not to cause disturbance to nearby buildings or infrastructures, failure of the facing, unequal loads' distribution between the nails, which then can lead to rupture of the most loaded reinforcement, ground-breaking strength. Several trial fields have been allowed, through continuous monitoring, to estimate horizontal and vertical displacements at the top of the structure. They may be considered acceptable if they are of the order of 0.1 to 0.4% of the height of the slope (Clouterre, 1991; Srinivasa et al.2002).

The different published design methods favour slip surfaces of varying geometry. In order to ensure a safe design the most critical of these should be identified. For long term serviceability the critical slip surface should be calculated taking the possible long term soil strengths into account. The published design methods, from a number of different countries, are discussed below together with their main recommendations. The different national methods are often developed in parallel and are based on differing design philosophies. Hence the assumptions which each method makes can be very different.

2.4 Limit state design

When a soil-nailed wall or slope, or part of it, fails to satisfy any of its performance criteria, the wall or slope is deemed to have reached a limit state.

Structures' stability can't be achieved also for deformations problem. Predicting deformations of soil-nailed walls and slope can be difficult. It is common to introduce partial factor of safety for a correct and safety design.

2.4.1 Ultimate limit state

Ultimate limit states are generally associated with total collapse or failure. It is achieved when disturbing forces exceed the available stabilising forces at any particular moment. The limit state can occurs in:

• External stability – the failure falls outside the zone of reinforcement.

• Internal stability – mechanical failure of the nail elements, generally more than one element, *pullout* may also result in internal failure of the soil nailed system. The nature of soil nailing allows transfer of load from one nail to another if an individual failure occurs.

• Compound stability – a possible mixture of external and internal stability.

The retaining wall system should be designed to exhibit sufficient ductility in approaching geotechnical limit states to give visible warning of failure. The ultimate state design requires to consider the likely hazard and risk could occur in the slope during is design life. For soil-nailed slopes and wall, these are:

- External stability hazards
    - loss of overall stability at any stage;
    - rotation;

- sliding;
- foundation failure;

- Internal stability hazards
  - *pullout* of nails through failure at the soil/nail interface;
  - rupture of the soil nails
  - toppling of the facing
  - bending or punching failure of structural facings
  - punching or bearing failure of head plates
  - failure of soil between the nails
  - bearing capacity of structural failure of head bearing pads.

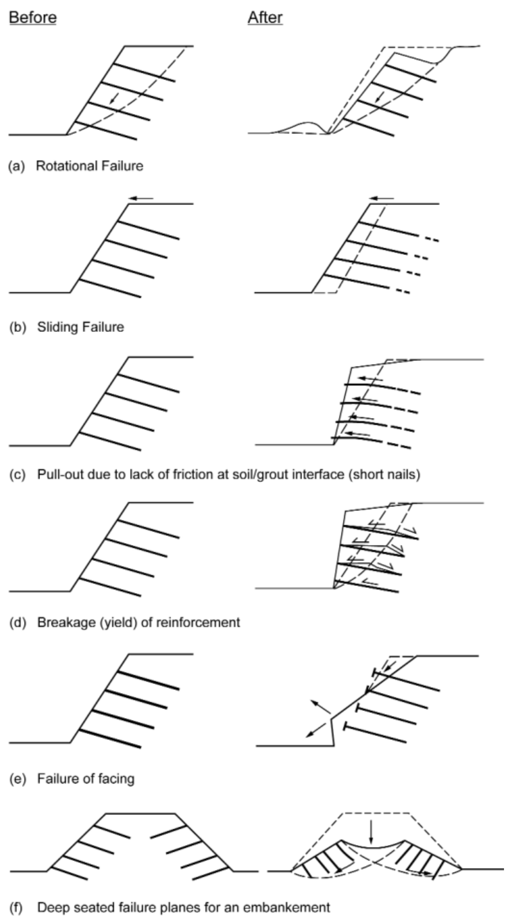


Fig. 2.7 Illustrations of ultimate limit states for soil-nailed slopes and walls

2.4.2 Serviceability limit state

A serviceability failure occurs if a structure deforms more than an allowable value. This type of failure is not necessarily catastrophic but could be hazardous to surrounding structures.

The serviceability state must be verified in:

- External stability – settlement of the slope foundation
- Internal stability – post construction strain in the reinforcement and creep of soil

The design life is the period for which all serviceability criteria need to be met. Common serviceability limit states may eventually lead to ultimate limit state failure through progressive deterioration:

- Strains or movement of the facing that could affect the visual appearance of the facing or result;
- Deformations in the facing that could affect the serviceability of any adjacent structures, service or infrastructure;
- Cracking of hard facings (when used);
- Excessive bulging of soft or flexible facings (where used).

Limits are set as to what would be acceptable to limit damage, or it may be human perception of what is dangerous. The only method to check the displacements with a soil nail system is numerical analysis.

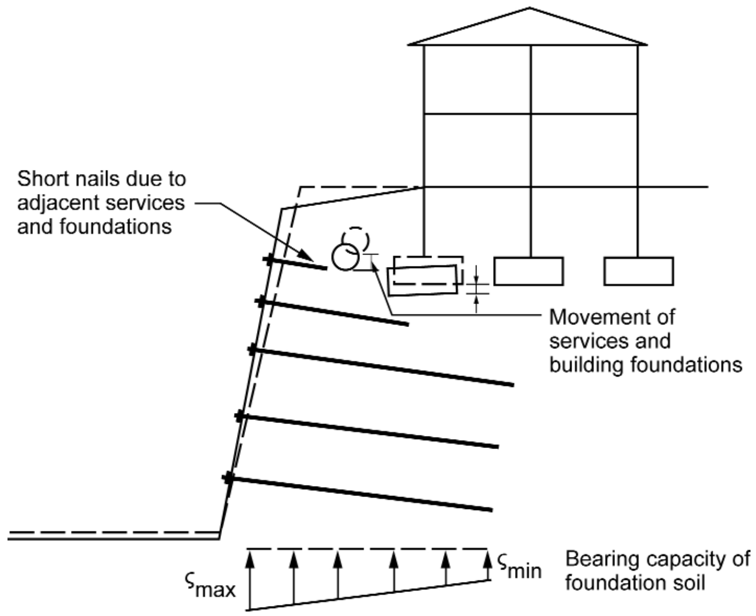


Fig 2.8 Illustrations of serviceability limit states for soil-nailed slopes and walls

2. 5 Conceptual design

The design of a soil-nailed wall or slope comprises two main stages: conceptual, which is followed by detailed design.

The first consists to identify the characteristics and the properties that describe the site used for the work. That includes the study of the site in its global properties and the accurate study of the soil characterizing the site. Both require tests, either lab or in-situ test.

Conceptual design is only the first stage in any project involving soil-nailed walls or slopes.

The factors to be considered for soil nailing are:

- Risk based approach;
- Characterisation of the ground;
- Groundwater;
- Construction sequence and buildability;

- Site constraints.
- Deformation.

2.5.1 Layout and spacing of nails.

The conceptual design concludes with the preliminary layout, angle of installation and lengths of the nails. The most important factors that influence these are:

- Ground strength;
- Height of face;
- Angle of face;
- Type of nail (drilled and grouted, or driven);
- Unit pullout resistance;
- Environmental constraints;
- Facing type (rigid or flexible).

Generally, nails are installed in rows at a slight inclination below the horizontal of between 5 and 20. For grouted nails, this is to permit gravity installation of the grout. While it is most practicable to make all the nails inclined at the same angle to the horizontal, different layouts may be required in special cases.

The spacing of the soil nails reflects the choice of facing type as well as overall stability requirements. Maximum horizontal and vertical nail spacing are typically in the range of 1.0 – 2.0 m, that is due to the behaviour of the soil as a coherent reinforced soil block becoming insignificant if the area covered by one nail is upper than 6mq in a rigid facing structure or upper than 2-4mq for a flexible facing structure (Phear *et al*., 2005).

2.5.2 Nail orientation

It is important to define the nail orientation to achieve to a cost effective and efficient. Many researches have been carried out to understand and find which nails' configuration could be the most effective, hence which practical angle of installation it is better to use in a soil nailed slope.

One important study by Johnson et al (2002) has shown that for a nail intersecting a failure plane inclined at 60 to the horizontal in a soil with an internal friction angle ϕ' of 25, the most effective nail inclination was 35° and it is possible to see this experience applied to a 6 metres high slope reinforced with a single 6 metres long. It is possible to see how the nail installed at optimum tensile has a short length (2.3m) in the resistant zone and little depth (1.2m) of overburden. Although the nail installed at 15 below the horizontal has an efficiency of 64% of the nail installed at the optimum angle, it has nearly twice the length in the resistant zone and more than four times the average overburden.

Based on the above theoretical analysis, it would appear that the optimum design angle is between about 10 and 15 below the horizontal (fig. 2.9).

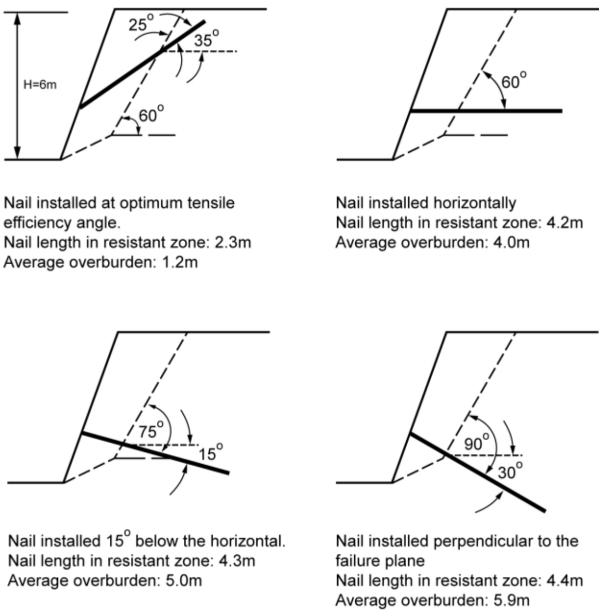


Fig. 2.9 Influence of nail inclination

2.5.3 Nail tendon design

The design tensile strength of the nail tendon (T_{nd}) is calculated as follows:

$$T_{nd} = \frac{f_y A_s}{\gamma_s}$$

γ_s, a partial factor for reinforcement material, is 1.05 for steel in tension and about 1.3 for geosynthetics (Eurocode7, 2004). As and f_y should be the values applicable at the end of the design life.

2.5.4 Detailed design

It is important to say that many codes and methods exist and every nation still uses its guidelines and norms but nowadays the principal guideline for European design is the Eurocode 7 .

For both serviceability limit state and ultimate state cases, design soil strengths are obtained by dividing (and so reducing) the characteristic strengths by a partial factor as follows:

$$Design\ strength = \frac{Characteristic\ strengt}{Partial\ factor}$$

For serviceability limit states, all partial factors for soil strength are 1.0.

2.5.5 *Pullout* resistance

The *pullout* resistance of a soil nail is dependent upon the overburden pressure of the soil, the vertical and lateral pressure around the nail and the nail/soil interface friction. The design nail resistance is the lowest of the following:

1) The *pullout* resistance between the soil nail and the ground.
2) The *pullout* resistance between the nail tendon and the grout (for grouted nails).

If the aim is to achieve a ductile slope failure mechanism, Criterion 1 is preferred.

The ability of a soil nail to generate sufficient *pullout* resistance (soil/nail) is of fundamental importance to the stability of a soil-nailed slope or wall. The ultimate *pullout* resistance of a soil nail is a function of the following:

- Soil type;
- Surface roughness;
- Drilling or installation technique;
- Time that the drillhole is left open and ungrouted (if this is too long, it will probably reduce the ultimate pullout resistance)
- Grout pressure (if grouted);
- Nail diameter;
- Nail length in the active zone;
- Nail length in the resistant zone;
- Elasticity of the tendon;
- Time (for soils susceptible to creep action);
- Presence of groundwater

There are at least five methods of determining the *pullout* resistance, as discussed below.

1. Empirical correlations and charts.
2. *Pullout* test.
3. Undrained shear strength methods (only for cohesive soil).
4. Effective stress methods (for cohesive and for granular soils).
5. From pressumeter tests.

Confirmation of ultimate *pullout* resistance is particularly important since a large proportion of failures of soil-nailed slopes and walls results from overestimation of the *pullout* resistance of the nails. Site *pullout* tests should be considered to be an extension of the design process.

2.5.6 External stability checks

The types of internal stability are described in par. 2.4.

The following external failure modes should be considered in the analysis of soil-nailed walls or slopes:

- Overall stability;
- Sliding failure;
- Bearing failure.

“Sliding stability”

Analysis of sliding stability considers the ability of soil-nailed walls or steep slopes to resist sliding along the base of the soil-nailed block in response to lateral earth pressure behind it. Sliding failure may occur when the lateral earth pressure exceed the sliding resistance along the base. Such failures can occur if there is a weak horizontal, or nearly horizontal, seam or zone at or slightly below the toe of the wall or slope.

"Bearing capacity"

Very rarely, bearing capacity may be a concern when a soil-nailed wall or steep slope is excavated in soft fine-grained soils. Since the soil-nailed block does not extend below the base of excavation, the unbalanced load caused by the excavation may cause the base of the excavation to heave. This may result in a bearing capacity failure of the foundation of the wall or slope.

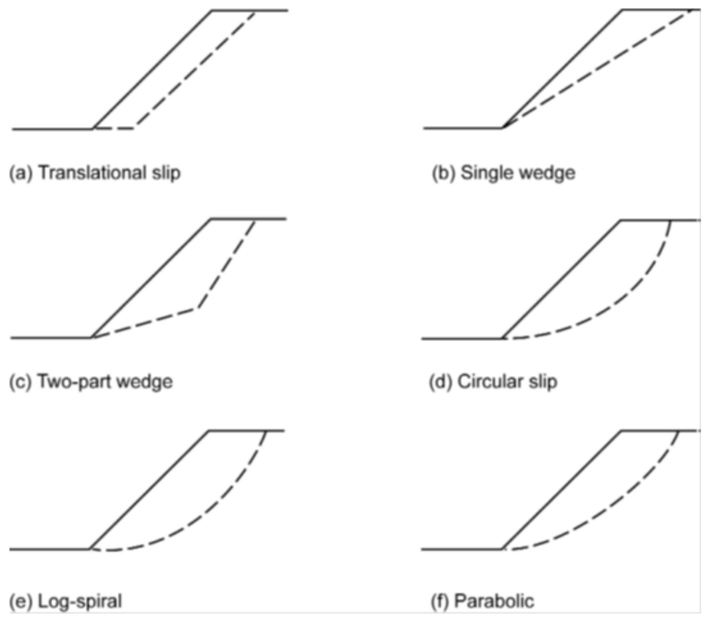
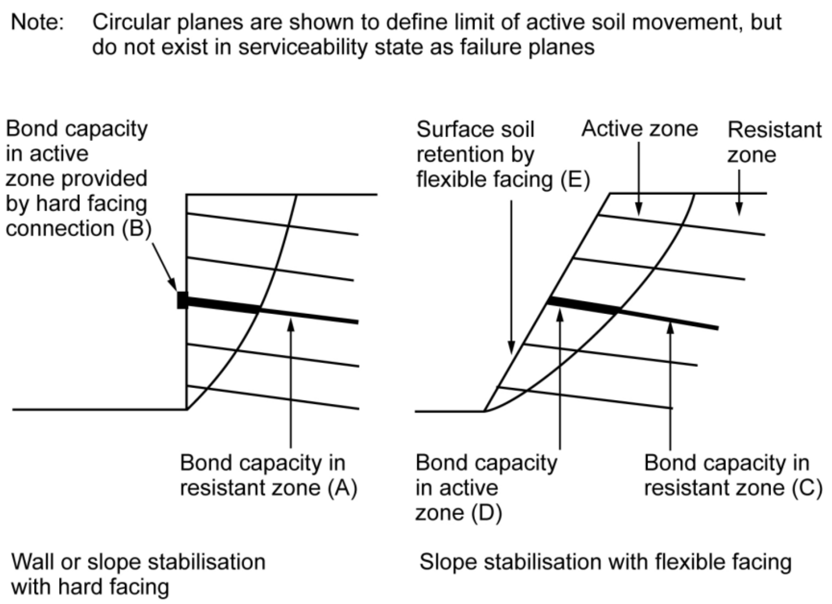


Fig 2.10 Failure surfaces used to assess stability of slopes (from Johnson et al, 2002)

3. FACING

Where soil nail are used to stabilise an existing slope, or to construct a new slope, they do not stabilise the surface soil. This is done by means of head plates and /or a facing. Separate measures to retain the surface (and near-surface) soil need to be adopted and integrated with the soil nail system. The facing system (hard, flexible or soft) can modify the internal failure mechanisms. The larger and smaller components of load transfer to a working soil nail are summarised in fig(3.1).



Major components of load transfer	Major components of load transfer
• bond capacity in the resistant zone (A) • plate bearing capacity to transfer load from facing structure (B) • strength of facing structures to retain active soil (B)	• bond capacity in the resistant zone (C) • bond capacity in the active zone (D) • surface soil retention by flexible facing (E)
Minor components of load transfer	**Minor components of load transfer**
• bond capacity in the active zone • bond capacity to the hard facing structure	• plate bearing capacity to supplement bond capacity of the active zone

Fig 3.1 Comparison of the larger and smaller components of load transfer between hard and flexible facing system

As the bond stresses transferred to the facing varies depending on the type and stiffness of facing.

It is important to observe the empirical correlation based on previous experience, collected by Bruce and Jewell (1987), of different features of the different types of facing .

They derived four parameters to allow comparison between the design of different projects. These are:

- Length ratio (*L/H*)

$$\frac{L}{H} = \frac{Maximum\ nail\ length}{Excavation\ height}$$

- Bond ratio (B_r)

$$B_r = \frac{Hole\ diameter\ \times\ nail\ length}{Nail\ spacing}$$

- Strength ratio (S_r)

$$S_r = \frac{Nail\ diameter^2}{Nail\ spacing}$$

- Performance ratio (P_r)

$$P_r = \frac{Outward\ \ movement}{Excavation\ height}$$

The most useful of these ratios for conceptual design is the length ratio and examples are presented from Table 3.1 to Table 3.3, there appears to be little correlation between slope angle, length ratio, area per nail and material type. That could be due to the fact that there is confusion over design methods.

However, there is a correlation between facing type and slope angle (steeper angle need hard facing and shallow angle could be built without a facing system)

It is possible to see how the selection and detailing of an appropriate facing for soil nailed slopes and walls just as important as the design of the soil nails themselves and is fundamental to the performance of the soil nailed slope. Where the surface of a wall or slope has proven long-term stability (such as an existing vegetated slope or stabilisation of an existing retaining wall) facing may be omitted by design. Usually a facing is required and its selection needs to consider the site constraints, and environmental and aesthetic requirements. As discussed above, the major role of the facing is to stabilise the surface (and near-surface depth) of the ground between the nails. It provides lateral confinement for the retained soil between the nail head locations. Progressive shallow failure will occur if the facing, precast panels or vegetation. There are three commonly used facing types:

1. Soft facings.
2. Flexible structural facings.
3. Hard structural facings.

3.1 Soft facings

These perform no long-term role but provide stability while vegetation becomes established. Their primary purpose is to retain the vegetation layer and topsoil and to prevent surface erosion. Typically the may be used on structures with a relatively shallow slope face. Material commonly used for this purpose are geogrids, cellular geofabrics, geosythetic sheet, light metallic mesh/fabric, or degradable coir mats. Various methods and proprietary systems of fixing such materials to the face are available. Such facings should not be used for slopes steeper than the angle at which the soil forming the slope surface is stable naturally (soil nailing may be needed in such cases to increase the stability of the slope against deeper-seated slips).

The long-term effectiveness of a structure with this type of facing is dependent on the growth, and subsequent maintenance, of the vegetation. It is also dependent on adequate drainage. This may be available naturally or may need to be installed as part of the soil-nailing works.

The characteristics for conceptual design for soft facing system are presented in table 3.1:

Soil type	Slope angle to horizontal (degrees)	Length ratio	Area per nail [m^2]	Source
Glacial till (north-east England)	About 30	0.9-1.2	0.5-2.3	Unwin (2001)
Poorly compacted cohesive fill over glacial till (north-west England)	28	0.9-1.25	5.8	Martin (1997)
High plasticity clay	24	1.38	2.4	Johnson *et al* (2002)

Tab 3.1 Empirical characteristic for flexible facings (in descending order of slope angle)

3.1.1 Design of soft (non-structural) facings

The primary function of a soft facing is erosion control and to support establishment of vegetation on the face of the slope. With such facings, the nail heads fix the facing to the ground. A soft facing will not:

- Be very effective against ravelling, and will therefore rely on the vegetation to provide protection against this
- Contribute significantly to slope stabilisation between the nails
- Guarantee the group action and integrity of soil nails. The nails will simply behave individually against destabilising force.

The use of soft facings should therefore be limited to shallow slope angles (of up to about 30° to the horizontal)

3.1.2 Construction of soft (non-structural) facings

The long-term effectiveness of a slope with a soft facing depends on the growth, and subsequent management, of the vegetation. For this reason, existing natural vegetation should be maintained where appropriate. The selection of suitable vegetation types or species is a specialist subject, but the primary factors include:

- The local climatic conditions
- Orientation of the slope face
- Rainfall pattern
- Topsoil and subsoil type
- Soil chemistry

Good practice is to vegetate the face with grass and/or shrubs. Seeding can be carried out by means of a seeded geotextile or hydro seeding. Consideration should be given to the time of year or season in which these systems are applied and the need for initial watering and protection.

When the performance of the facing is dependent on the vegetation, it should be applied as soon as is practicably possible. Before application, the exposed slope face may need to be protected to prevent degradation.

Secondary pins should be installed between the soil nails to control/restrict the movement of the soft facing until the vegetation becomes fully established. These need to be sufficiently robust to be satisfactorily and need to have the same durability as the other facing components. Where seed-impregnated mats are used, care should be taken to install them with the correct side upwards. The incorporation of a geotextile within turf rolled out and pinned to the slope reduces short-term instability caused by heavy rainfall.

3.2 Flexible structural facings

These structural facings provide long-term stability of the face of the soil-nailed structure by transfer of the soil load from the soil nails to the nail heads. The facing materials allow greater soil movement and minor bulging between the head plates should be expected, although this reduces with closer nail spacings. Materials used commonly comprise coated metallic meshes appropriately designed, in conjunction with the head plates, for the structural loads and durability requirements. Proprietary heavy rock meshes, which are used to prevent rock falls on steep slopes, also work well as flexible facing for soil-nailed slopes. Their opening sizes are typically about 80 mm x 80 mm. The mesh wires typically are plastic-coated to improve their durability.

Careful consideration needs to be given to the facing material's ability to resist the loads imparted by the nail heads and head plates to avoid failure by puncturing or rupture and/or excessive bulging under working conditions. Only a few geosynthetics (including geogrids) are suitable for flexible facings, because the punching resistance of many of them is too low to fulfil satisfactorily the static (i.e. structural) function required for these facings. This often results in such geosynthetics being suitable only for soft facings.

Sometime the mesh is placed side by side with a geotextile impregnated with grass seed and fertiliser to achieve rapid vegetation growth. Such facings should not be used for slope steeper than about 60-70° to the horizontal and will require either jointing or lapping to provide structural continuity between the soil nails. Allowance needs to be made in the facing design for the method, and health and safety aspects, of replacing the facing material, as this will normally have a shorter design life than the soil-nailed structure.

Fig. 3.2 Structure with flexible facing

Commonly used soil nails are made of steel bars covered with cement grout. The grout is applied to protect the steel bars from corrosion and to transfer the load efficiently to nearest stable ground. Some form of support, usually wire mesh-reinforced *shotcrete*, is provided at the construction face to support the face between the nails and to serve as a bearing surface for the nail plates. The use of wire mesh-reinforced *shotcrete* facing can require the mobilization of a specialty contractor and increase the cost of a project. Use of flexible facing material such as geosynthetic, steel wire, or chain link without *shotcrete* could provide significant savings. The use of high strength steel wire mesh wire is economical, eliminates the need of drainage, and facilitates the greening of the slopes (Geobrugg 2007). The mechanism of increased stability of the soil nailed walls can be explained by :

1) the increase in the normal force and the shear resistance along the potential slip surface in frictional soil ;
2) the reduction in the driving force along the potential slip surface in both frictional and cohesive soils.

When the wire mesh is used as a facing material, the mesh and nails act together as a system to provide stability to the slope, preventing deformations in the top layers and restricting movement along planes of weakness. With the high strength of the mesh, it is possible to pre-tension the system against the slope, and the pre-tensioning enables the mesh to provide active pressure against the slope, preventing break-outs between the nails (Geobrugg 2007).

Soil nailing has proven to be an effective and economical means of protecting unstable slopes and providing temporary shoring. Construction facing alternative such as steel wire mesh and is considered in the present research.

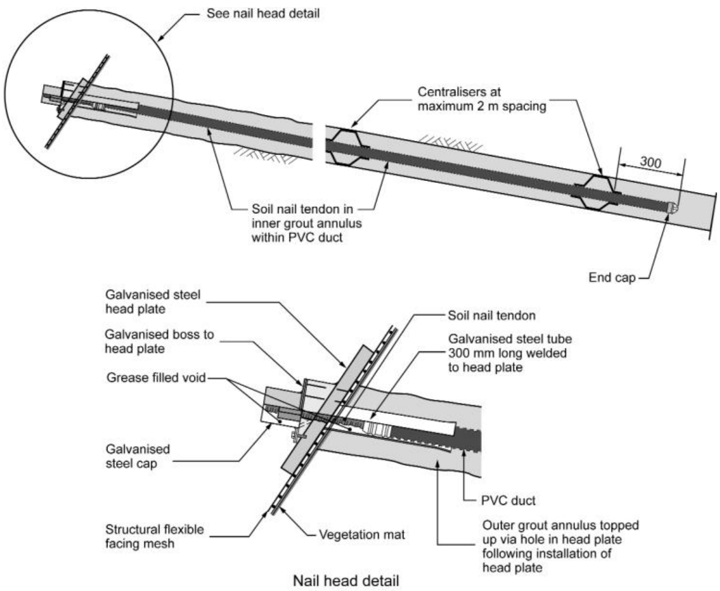


Fig. 3. 3: 120-year-design-life drilled and grouted soil nail with flexible facing.

The characteristics for conceptual design for flexible facing systems are presented in table 3.2:

Soil type	Slope angle to horizontal (degrees)	Length ratio	Area per nail [m^2]	Source
Silty clay and clayey sand (Tunbridge Wells Sand and Wadhurst Clay)	70	0.63-1.1	2.3-2.9	Pedley and Pugh (1995)
Old cohesive embankment fill (north-east England)	About 70	0.8-1.0	0.5	Martin (1997) (temporary works)
Mercia Mudstone Group (marl)	68	2.2	1.0	Johnson *et al* (2002)
Firm to stiff sandy clay (glacial till)	68	1.3	2.25	Johnson *et al* (2002)
High-plasticity clay	57	1.0	0.8-1.5	Johnson *et al* (2002)
Various cohesive soils	Steeper than 45	0.42-1.0	/	Barley (1997)

Tab 3.2 Drilled and grouted soil nails – empirical length ratio and area of facing per nail for various soils, slope angles for flexible facings (in descending order of slope angle)

3.2.1 Design of flexible structural facings

Flexible facings are not suitable for large spans between nails. The entire system of flexible facing, head plates and the soil nails themselves needs to be rigorously designed to provide restraint at the slope surface, to provide a restraining force at the nail head and to satisfy the agreed durability and maintenance requirements.

Flexible facing design needs to consider:

- Nail spacing
- Soil type
- Potential deformation mechanisms
- Head plate dimensions
- Nail and soil stiffness
- Groundwater and drainage conditions
- Steepness of the slope and future role of the vegetation

There are few recognised design methods for flexible facings other than published by Ruegger et al. in 2001. The design considerations should include:

- Check the ultimate limit states. The serviceability limit state conditions (such as deformation) cannot be checked except by numerical analysis methods
- Check the punching resistance of the facing
- Calculate the head plate size based on the punching resistance of the facing
- Check that the structural capacity of all connections is adequate – in particular the detail for lacing together adjacent panels of mesh needs to have adequate capacity
- Check durability of the facing material itself and all connections for the design life.

The facing should be designed to resist potential out-of-balance forces, which can be transferred to the facing by failure of three-dimensional blocks between the nails. Such blocks can be conservatively modelled in two dimensions (ignoring side friction) using shallow two part wedge mechanisms or slip circles, to achieve force equilibrium. This is one approach for determining the force that the mesh need to carry. However, this is only an empirical method without experimental validation. Depending on the slope angle, the load on the facing may increase as the height below the top of the slope increases, and may be greatest at the toe of the slope. For medium and high-plasticity clays, the influence of water-filled tension cracks should also be considered.

The pore water pressures that need to be adopted in the calculations should be carefully considered.

The reinforcement strength adopted needs to be a characteristic strength reduced by partial factors appropriate to the required design life. For polymeric grids allowance should be made for the effects of creep over the design life.

If the load in the reinforcement is excessive, a grid of short nails should be added in between the other nails to carry some of this load. Alternatively, or in addition to the short nails, intermediate pins can be introduced between the soil nails to reduce the span of the facing further and thus to reduce further the tensile load that it needs to carry. The size of potentially out-of-balance blocks between the nails will also then be reduced.

Once such potential out-of-balance forces are less than the punching capacity of the facing, the remaining design to produce a satisfactory flexible facing needs to be via careful detailing rather than calculations.

The above design method is much simplified. In this research we study the way in which a flexible facing performs involves complex interaction between the soil, soil nails and the facing, and is dependent on their relative stiffness.

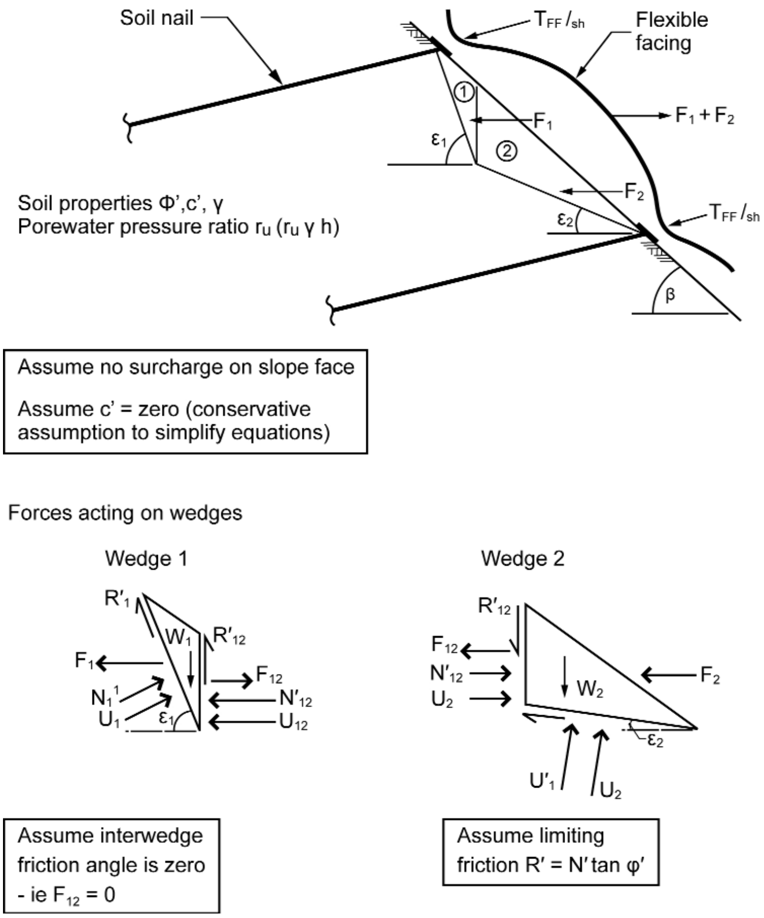


Fig. 3.4 Two-part wedge potential failure mechanism between rows of nails (based on HA 68/94 (Highways Agency, 1994))

Equation for force: total out-of-balance force per meter width, $F_{tot} = F_1 + F_2$, has to be less than the punching resistance of the facing.

$$F = \frac{W_1\,(\tan\varepsilon_1 - \tan\phi') + U_1\,\dfrac{\tan\phi'}{\cos\varepsilon_1}}{(1+\tan\varepsilon_1\tan\phi')} + \frac{W_2\,(\tan\varepsilon_2 - \tan\phi') + U_1\,\dfrac{\tan\phi'}{\cos\varepsilon_2}}{(1+\tan\varepsilon_2\tan\phi')}$$

Vary wedge sizes and angles, ε_1 and ε_2 to obtain maximum out-of-balance force, F_{tot} .

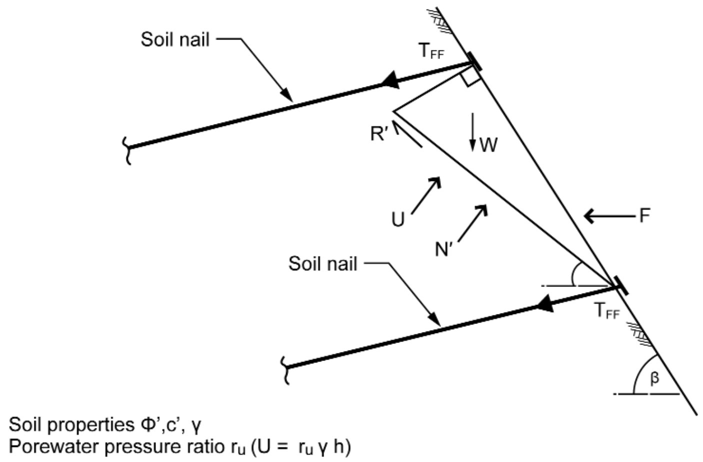


Fig.3.5 Simple wedge potential failure mechanism between rows of nails for steeper slopes (based on Ruegger et al, 2001)

3.2.2 Elements of the flexible facing system and their design

In the absence of definitive simplified design techniques for flexible facings, design approaches can be developed with finite difference numerical modelling techniques. Numerical modelling is chosen due to the absence of routine design methods for special structures of this nature and to:

- understand the load transfer mechanism;
- gain a robust understanding of likely face deformation;
- identify strain concentrations;
- refine the facing design;
- confirm that currently employed construction practices have a sensible factor of safety.

The analysis also facilitated comparison with the limit equilibrium predicted failure modes. A summary of the five principal elements of the facing system that could be taken in account and the role of the numerical modelling are presented below:

1. Secondary facing nails. To prevent failure between adjacent horizontal nails in the upper half of the slope where the primary reinforcement nails could be installed. The secondary nails provide additional support to the facing and extend beyond the active wedge immediately behind the cut face.

2. Structural facing mesh. Is possible to use steel mesh as the primary mechanism for face containment or hexagonal steel wire mesh, typically used for rock retention systems.

3. Vegetation and erosion mats. For the cutting to remain stable throughout the design life, it is important to establish vegetation on the cut face without allowing surface water runoff to erode the slope. The facing design first requires a vegetation mat to be placed against the cut face of the slope. The structural mesh is placed over this, followed by an erosion mat to prevent the washout of fines while the vegetation become established. The erosion mat usually comprises a hexagonal treble-twist wire mattress similar to that used in gabion baskets, with a plastic fabric bonded to the wire. On a 60° cut slope, this relatively flexible product could not be relied upon to provide a structural function to retain the soil behind the facing.

4. Head plates of variable size and thickness. For each row of nails the head plates for both the primary and secondary nails i.e. assessed in terms of both dimensions and thickness.

5. Concrete toe wall. If significant strains could be developed at the toe of the wall In the long term and subject to swelling and softening, these might propagate slope failure. To counteract this failure mechanism, a

reinforced concrete wall has to be designed to provide additional stiffness in the toe of the cutting. An additional “toe pin” could be also installed, to prevent the toe wall from rotating and kicking out.

3.2.2 Construction of flexible structural facing

Materials used for flexible facings include geogrids and coated metallic meshes, in conjunction with head plates. These facing materials need to have sufficient strength and durability and will require either jointing or lapping to provide structural continuity between the soil nails.

To minimise degradation of the slope face, the flexible facing should be applied after soil nail installation but before the next excavation stage. As with soft facing, intermediate/secondary pins are usually required between the nails to reduce the span of the facing. The length of the pins should be defined by design. The flexible facing should be securely fixed at the top, preferably above the upper row of nails.

Completion of the flexible structural facing includes the tying in of the facing material along vertical joints and at the base of the excavation. Up to this point temporary stability conditions apply and the potential for shallow translational movement of the soil under the flexible structural facing exists.

Typically, where a flexible structural facing is adopted, a vegetation layer is applied. Application of the vegetation layer or soft facing prevents degradation and softening of the soil nail face slope on to which the flexible structural facing is located. Hence it is important that this is applied and established without delay. On steeper soil-nailed slopes or in environments where growth of vegetation is unreliable (such as under bridges), a crib facing or other proprietary facing system may be used.

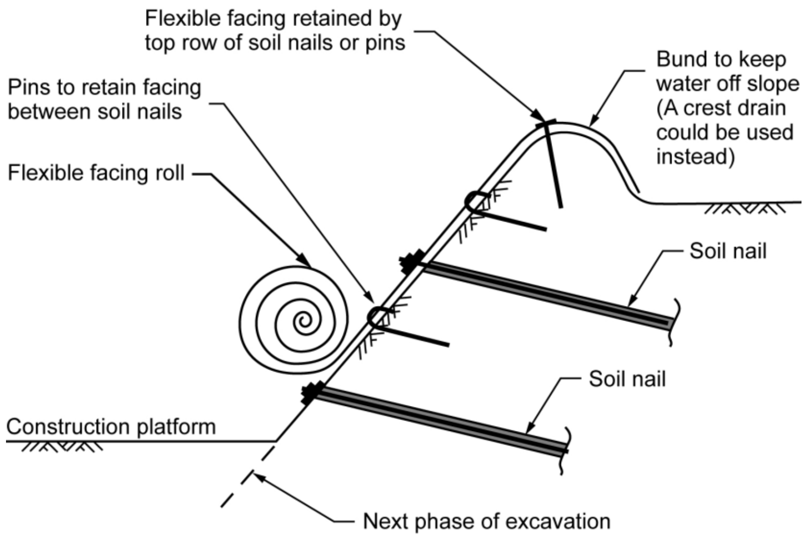


Fig 3.6 Schematic of the installation of a flexible facing (adapted from Barley et al, 1997a)

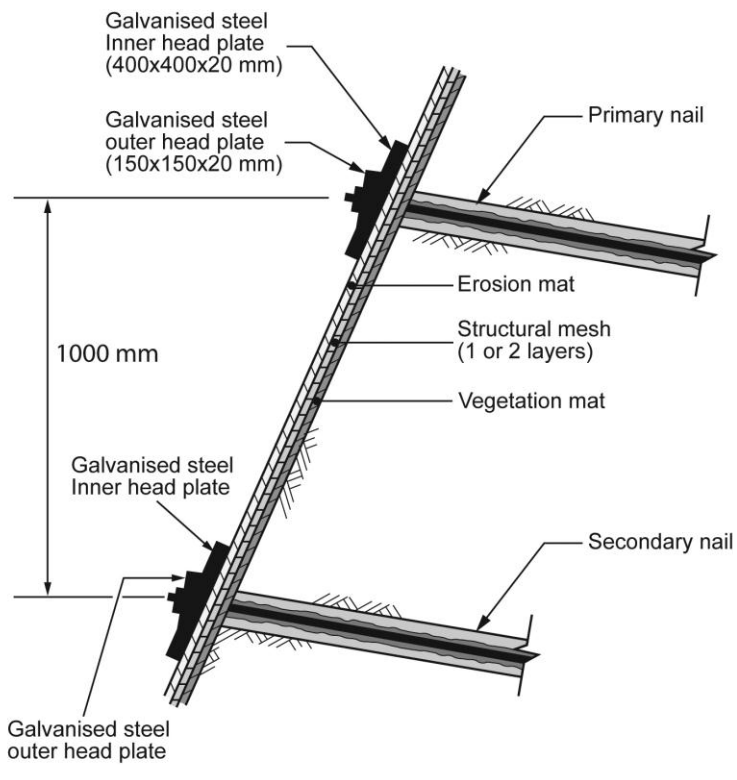


Fig 3.7 Schematic of the installation of a flexible facing (adapted from Barley et al, 1997a)

3.3 Hard structural facing

These perform the same function as the flexible structural facings but with less deformation. They generally comprise sprayed concrete reinforced with steel mesh. Most early soil-nailed structures were faced in this way. Other structural facing materials include conventional cast-*in-situ* concrete or precast concrete panels. Hard structural facings are often used where steep, or vertical, soil-nailed slopes are required because of the face loading to be resisted. Cost and aesthetic considerations, particularly for sprayed concrete, have limited their use on less steep soil-nailed slope faces where flexible structural facings can be used.

Unlike flexible structural facings, which usually are permeable, water pressure can readily build up behind the hard structural facing, so weep holes need to be included within the facing and/or a drainage system installed behind the facing.

3.3.1 Design of hard structural facings

The design sequence for hard facings generally includes the following steps:

- Decide on facing type based on performance and aesthetic requirements
- Determine the nail head forces
- Calculate head plate size and/or initial sprayed concrete thickness based on the punching resistance of facing
- Check the flexural resistance of facing and reinforcement detailing
- Check performance requirements under serviceability limit state conditions (crack width, deflection and durability) that are required for long-term applications.

Since the majority of resisting forces are generated along nails, only some of the maximum tension force in the nail is transferred to the facing. Therefore, soil nailing requires lighter facing elements than other conventional earth retention techniques.

The characteristics for conceptual design for hard facing systems are presented in table 3.3:

Soil type	Slope angle to horizontal (degrees)	Length ratio	Area per nail [m^2]	Source
Weakly cemented sand; silty sand; alluvial silt, sand and gravel (three sites)	70-90	0.5-0.8	1.5-2.8	Bruce and Jewells (1987)
Weathered schists, shales and mudstone (two sites)	75 and 80	0.55-0.75	2.0-2.3	Bruce and Jewells (1987)
Moraines and marl (five sites)	70-90	0.5-1.0	2.4-6.0	Bruce and Jewells (1987)
Soil investigated by Clouterre	70-90	0.8-1.2	2.5-6.0	Clouterre (1991)
Medium gravel over weakly cemented sand	90	1.0	1.5	Johnson and Card (1998)
Sandy clays and silty sands	80	1.0	0.7	Pedley (2000) (temporary works)
Clayey sand fill over firm sandy clay/ silty sand	70	1.0	1.0	Pedley (2000) (temporary works)

Tab 3.3 Drilled and grouted soil nails – empirical length ratio and area of facing per nail for various soils, slope angles for hard facing systems (in descending order of slope angle)

3.3.2 Construction of hard structural facing

Hard structural facing perform the same function as flexible structural facing but generally comprise steel- mesh-reinforced sprayed concrete. Other structural facing materials could include conventional cast-*in-situ* concrete or precast concrete panels.

Before using sprayed concrete on the permanent works, trials are recommended.

There is a natural tendency for temporary stability to become more critical as the slope face angle increases. In some case excavation, therefore, soil nail construction and the application of sprayed concrete are carried out in bays of limited width.

Alternatively, the nails may be installed before the final cut slope is excavated, followed by rapid excavation and spraying the concrete in stages. The speed of sprayed-concrete application, achievable strength gain and stiffness are significant advantages in these conditions.

Sprayed concrete may be applied either as a single layer or in two phases. The latter may be adopted where temporary excavation stability is marginal, as the thin initial layer protects the slope face and enhances the localised stability of the soil.

3.4 Head plates

The correct sizing of head plates is important so that they do not fail through insufficient bearing capacity. For slope with flexible facings, head plates play an important role in promoting arching between the nails.

3.4.1 Design of head plates

The head plate should be correctly sized to prevent bearing failure and to promote soil arching and hence to reduce local surface instability between the soil nails. The head plate should also be in good contact with the soil behind it to prevent ravelling. If the nails are too far apart and/or the head plates are too small then the soil may fail between the nails. For the design of head plates for hard facings.

The head plate transfers the load from the facing material to the soil nail and should be parallel to and in contact with the facing material to avoid localised unacceptably high stress in the facing material and the risk of bursting failure around the head plate. The head plate detail needs to take account that the head plate is not usually perpendicular to the soil nail.

There is little previously published advice on how to estimate the design nail head load for flexible facings (T_{ffd}), but it should first be derived from the calculations for the flexible facings. The head plate should then be sized for bearing capacity and its resistance to punching through the facing should be checked.

The interaction between the bearing pressure of the head plate and the deflection of the flexible facing is complex and the suggested approach greatly simplifies this.

For shallow slopes, Terzaghi's basic bearing capacity equation for square footings gives a simple preliminary estimate of the plate size required. Note this gives ultimate load whereas the methods below are for design parameters and loads.

$$q_u = 1.3\, c\,'N_c + \gamma\, D\, N_q\ + 0.4\, a\, \gamma\, N_\gamma$$

Appendix E of HA 68/94 (Highways Agency, 1994) includes a method for calculating the size of head plates for flexible or soft facings. This checks the adequacy of the head plate in bearing, to guard against front face pullout, using a lower bound and an upper bound solution, as shown in Figures 8.8a and 8.8b respectively. The lower bound solution is simpler and is presented below. It is conservative because it is two-dimensional and ignores side friction. It is more applicable for steep slopes (up to 60°).

$$a = \left(\frac{T_{ffd}}{\eta}\right)^{1/3}$$

where:

$$\eta = \frac{\gamma(1 - r_u)\tan\beta\ e^{3\left(0.785 - \frac{\phi'}{114.6} + \frac{\omega}{57.3}\right)tan\phi'}}{2\cos\left(0.785 + \ \frac{\phi'}{114.6}\right)\cdot(1 - sin\phi')}$$

where ϕ', β, and ω are in degrees.

The upper bound mechanism comprises a two-part wedge acting passively.

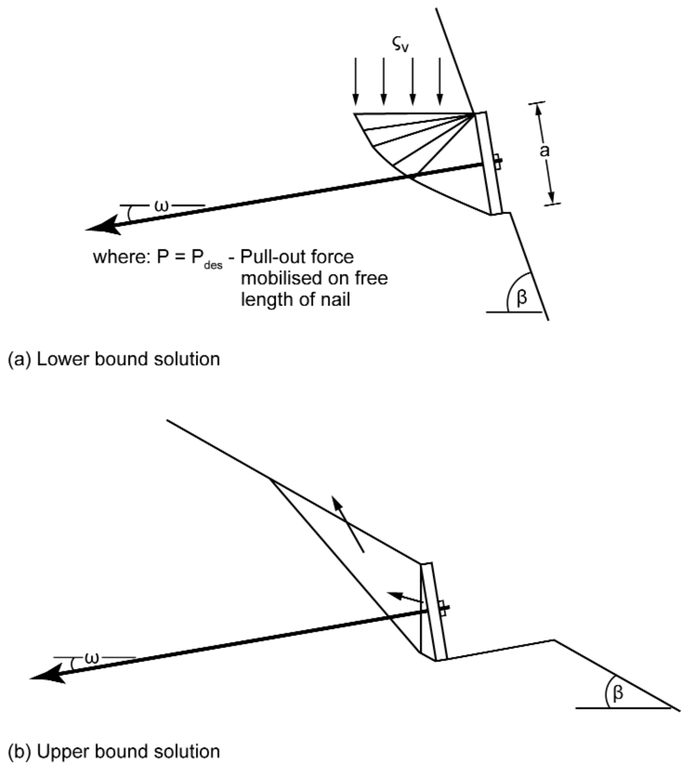


Fig. 3.7 Calculation of head plate bearing capacity (from Highways Agency, 1994)

Having determined the plan area of the head plate, the thickness then needs to be calculated to avoid overstressing the head plate in bending.

Such checks on head plate bending overstress are not required where the head detail is a concrete pad. Any stabilizing influence or benefit derived from the use of a flexible facing is ignored in the above approach, so it may be conservative in some situations. When used with flexible facings, the design of head plates can be optimized by determining the punching resistance and deformation characteristics of the proposed facing system. Where such tests are not available, the simplified approach given above can be used to size the head plate.

4. NUMERICAL ANALYSIS OF SOIL NAILED WALLS WITH FLEXIBLE FACING

The main aim of this study is to understand the behavior of a soil nailed structure with a flexible facing, hence to find when it represents a cost effective solution and which limits it shows. For this reason it is important to understand how every element composing the structure acts to guarantee the stability of the structure itself. To do that, also comparisons with other different types of facing were run.

Although several thousand soil nail structures have been constructed worldwide, only a limited number have been instrumented to provide performance data to support design procedures and ensure adequate performance (FHWA, 2003).

The main shortcoming of the limit equilibrium design method is that they do not give a prediction of stresses and deformations. They also do not consider the deformation required to mobilize the resisting forces in the soil and soil nails. These methods cannot therefore provide a well define description of the contribution of each soil nail to overall stability.

Stresses and deformation can be predicted approximately using empirical correlation and in many cases they have limitations

In situations where more confidence is required, a higher level of analysis should be adopted by using numerical modeling such as finite element and/or finite difference methods. The accuracy of numerical modeling depends on the quality of data acquired, the estimation of in-situ stress and soil stiffness and the availability of good case histories to calibrate numerical models.

Observation during construction are essential and cannot be replaced by numerical modeling. Even using numerical modeling, it is still relatively difficult to predict stresses and displacements. The accurate modeling of the grout/soil interface is also difficult, as mobilization of tension forces is often not directly proportional to facing deflections and/or construction stages (Phear *et al.*, 2005)

A series of *FLAC* finite differences models were constructed to simulate the performance of different soil nail slopes with steel wire mesh. Numerical modeling of the soil nailed wall was conducted using *FLAC*^{3D}. The numerical modeling was run for different types of facing, most of all where flexible facing type.

Seven models were developed to simulate the different behavior of different slopes, including the effects of different type of facing, the effects of different slope angles to the horizontal and the effects on the behavior using different spacing of the nails.

Characteristics of the models are shown in Table 4.1.

script	facing	slope's angle [°]	spacing [m]
1	flexible	45	1.5
2	flexible	60	1.5
3	flexible	75	1.5
4	flexible	60	2.0
5	hard	75	1.5
6	soft	45	1.5
7	soft	60	1.5

Tab 4.1 Implemented models

A further model was developed to have a comparison with a real structure. That is a hard facing structure built in Istanbul with inclinometers to register the displacements occurring at the surface.

Properties of soil, cable, and grout used for the numerical modeling are listed it table 4.2 :

Soil	
Young's modulus	50 [MPa]
Friction angle	32°
Cohesion	0.005 [MPa]
Density	1840 [kg/m^3]
Nails	
Length	7 [m]
Diameter	ϕ20 [mm]
Young's modulus	200 000 [MPa]
Yield stress	2500 [MPa]
Grout	
Diameter	Φ100 [mm]
Young's modulus	35000 [MPa]
Cohesion	7.5 [MPa]
Interface friction angle	35°

Tab 4.2 Elements' properties

The properties of facings used for the numerical modeling are given in Table 4.3.

Steel hexagonal wire mesh (flexible)	
Young's modulus	8500 [MPa]
Poisson's ratio	0.33
Shotcrete (hard)	
Young's modulus	30000 [MPa]
Poisson's ratio	0.2

Tab 4.3 Properties of facings

The model boundary conditions were fixed by standard fixities, where side vertical boundaries were fixed in horizontal x-direction but free to move vertically, while the bottom boundary was restrained from any movements in all directions. The initial stresses of the model were calculated by gravity loading to reach its equilibrium.

Fully drained Mohr-Coulomb Model was implemented as representing soil and interfaces behavior. The elastic-plastic Mohr-Coulomb Model represents a "first-order" approximation soil or rock behavior. It is recommended to use this model for a first analysis of the problem considered. Mohr-Coulomb Model involves five input parameters: Young's modulus (E) and Poisson's ratio (υ) for soil elasticity, internal friction angle (ϕ) and cohesion (c) for soil plasticity and an angle of dilatancy (ψ).

The secondary aim of this research is to simulate and understand the real behavior of flexible facing, hence how the stresses develop in the different element by their own stiffness. In particular it is focused on the stress developed in the facing and how it could be design to be cost effective and safe at the same time. To study this behavior a 10.5 meters height and 3 meters width slope was implemented.

4.1 Elements modeling

In this section the characteristics of the different elements are shown to see how they work, their function and their mechanical behavior . In particular, it is important to note the different element used to model soft/flexible and hard facing elements. For the first type a *geogrid* simulation was chosen and not a *shell* element as for the hard facing, used as usual in a numerical model. That is due to the membrane behavior of the *geogrid* element, not resisting to compressive stress. More information of their mechanical behavior are written below.

4.1.1 Mechanical behavior of a *geogrid* element

This type of element is used to model a soft/flexible facing element.

A *geogrid* is embedded in the interior of the $FLAC^{3D}$ grid.

These stresses, consisting of an effective confining stress, σ_m, and a total shear stress, τ , are balanced by the membrane stresses that develop within the *geogrid* itself. These membrane stress resultants are denoted by *N* in fig4. The interface behavior is represented numerically at each *geogrid* node by a rigid attachment in the normal direction and a spring-slider in the tangent plane to the *geogrid* surface. The orientation of the spring-slider changes in response to relative shear displacement u_s between the *geogrid* and the host medium.

The spring-slider carries the total shear force acting over the tributary area on both sides of the *geogrid* surface. Also, the effective confining stress is assumed to be acting equally on both sides of the *geogrid* surface.

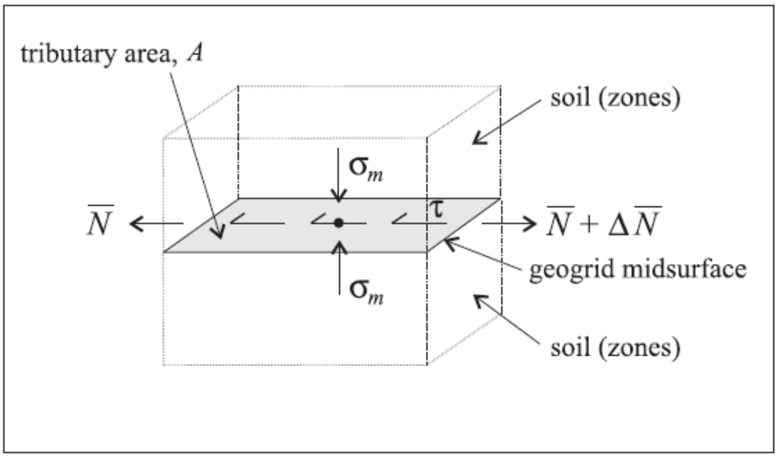


Fig 4.1 Stresses acting on the *geogrid* element

The shear behavior of the *geogrid*-soil interface (fig4.1) is cohesive and frictional in nature, and is controlled by the coupling spring properties of:

1. stiffness per unit area, k;
2. cohesive strength, c;
3. friction angle, φ;
4. effective confining stress, σ_m.

The effective confining stress, σ_m, acts perpendicular to the *geogrid* surface, and is computed at each *geogrid* node, based on the stress acting in the single zone to which the node is linked.

The value of σ_m is taken as $\sigma_m = \sigma_{zz} + p$, where p = pore pressure.

This type of element was used because the stress in coupling spring is always positive and that is because is acting on *geogrid* in tangent plane on *geogrid* surface. Exactly the ideal behavior of a steel wire mesh.

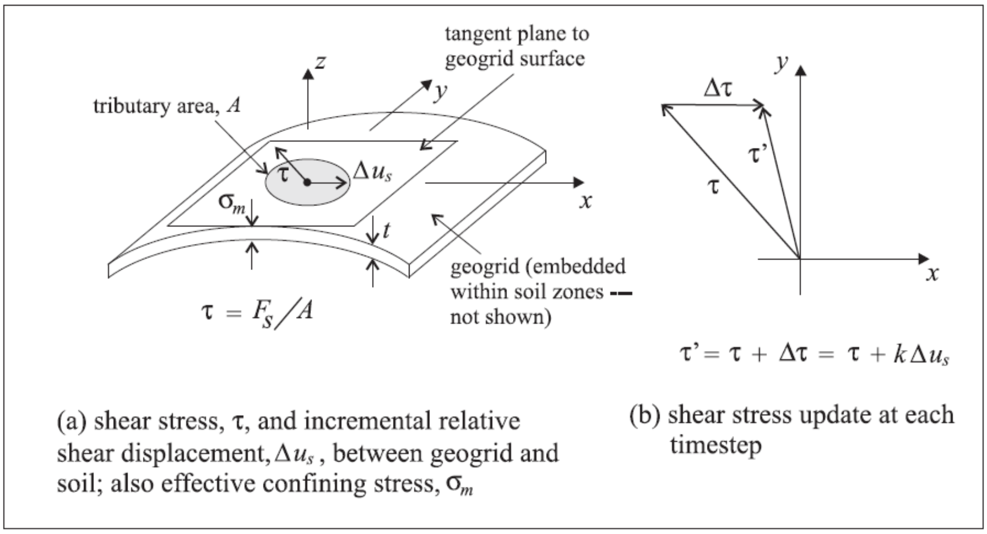


Fig 4.2 Idealization of interface behavior at a *geogrid* node

4.1.2 Mechanical behavior of a *shell* element

This type of element is used to model a hard facing element.

A shell-type structural element is assumed to be a triangle of uniform thickness lying between three nodal points.

Each *shell*-type element provides a different means of interacting with the grid. The structural response of the shell is controlled by the finite element assigned to the element. There are five finite elements available: 2 membrane elements, 1 plate-bending element and 2 shell elements.

Because these are all thin-shell finite elements, *shell*-type elements are suitable for modeling thin-shell structures in which the displacements caused by transverse-shearing deformations can be neglected.

Each *shell*-type element has its own local coordinate system shown in fig4. This system is used to specify applied pressure loading.

The shell-type SEL coordinate system is defined by the locations of its three nodal points, labeled 1, 2 and 3 in fig. 4.3. The *shell*-type element coordinate system is defined such that:

1. the shell-type element lies in the xy-plane;
2. the x-axis is directed from node-1 to node-2;
3. the z-axis is normal to the element plane and positive on the "outside" of the shell surface.

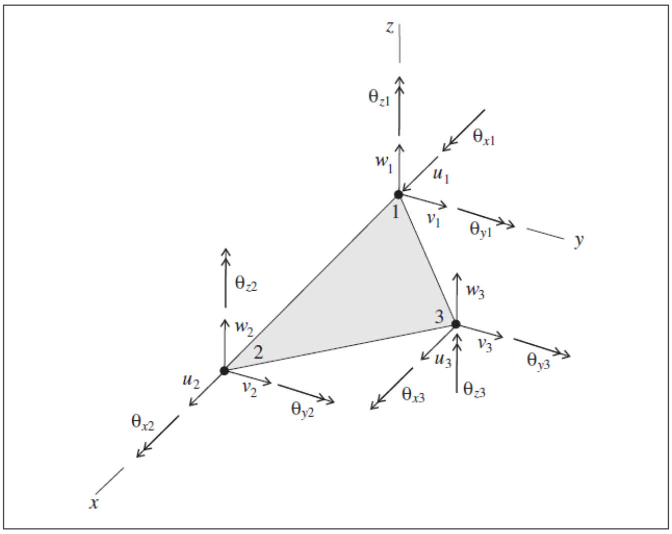


Fig. 4.3 Shell-type coordinate system and 18 degrees of freedom available to the shell finite elements

4.1.3 Mechanical behavior of a *cable* element

In *FLAC3D* if bending effects are not important, *cable* elements are sufficient, because they provide a shearing resistance (by means of the grout properties) along their length.

Each cable structural element is defined by its geometric, material and grout properties. A *cable* element is assumed to be a straight segment of uniform cross-sectional and material properties lying between two nodal points.
The *cable* element behaves as an elastic, perfectly plastic material that can yield in tension and compression, but cannot resist a bending moment. A cable may be grouted such that force develops along its length in response to relative motion between the cable and the grid. The grout behaves as an elastic, perfectly plastic material, with its peak strength being confining stress dependent, and with no loss of strength after failure.

Each cable has its own local coordinate system, shown in fig 4.4. This system is used to define the average axial cable direction.

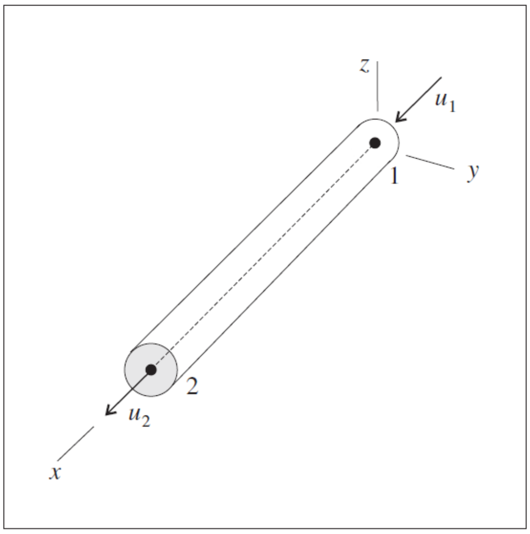


Fig 4.4 Cable element

4.2 Macro-scale model

The model represents a 10.5 meter high slope, filled with a granular incoherent soil with a very low cohesion.

To represent the reality as much as possible, the soil nailing model was built step by step. Every step figures a cut with a height equal to the spacing of the nails to simulate the real behavior of the soil hence the behavior of the structure.

In this way the slope model can reach the equilibrium in every step and if it does not, it is possible to see where it begins to be unstable hence where a failure mode is developing.

The first step consists of the consolidation of the soil block, under the effect of his own weight by gravity (fig 4.5).

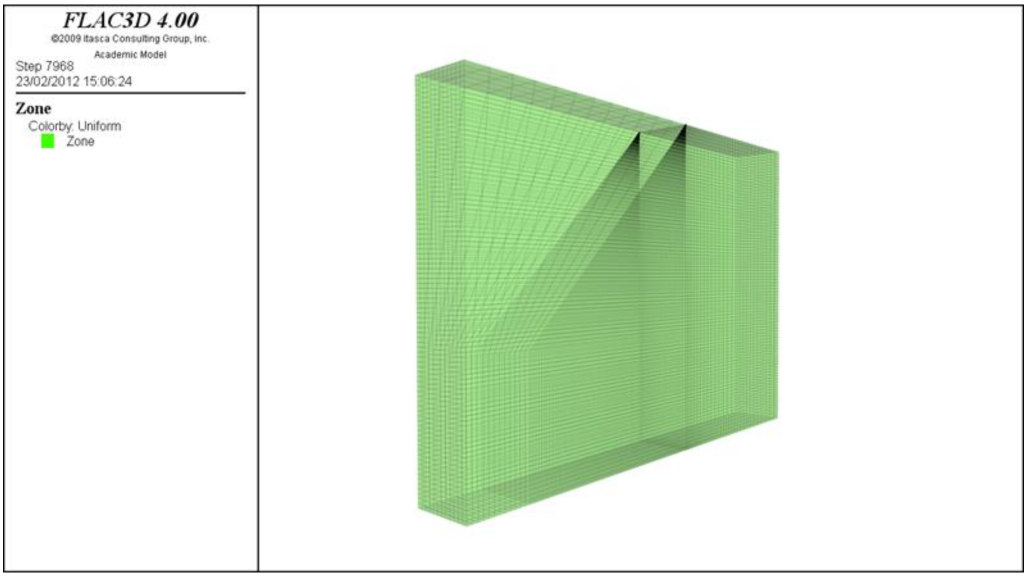


Fig 4.5- Zone

From figure 4.6 To 48. different “cuts” are shown.

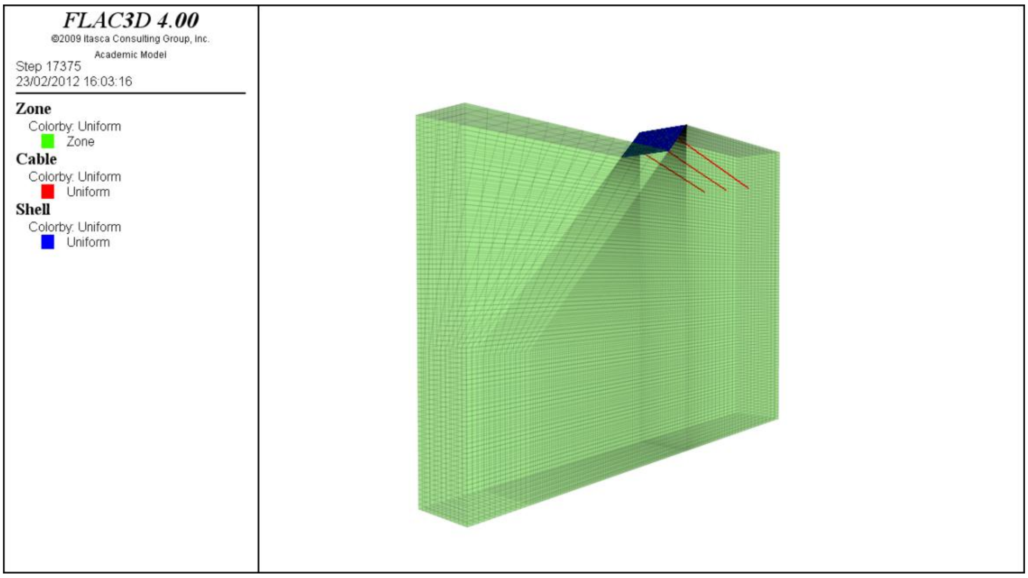


Fig. 4.6 – First “cut”

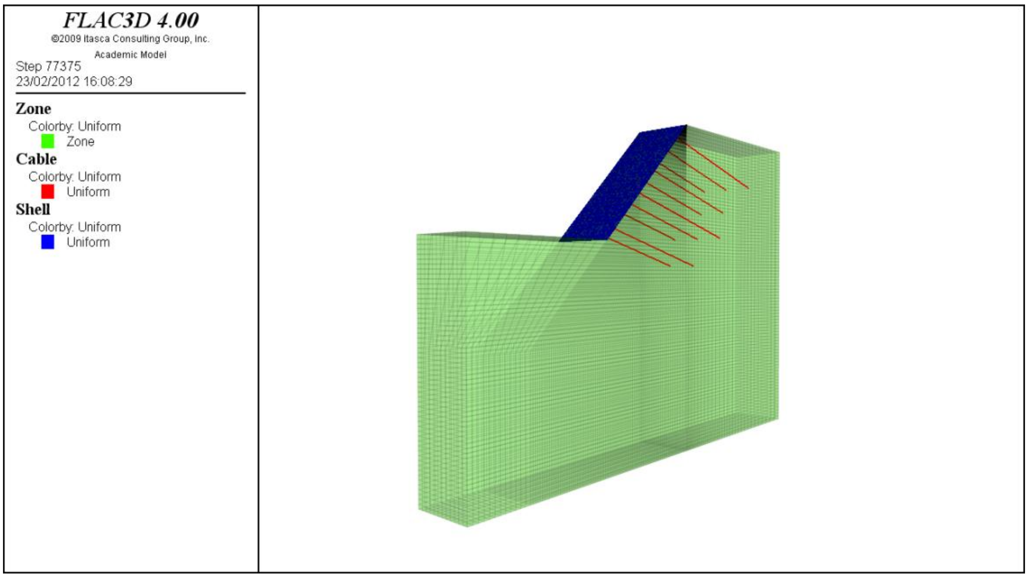


Fig. 4.7 – Fourth “cut”

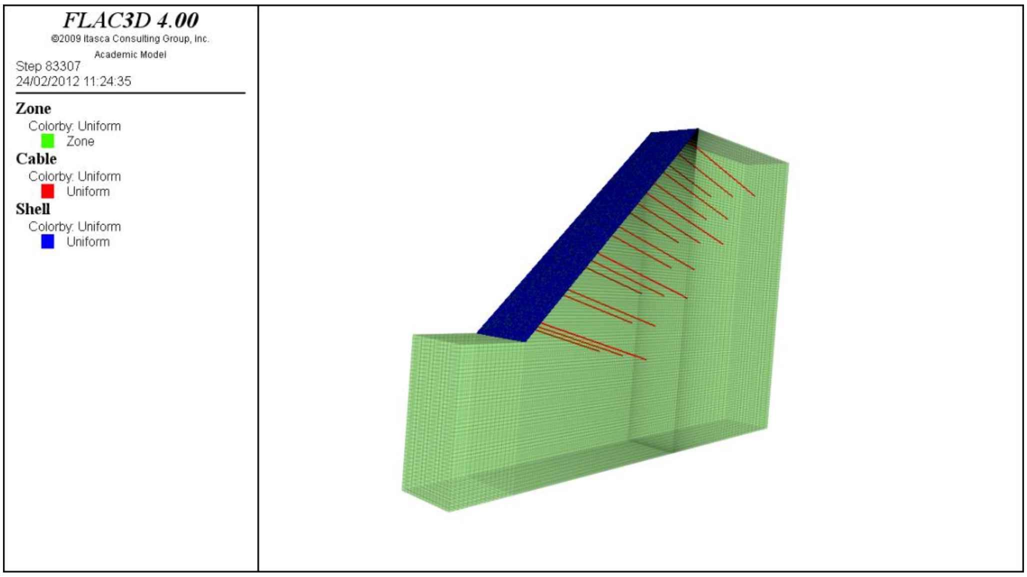


Fig. 4.8 – Entire structure

For the first three scripts a higher level of refinement was used: every zone corresponds to a 20 cm wide block. The other script, instead, were developed with a level of refinement that correspond to a 25 cm wide blocks.

Once the model has been run, it is possible to see the different forces and stress acting in every different elements.

4.2.1 Parametric analysis of the model

To understand whether the model used to simulate the behavior of a soil nailed slope was appropriate, results were initially compared to a real slope for which monitoring data was available. Due to the lack of available data regarding structures with flexible facing a site with a hard facing system was analysed.

This structure is situated in the city of Istanbul, and is a 10 meter high structure (fig. 4.9) , with a facing built with a *shotcrete* technique and with an inclination of 85°. Inclinometers were installed to register the displacements occurred in the structure.

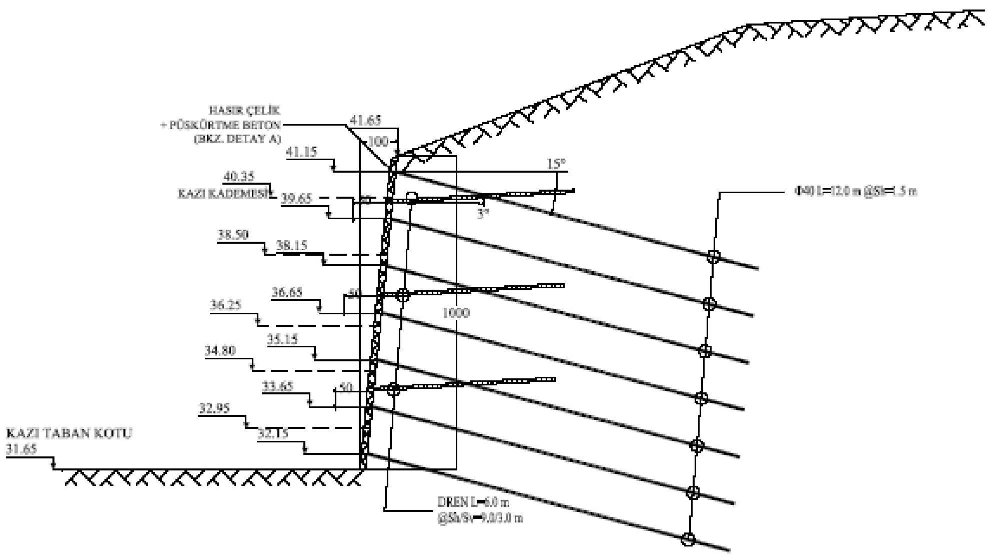


Fig. 4.9 Cross section of the structure

Data collected by the inclinometer are shown in figure 4.10 . That shows a maximum value of the displacements at the top of the wall of about 24 mm, and a value of about few millimeters at the bottom.

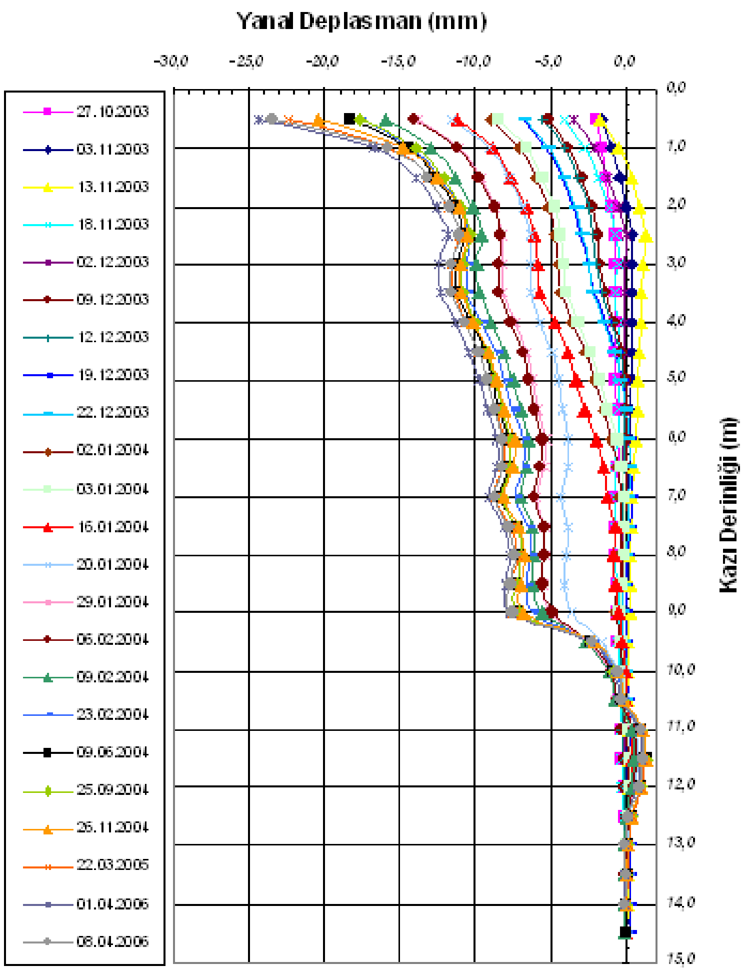


Fig. 4.10 Inclinometer readings (Durgunoglu *et al.*, 2007)

The finite element back analysis of this structure was performed on the cross section in fig. 4.9 . Subsoil parameters are tabulated in table 4.4.

The slope geometry designed with the software and the displacements calculated with it are presented in figure 4.11 and figure 4.12 , respectively.

The structure was built using the soil nailing technique and the characteristics of its composing elements are listed below:

Soil	
Young's modulus	30 [MPa]
Friction angle	33°
Cohesion	0.005 [MPa]
Density	1840 [kg/m^3]
Nails	
Length	12 [m]
Diameter	Φ40 [mm]
Young's modulus	200 000 [MPa]
Yield force	2500 [MPa]
Grout	
Diameter	Φ110 [mm]
Young's modulus	35000 [MPa]
Cohesion	7.5 [MPa]
Friction angle	35°

Shotcrete	
Young's modulus	30000 [MPa]
Poisson's ratio	0.2

Tab. 4.4 Properties of elements (Durgonoglu *et al.*, 2007)

The cross section made with the software is shown in the following figure (4.11)

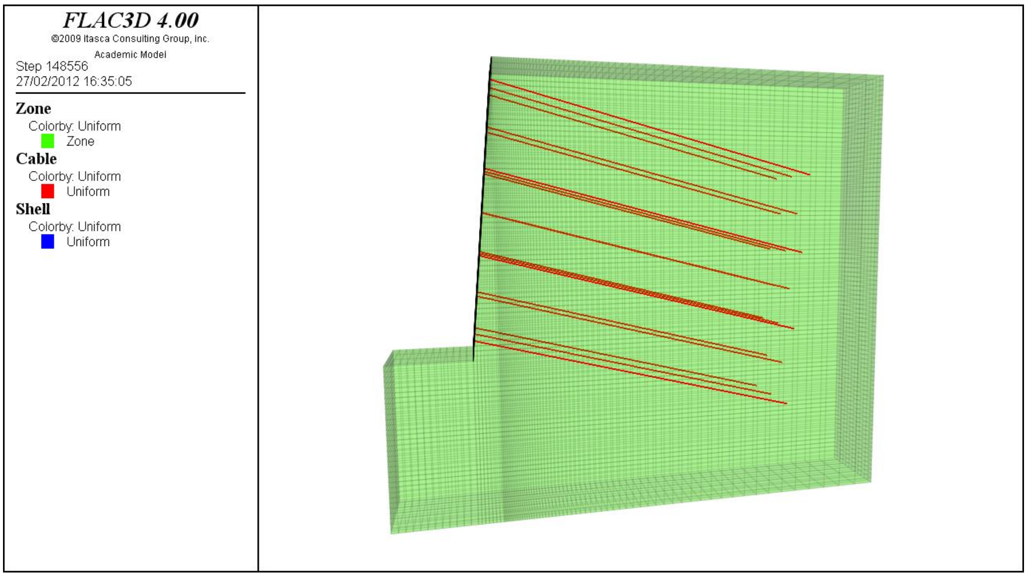


Fig. 4.11 Cross section obtained with *FLAC^{3D}*

The simulation was conducted to compare the deformation with the data collected by the inclinometer in the real structure. The result is shown in the following figure.

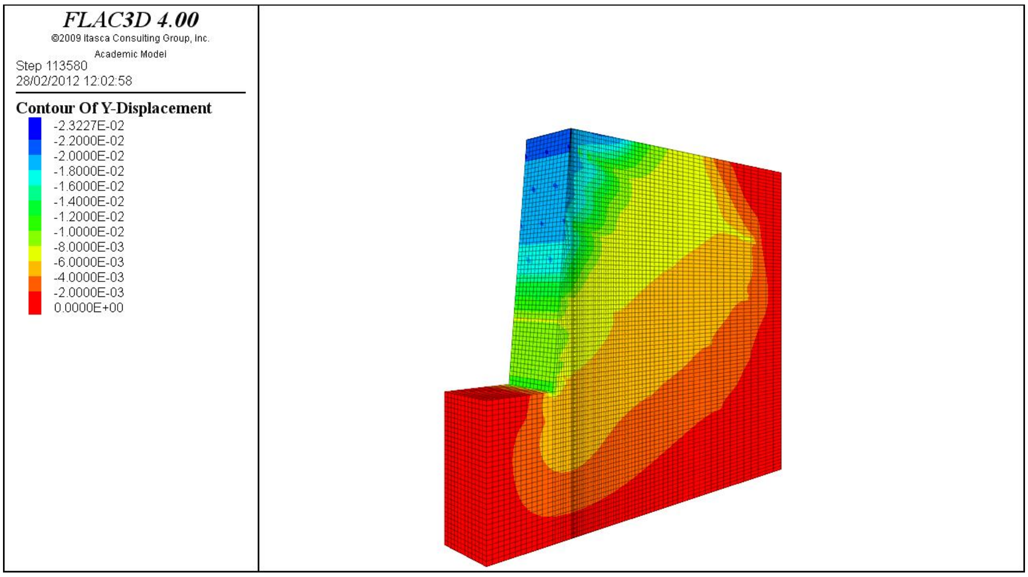


Fig 4.12 Deformations result in the model

As it possible to see the maximum displacement that occurs in the model has a value of 23 mm that is comparable with the value registered that has a value of 24 mm. Displacements are decreasing with the depth until a value of about few millimeters.

With this data it is even possible to confirm an appropriate implementation of the model and the effective potential of the software.

4.3 Study of the stress acting in the nails with different inclination of the slope

The first three structure were developed with a higher level of refinement respect the other models developed and they were compared to the literature data, to understand the behavior of this kind of structure and to understand if they could show analogies and differences with the reality . These three scripts were developed with a value of the slope angle of, respectively, 45°, 60° and 75°. The spacing considered amounts to a value of 1.5 m. This choice was made with the typical design of this kind of structures.

4.3.1 Stress acting in the nails in the first model – 45°

The first script consists in a slope 10.5 m high. It is a slope with an angle with a value of 45° to the horizontal . The horizontal and the vertical spacing between two row of nails have a value of 1.5 m (fig. 4.13).

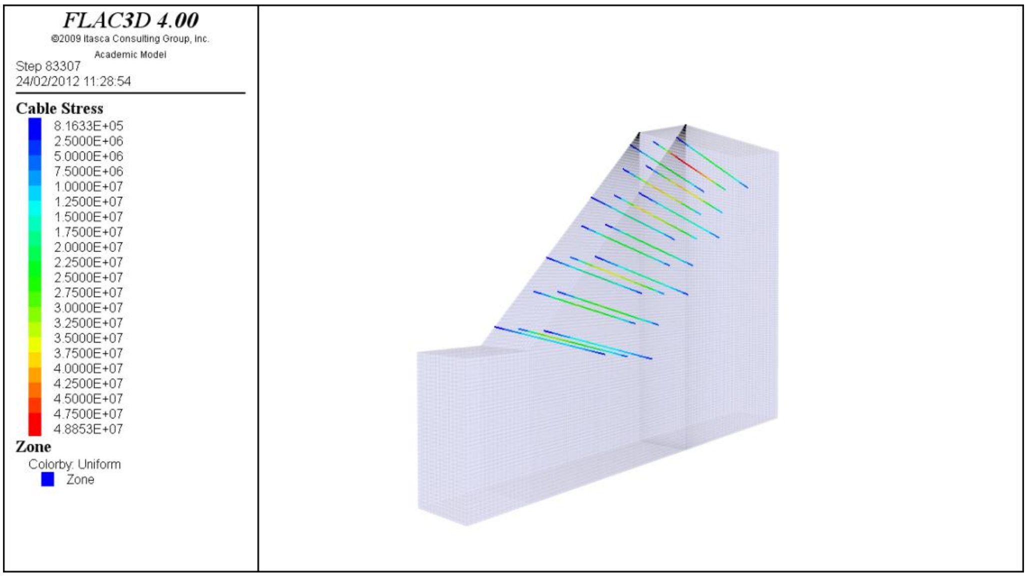


Fig. 4.13 – Cable stress in the first structure

The development of the stress in the different rows was studied. The first row is considered the highest one at a height of 9.75 m and the seventh row is considered the lowest, at a height of 0.75 m. The results are shown from fig.4.14 to 4.20. This data were taken at the last step of construction, when the model were completely built.

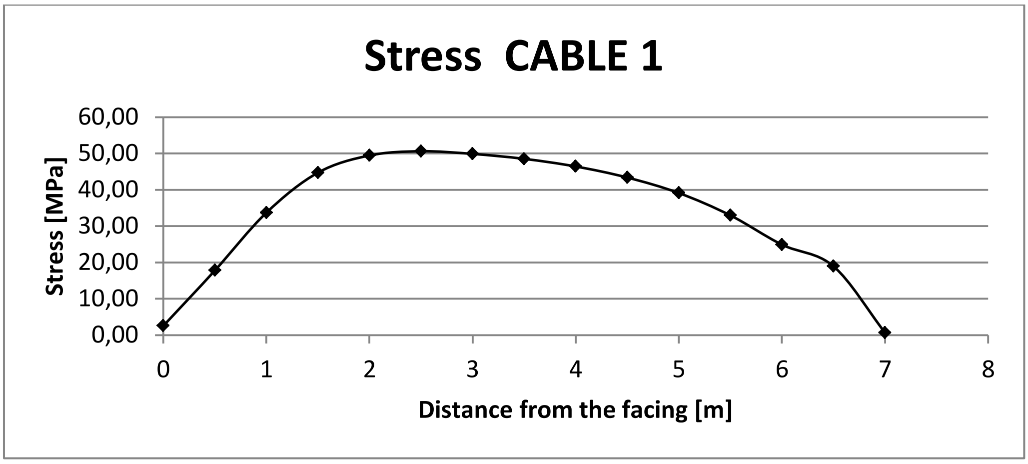


Fig. 4.14 – Cable stress in the first row

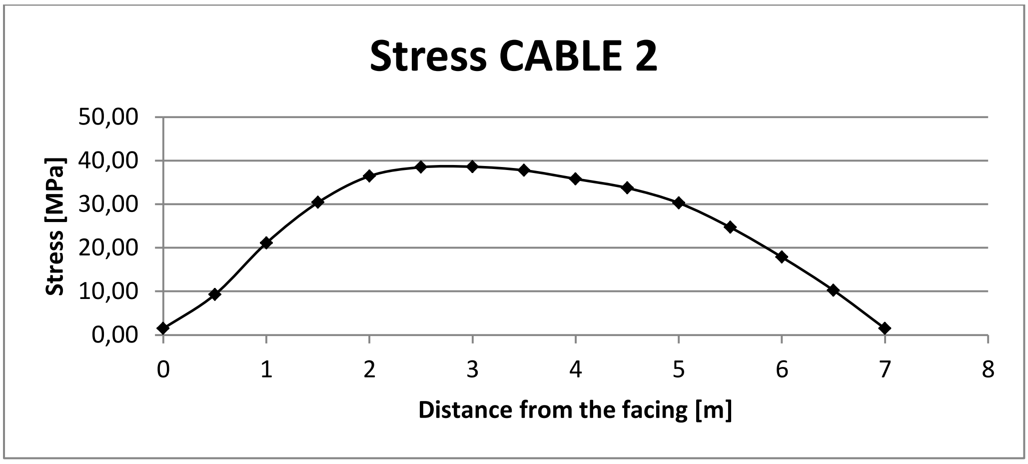


Fig. 4.15 – Cable stress in the second row

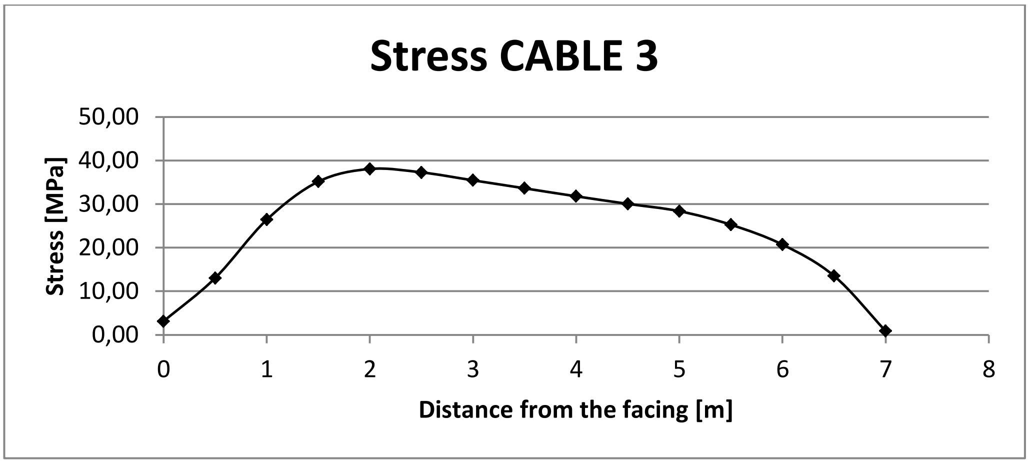


Fig. 4.16 – Cable stress in the third row

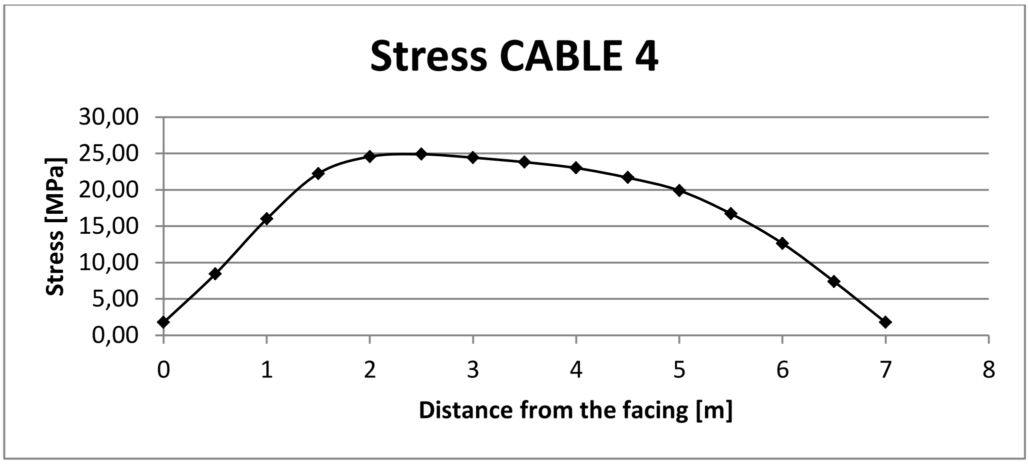


Fig. 4.17 – Cable stress in the fourth row

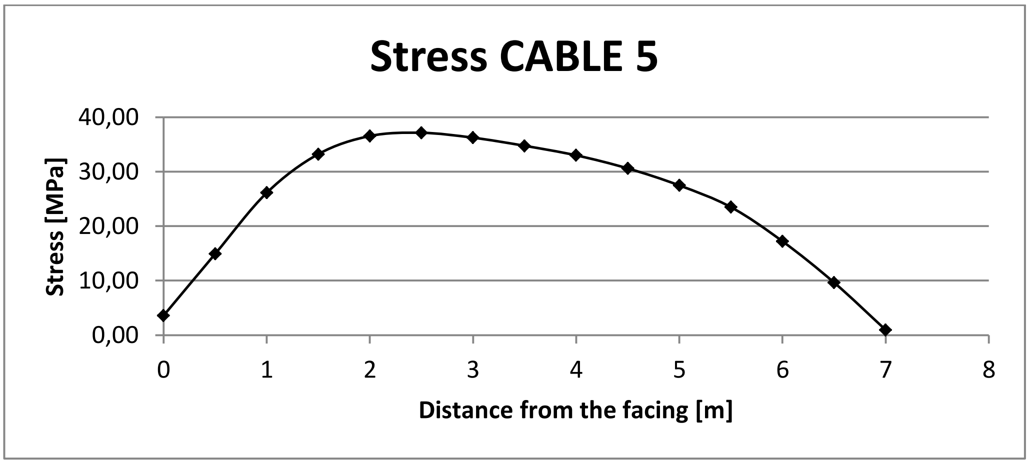


Fig. 4. 18– Cable stress in the fifth row

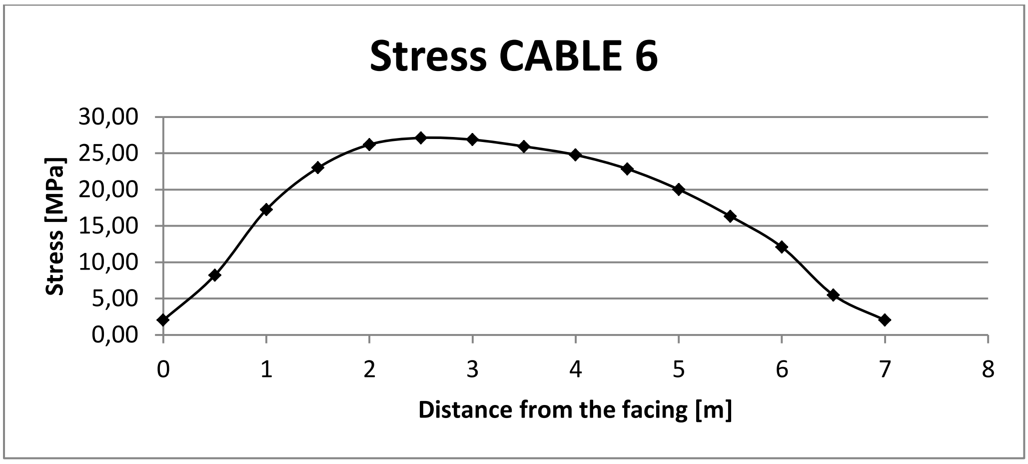


Fig. 4.19 – Cable stress in the sixth row

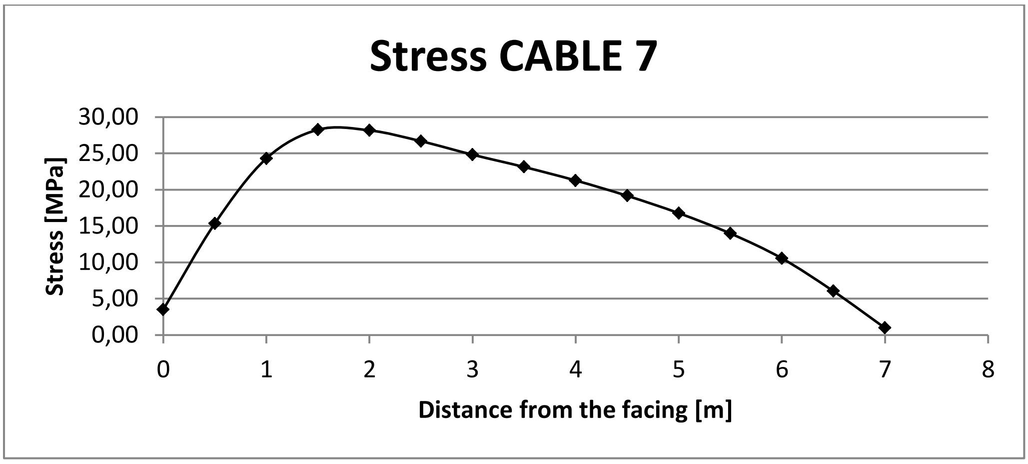


Fig. 4.20 – Cable stress in the seventh row

These data confirm the highest row is the more loaded and the stress in the cable is decreasing with the depth. That is due to the fact that the first row is, of course, the first installed row and step by step the stress is increasing because the amount of soil that needs to be sustain is clearly higher on every step.

There is an exception that is possible to see in the fourth row: the stress developed in this row is not higher than the fifth one. That is probably due to the fact that the slope has achieved an equilibrium and there is not developing plasticity in that cut, hence the nails are not working at their maximum bearing capacity.

In fig. 4.21 is possible to see how the stress is increasing step by step in the most loaded row. That confirm that a complete equilibrium in the slope has been reached, because the stress has increased in every point at every step.

The low level of plasticity is confirm in fig 4.22 where it is not possible to see a high level of maximum shear strain increment. That means the structure has achieved an equilibrium and there are not considerable soil movement.

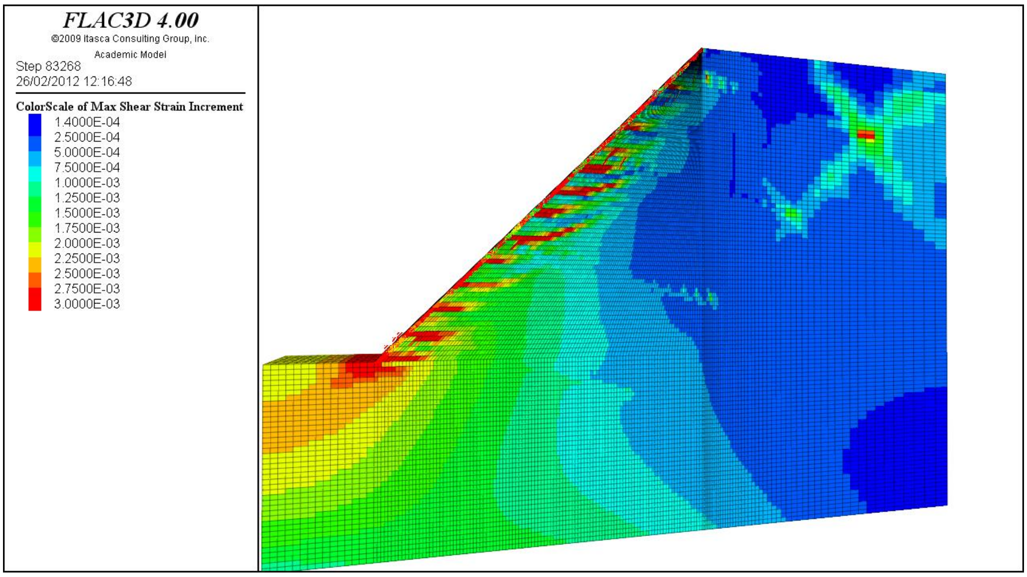


Fig. 4.22 – Potential slip surface of the slope

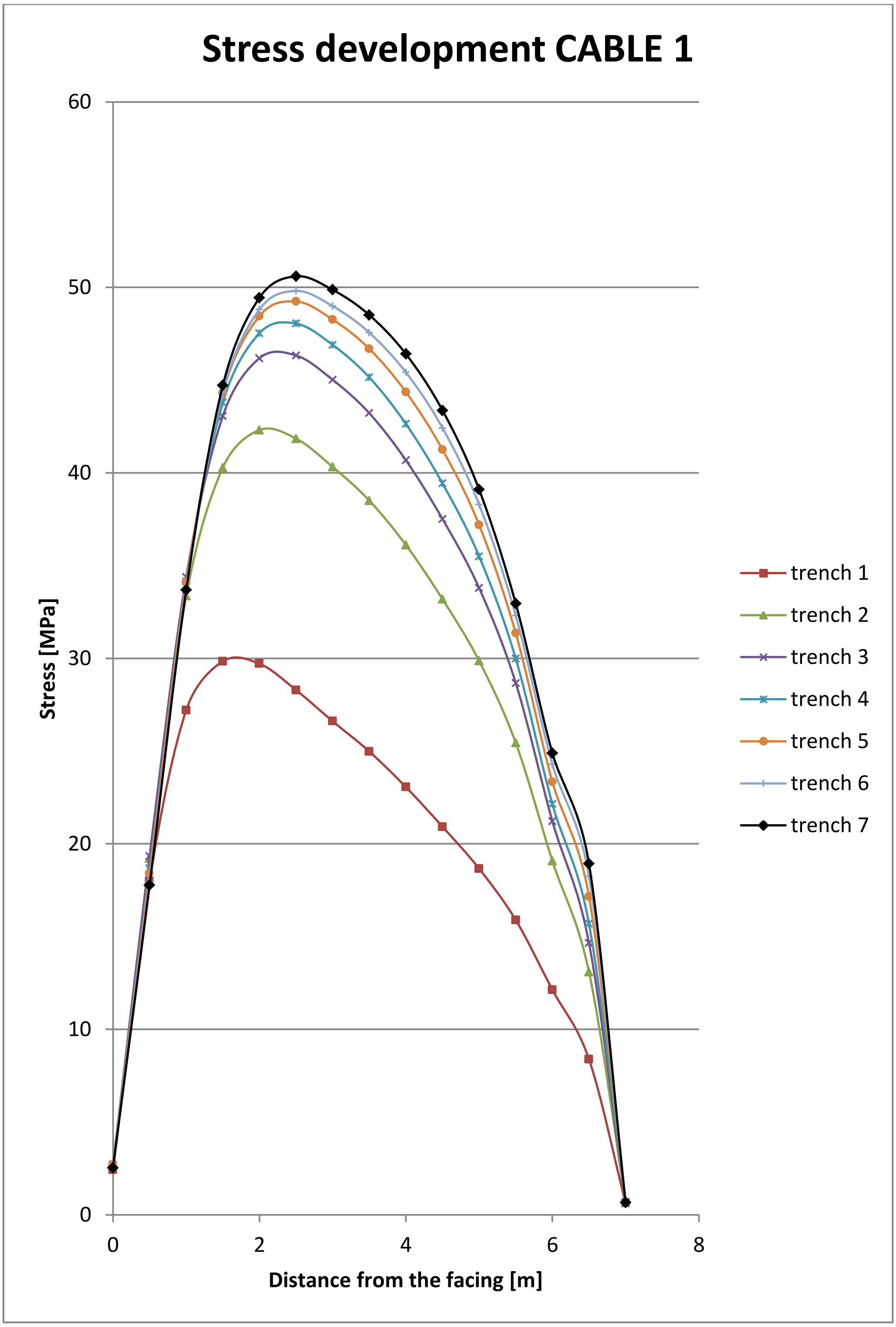


Fig. 4.21 – Cable stress developed step by step in the first row

4.3.2 Stress acting in the nails in the second model – 60°

The second script consists in a slope 10.5 m high. It is a slope with an angle with a value of 60° to the horizontal . The horizontal and the vertical spacing between two row of nails have a value of 1.5 m (fig 4.23).

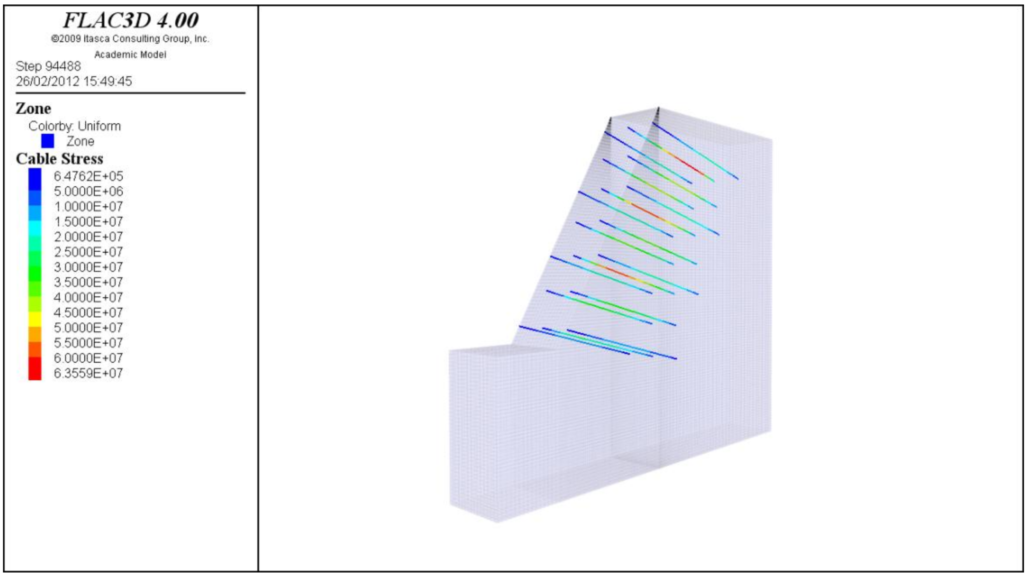


Fig. 4.23 – Cable stress in the first structure

The development of the stress in the different rows was studied and it is shown from fig. 4.24 to fig. 4.25

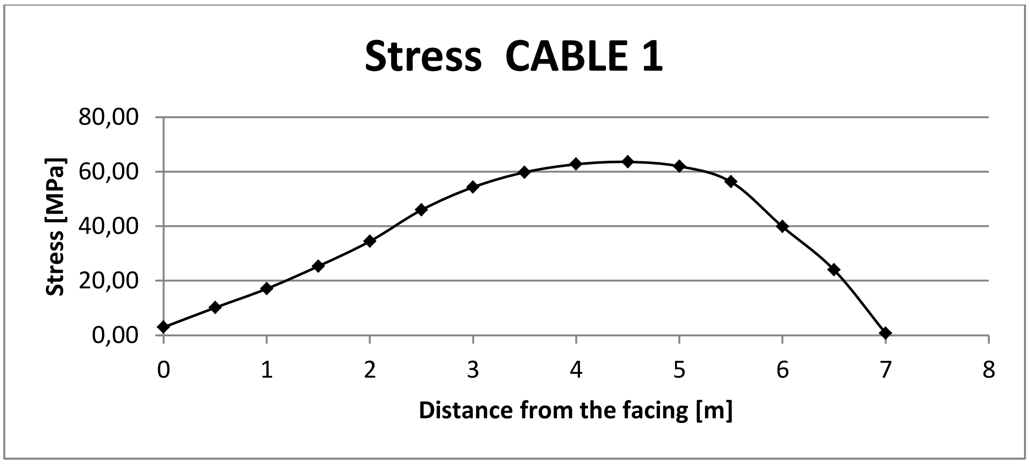


Fig. 4.24 – Cable stress in the first row

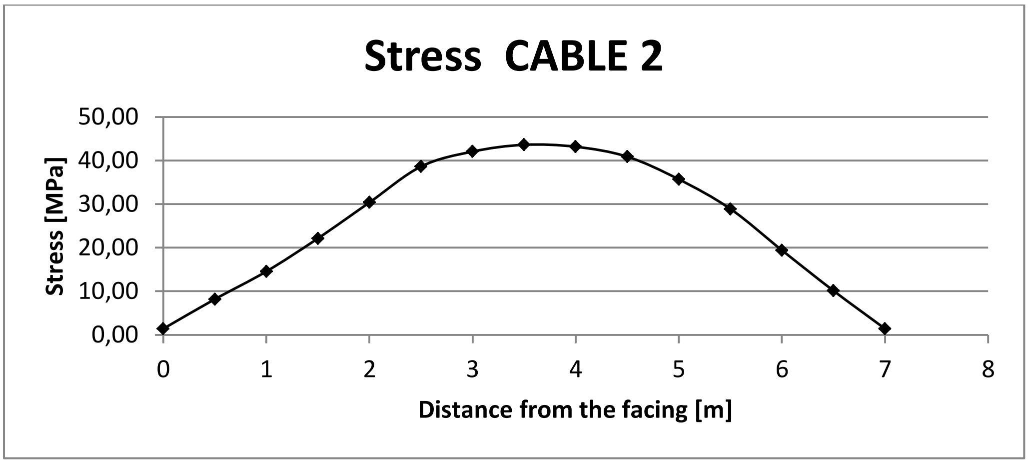


Fig. 4.25 – Cable stress in the second row

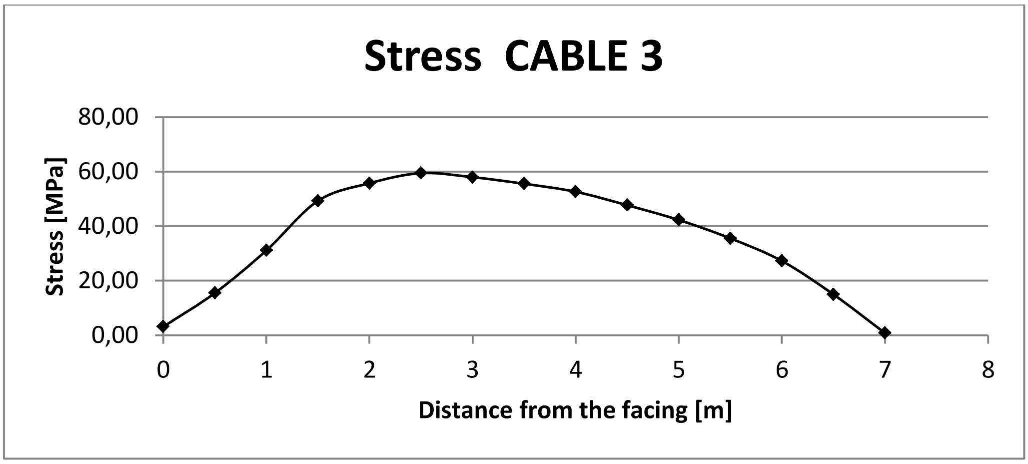


Fig. 4.26 – Cable stress in the third row

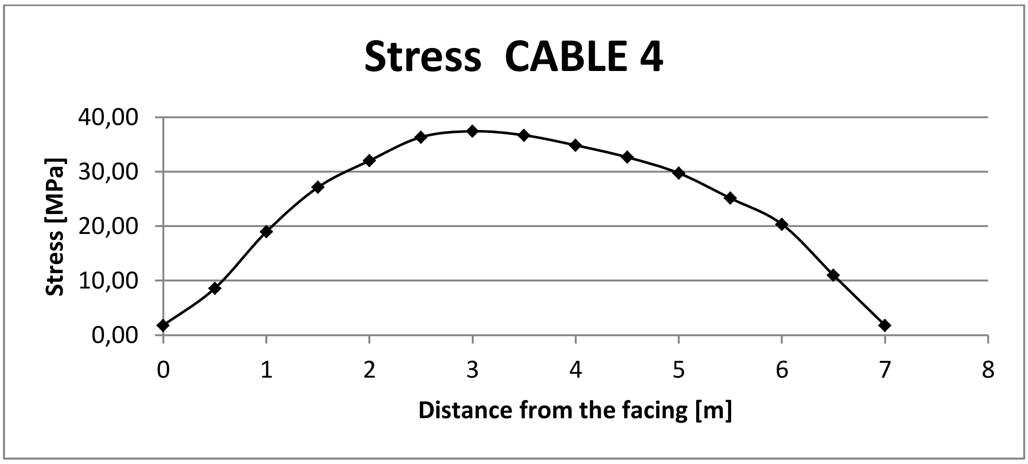


Fig. 4.27 – Cable stress in the fourth row

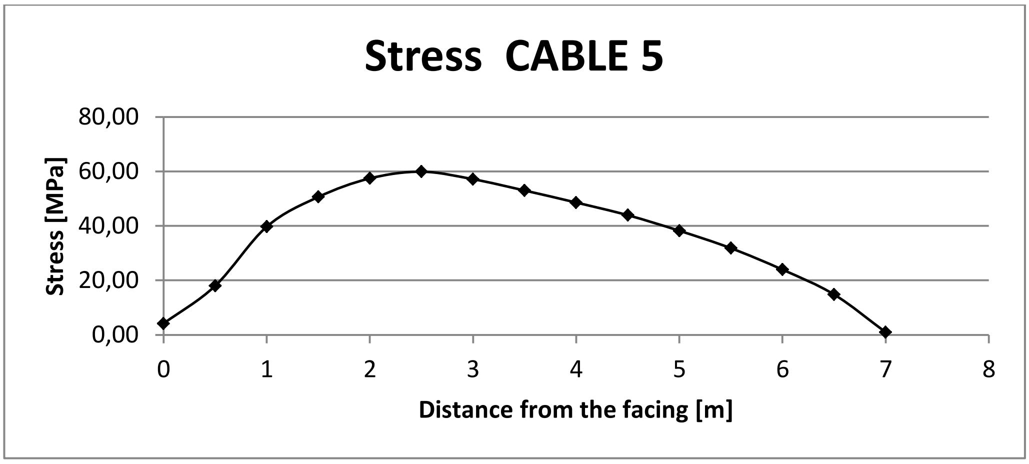


Fig. 4.28 – Cable stress in the fifth row

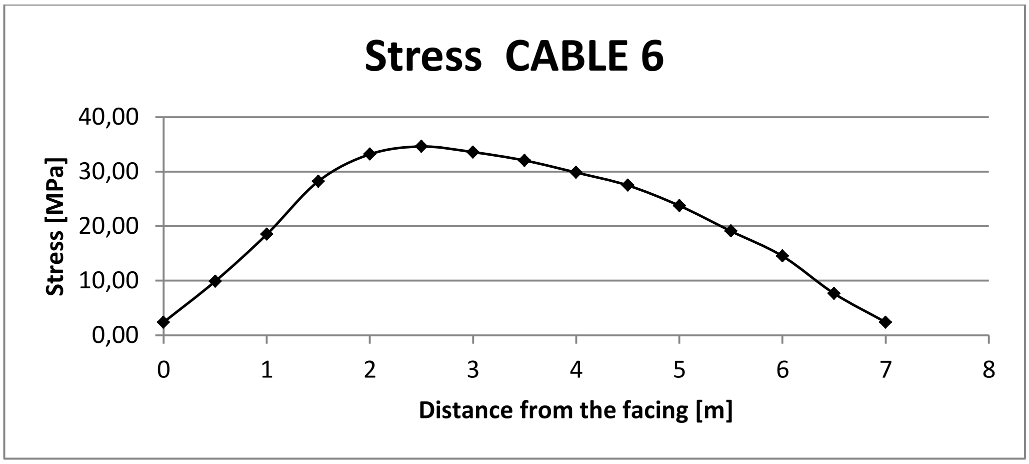


Fig. 4.29 – Cable stress in the sixth row

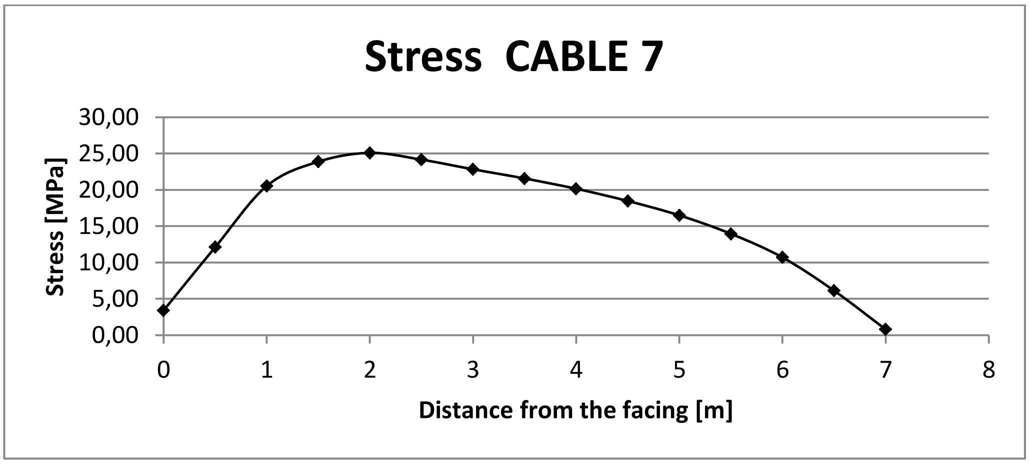


Fig. 4.30 – Cable stress in the seventh row

Also these data confirm the highest row is the more loaded and the stress in the cable is decreasing with the depth. It is clear to see the stress developing in this case is greater than shallower slopes.

In this case there are two exceptions that is possible to see in the second and in the fourth row: the stress developed in these rows are not higher than, respectively, the third and the fifth row.

In fig. 4.32 is possible to see how the stress is increasing step by step in the most loaded row. In this case the stress is not increasing in every point at every step. From the fourth trench the angle with which the nail is achieving the maximum stress is getting lower. That means that from this step the plasticity is developing in the slope and a potential failure plain is occurring (fig. 4.31)

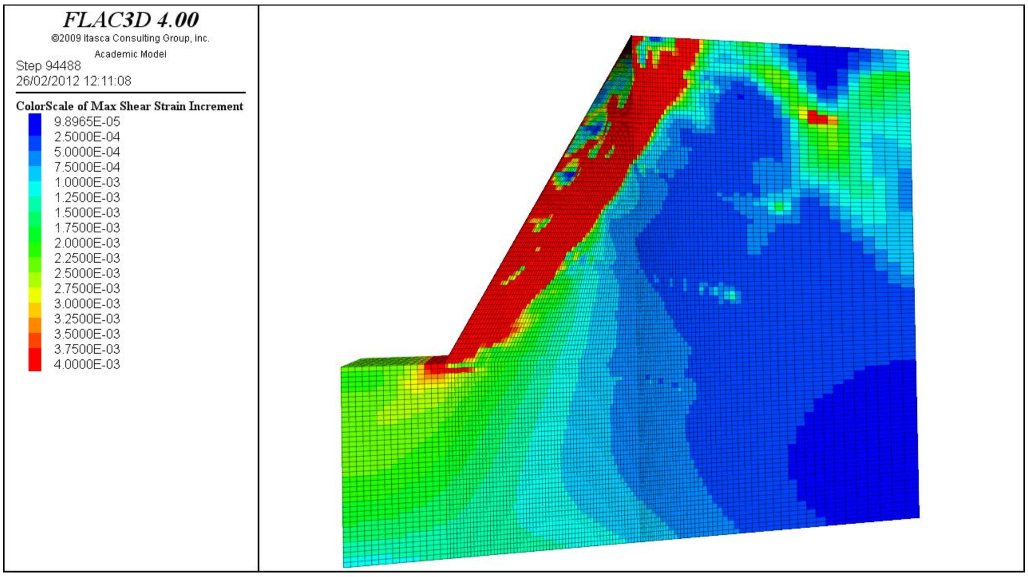


Fig. 4.31 – Potential slip surface of the slope.

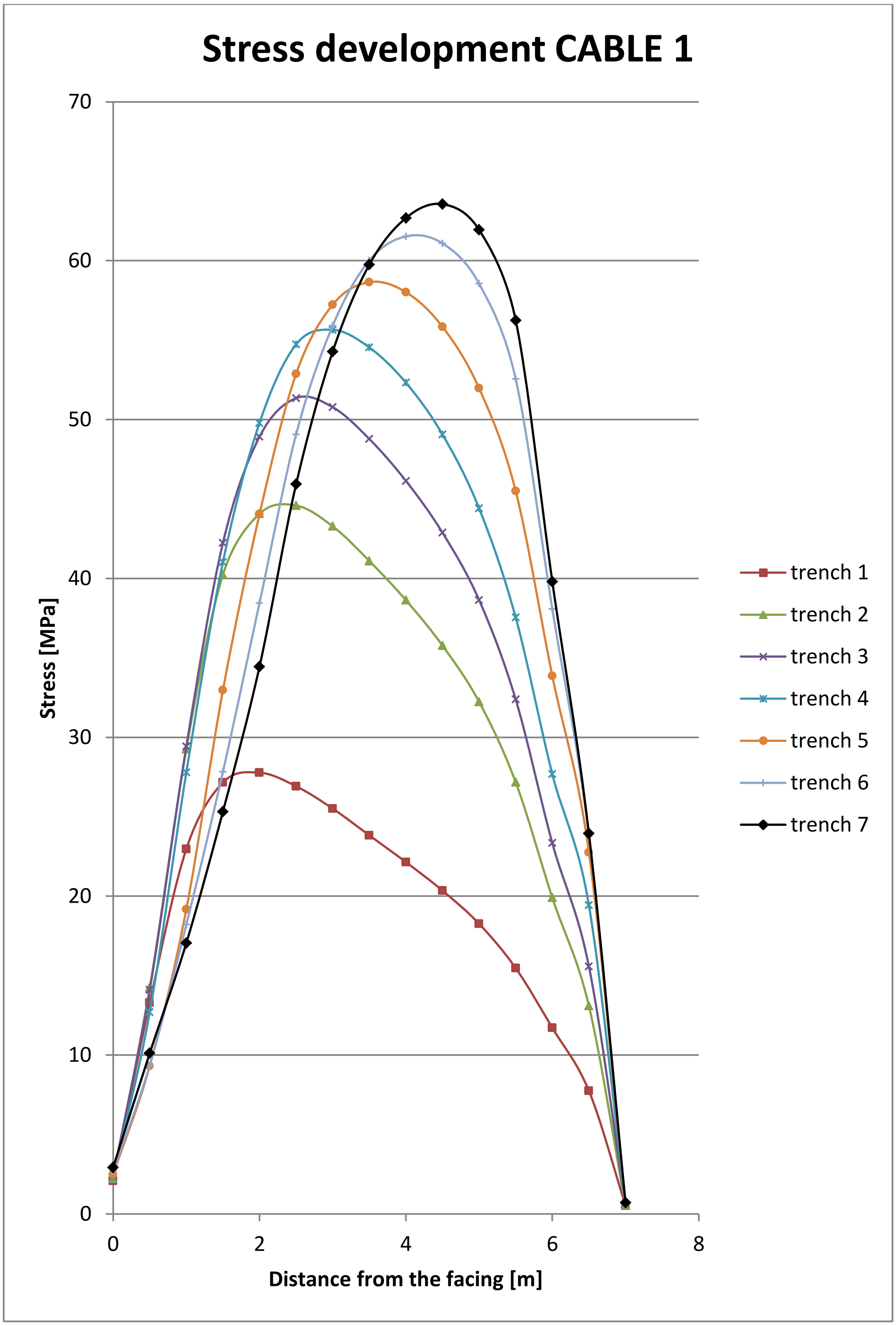


Fig. 4.32 – Cable stress developed step by step in the first row

4.3.3 Stress acting in the nails in the third model – 75°

The third script consists in a slope 10.5 m high. It is a slope with an angle with a value of 75° to the horizontal . The horizontal and the vertical spacing between two row of nails have a value of 1.5 m.

In this case, it is not possible to collect comparable data. That is due to the fact that a failure has occurred in the model. In the following figures is possible to understand where it has occur. Studying the stress increasing in the first row of cables is clearly show that since the fourth cut has been made the system did not resist to the soil movement and a decrease of the stress in the nail occur.

In figure 4. is more clear the development of global failure plane rather than localised as in shallower slope. The potential critical slip surface is now well define and its dimension are notable.

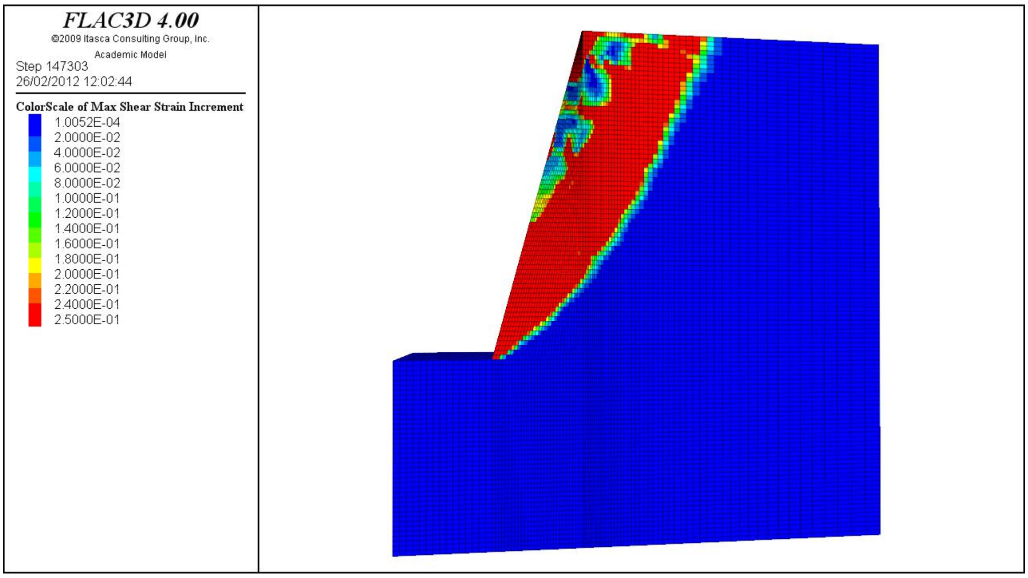


Fig. 4.33 – Potential critical slip surface

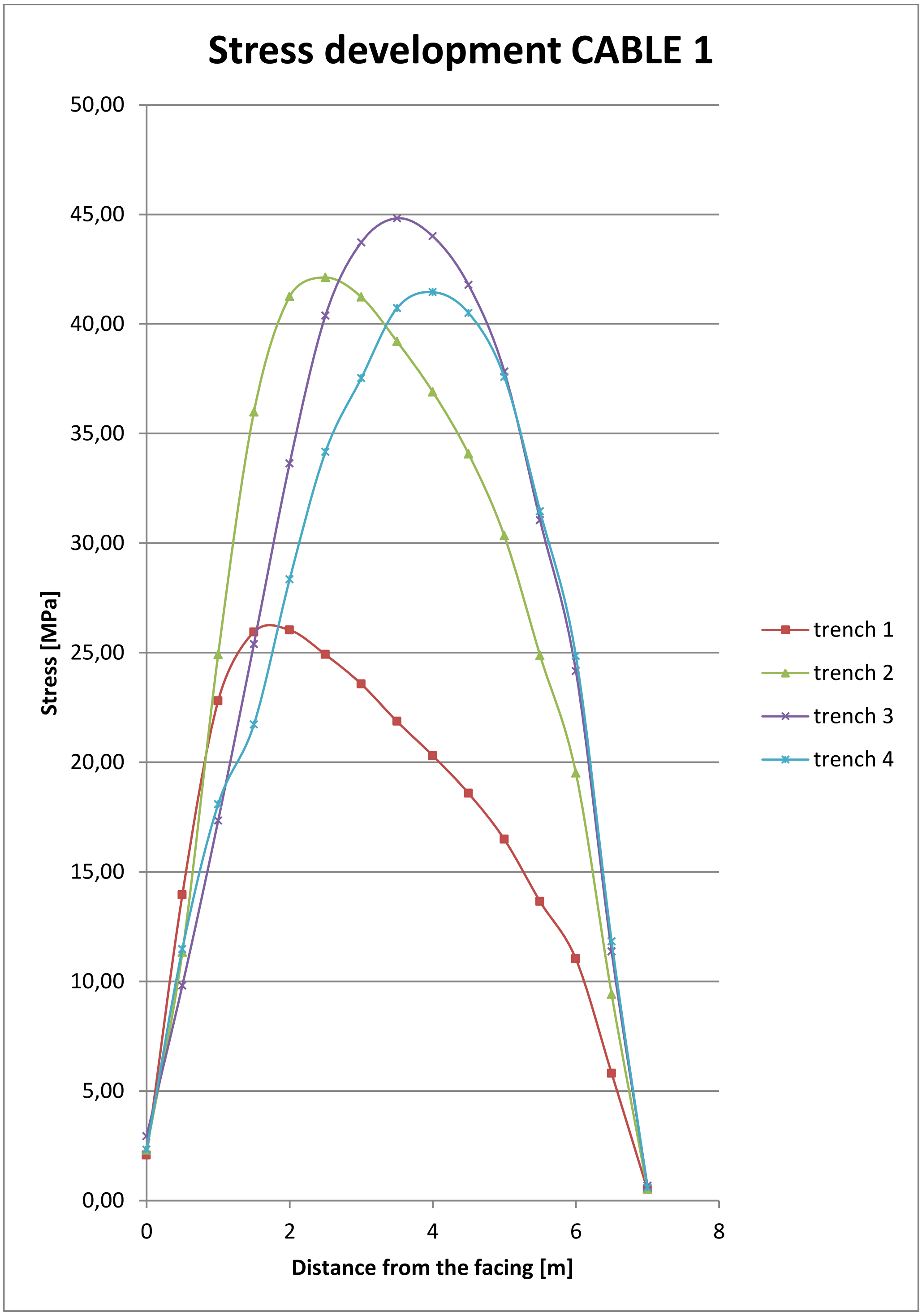


Fig. 4.34 – Cable stress developed step by step in the first row

4.4 Study of the displacement of the slope surface with different inclination of the slope

In this paragraph the displacement occurring in the different slope with different inclination has been studied. The same three models were used to collect and compare data.

4.4.1 Displacements in the first model – 45°

The displacements occurring in the first slope are shown in fig 4.35

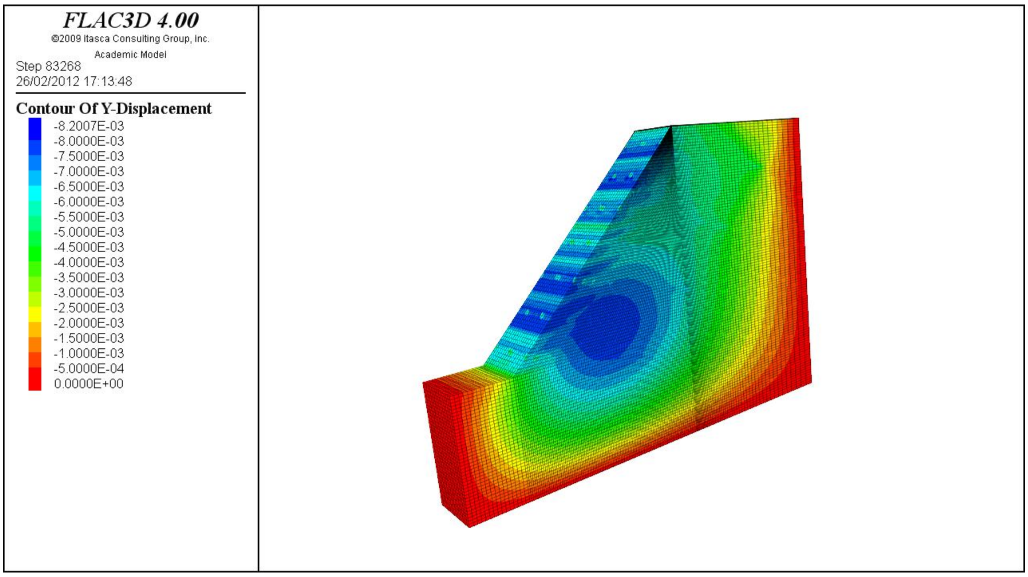


Fig. 4.35 – Displacements in the first model

The displacements were collected in correspondence of the two rows of nails, with *x*-coordinate from the global point system, respectively of 0.75 and 1.5 m, and a third row of points was collected between these two rows of nails with x-coordinate of 1.125 m.

The first vertical column has the nails at a height of: 2.25, 5.25 and 8.25 m. The second vertical column of nails presents the nail at a height of: 0.75, 3.75, 6.75 and 9.75 m.

The presence of the nails and its effect on displacements is more understandable watching the diagrams below from fig. 4.36 To 4.38.

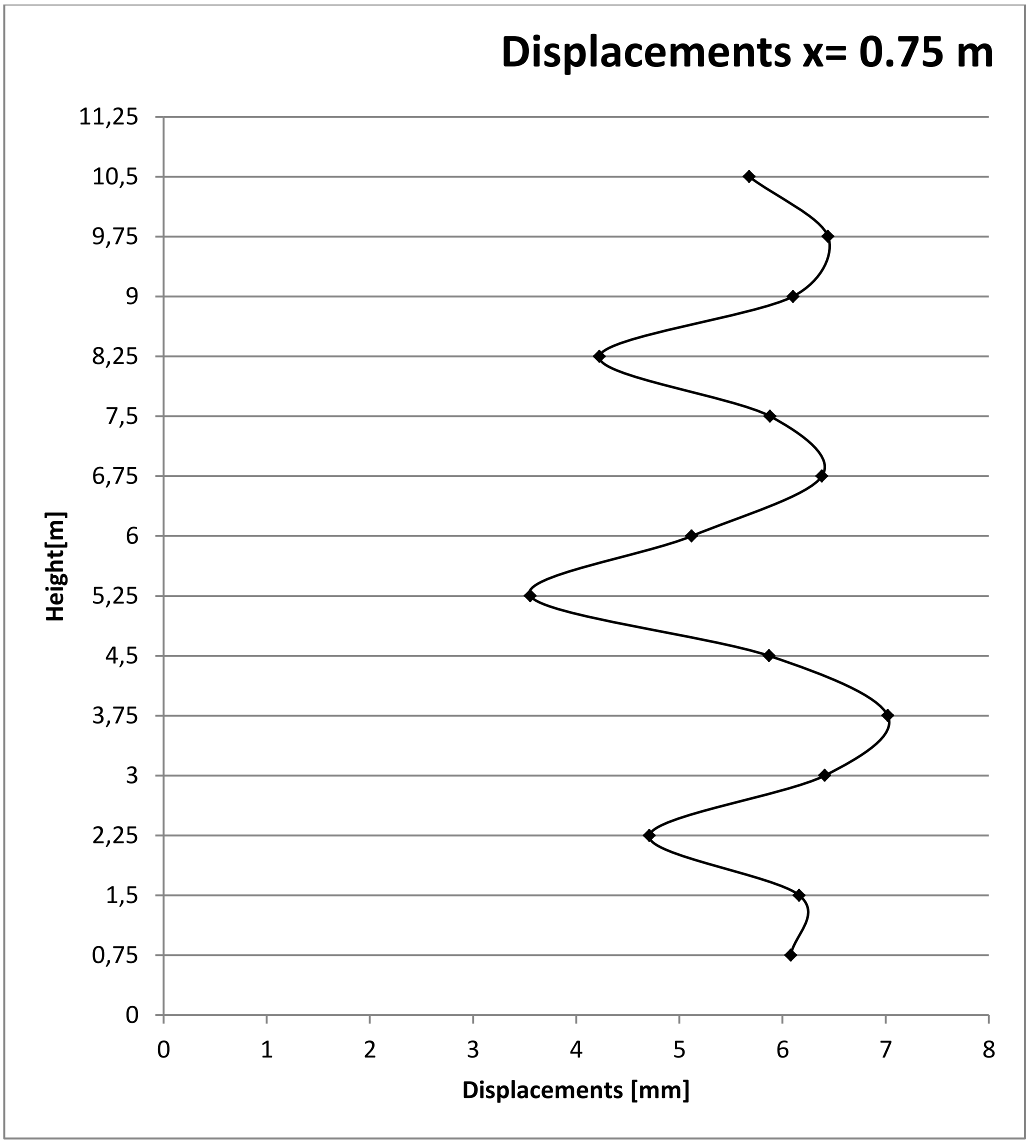


Fig. 4.36 – Displacements in the first row (0.75 m)

This diagrams show how the value of the displacements is very low. However is easy to differentiate the position of the nails that correspond to the lower displacement.

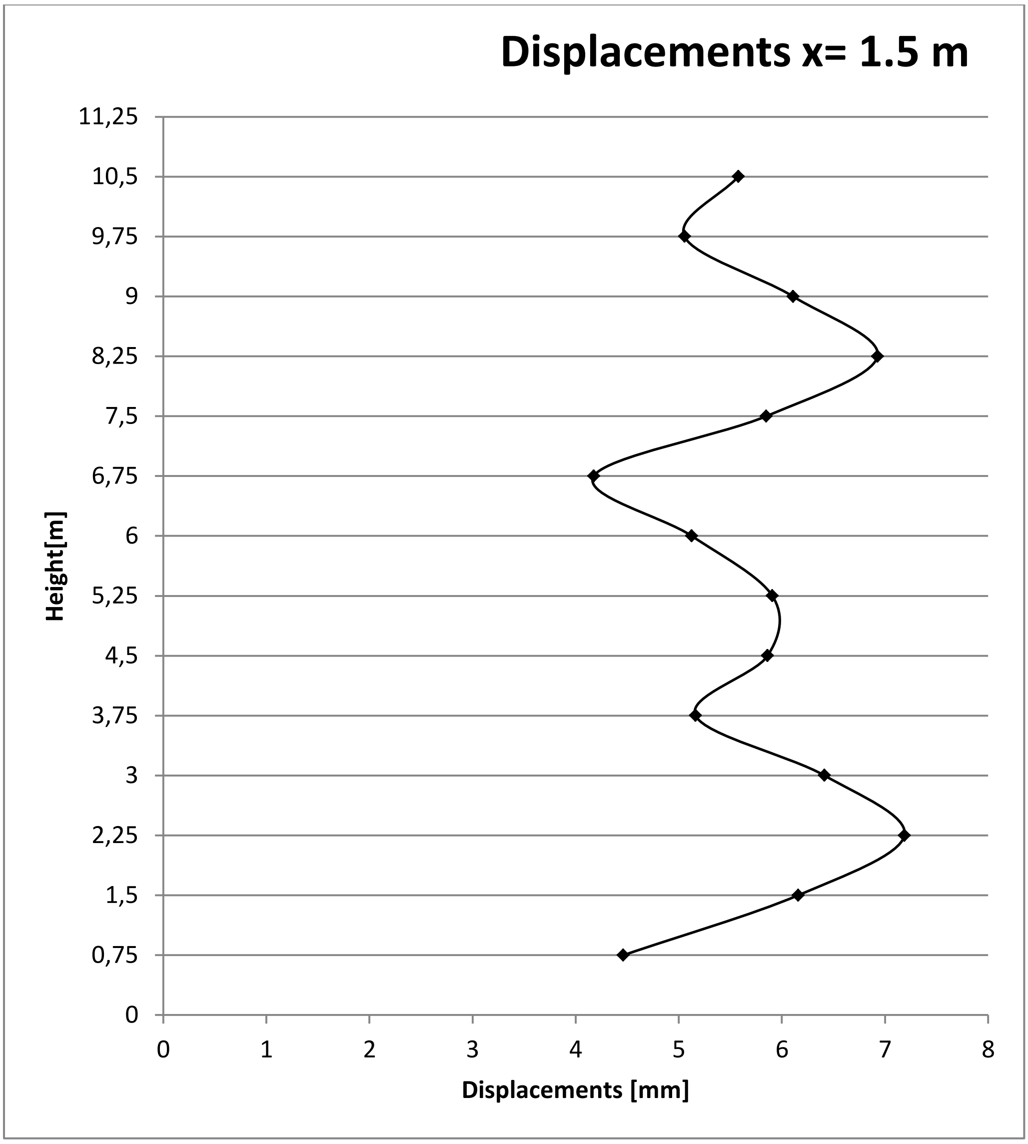


Fig. 4.37 – Displacements in the second row (1.5 m)

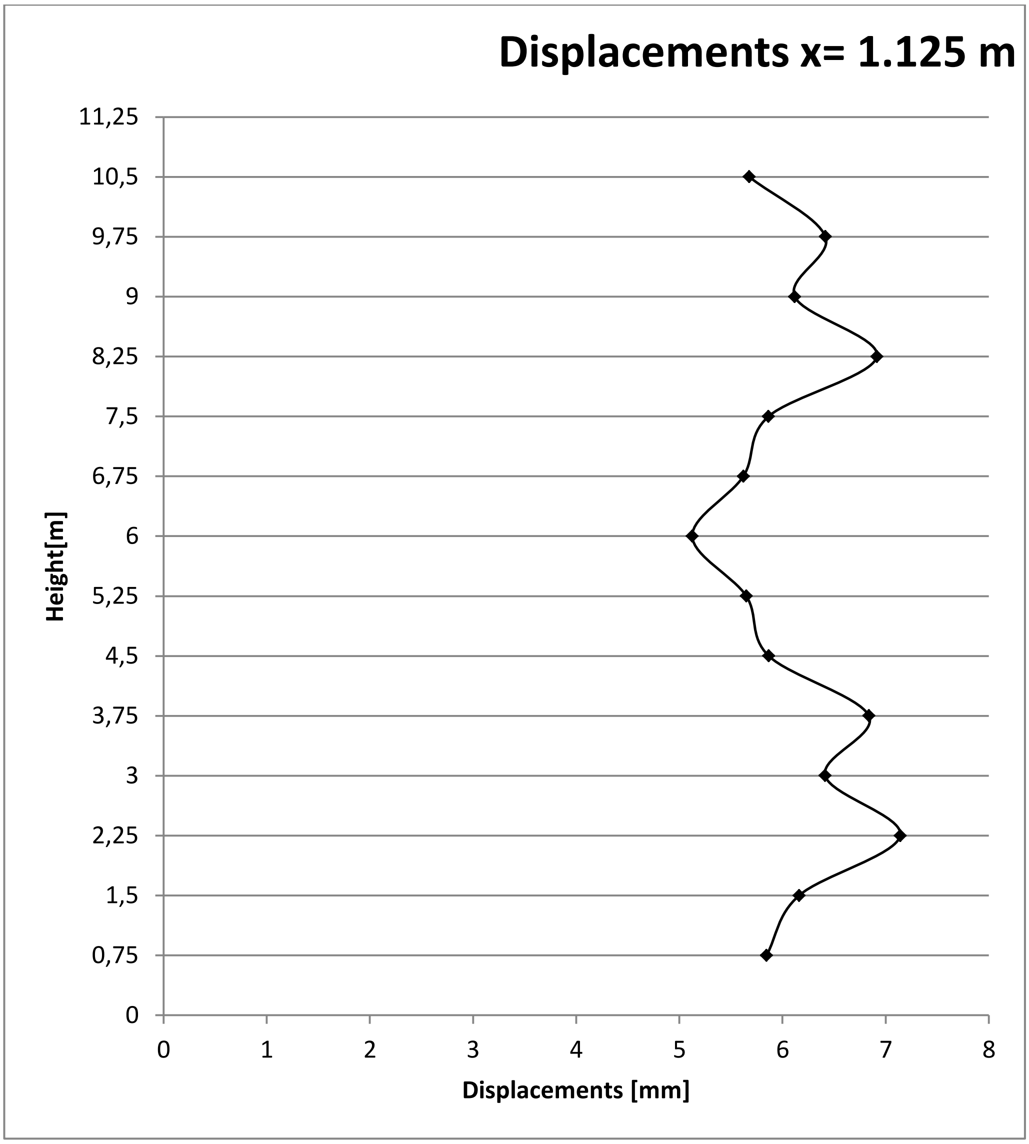


Fig. 4.38 – Displacements between the two rows of nails (1.5 m)

This diagram shows that, in this case, the biggest displacements occur at the top and at the bottom of the wall. In the middle, instead, they are lower hence they are not decreasing with height.

4.4.2 Displacements in the second model – 60°

The displacements occurring in the first slope are shown in fig. 4.39

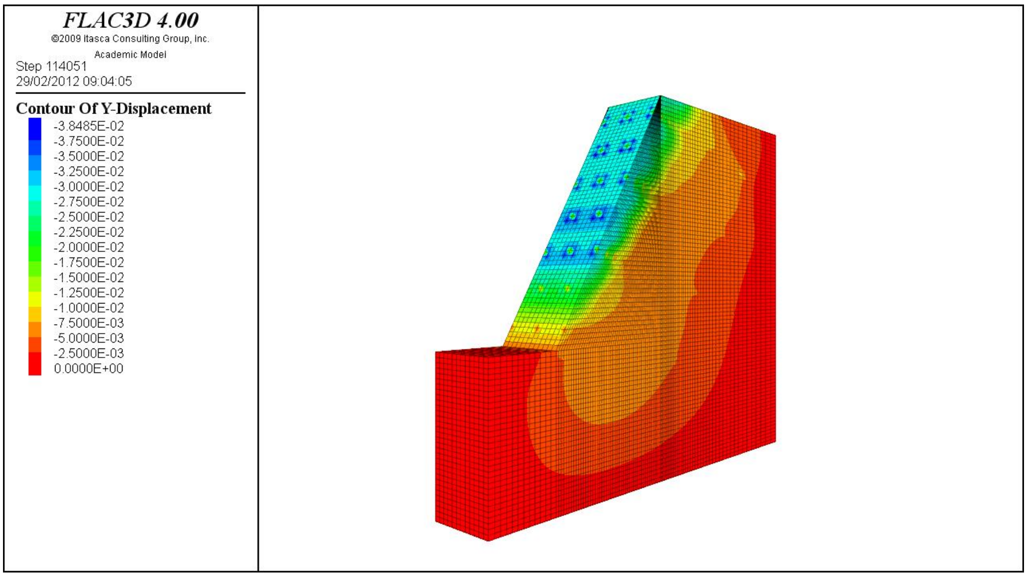


Fig. 4.39 – Displacements in the second slope

As for the first script the displacements were collected in correspondence of the two rows of nails, with *x*-coordinate from the global point system, respectively of 0.75 and 1.5 m, and a third row of points was collected between these two rows of nails with x-coordinate of 1.125 m.

Results are shown in the following figures.

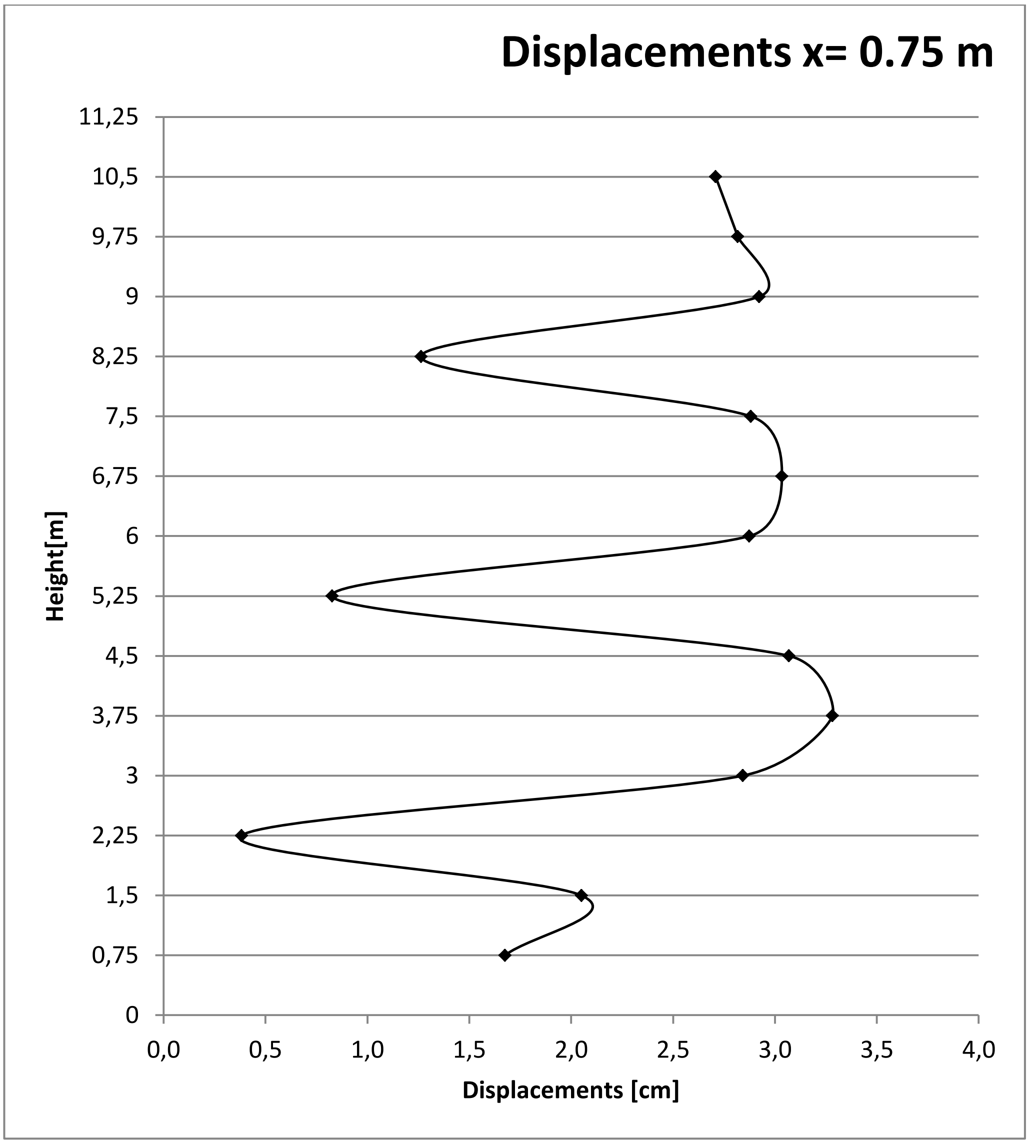


Fig. 4.40 – Displacements in the first row (0.75 m)

In this diagram is possible to see where the nails are installed: at a height of 2.25, 5.25 and 8.25 m. It shows that in this kind of structure with a flexible facing installed, the displacements is not increasing with the height, but it depends on how the slope is punching the face hence the wire mesh.

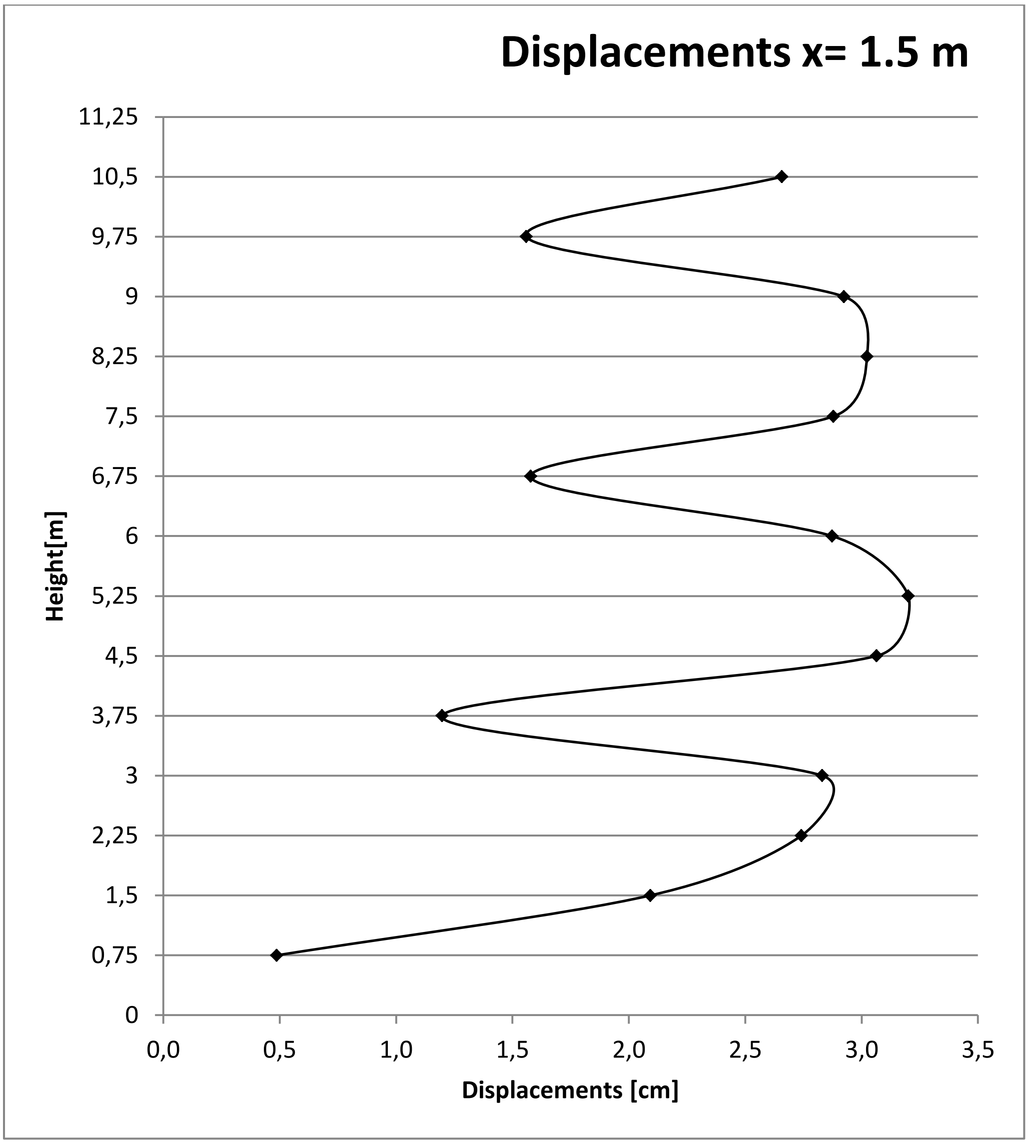


Fig. 4.41 – Displacements in the second row (0.75 m)

Even in this diagram it is clear to see where the nails are installed: at a height of 0.75, 3.75, 6.75 and 9.75 m. It is possible to see that the displacements are comparable in every spacing, that means the nails and the facing are working in every cut to maintain the equilibrium.

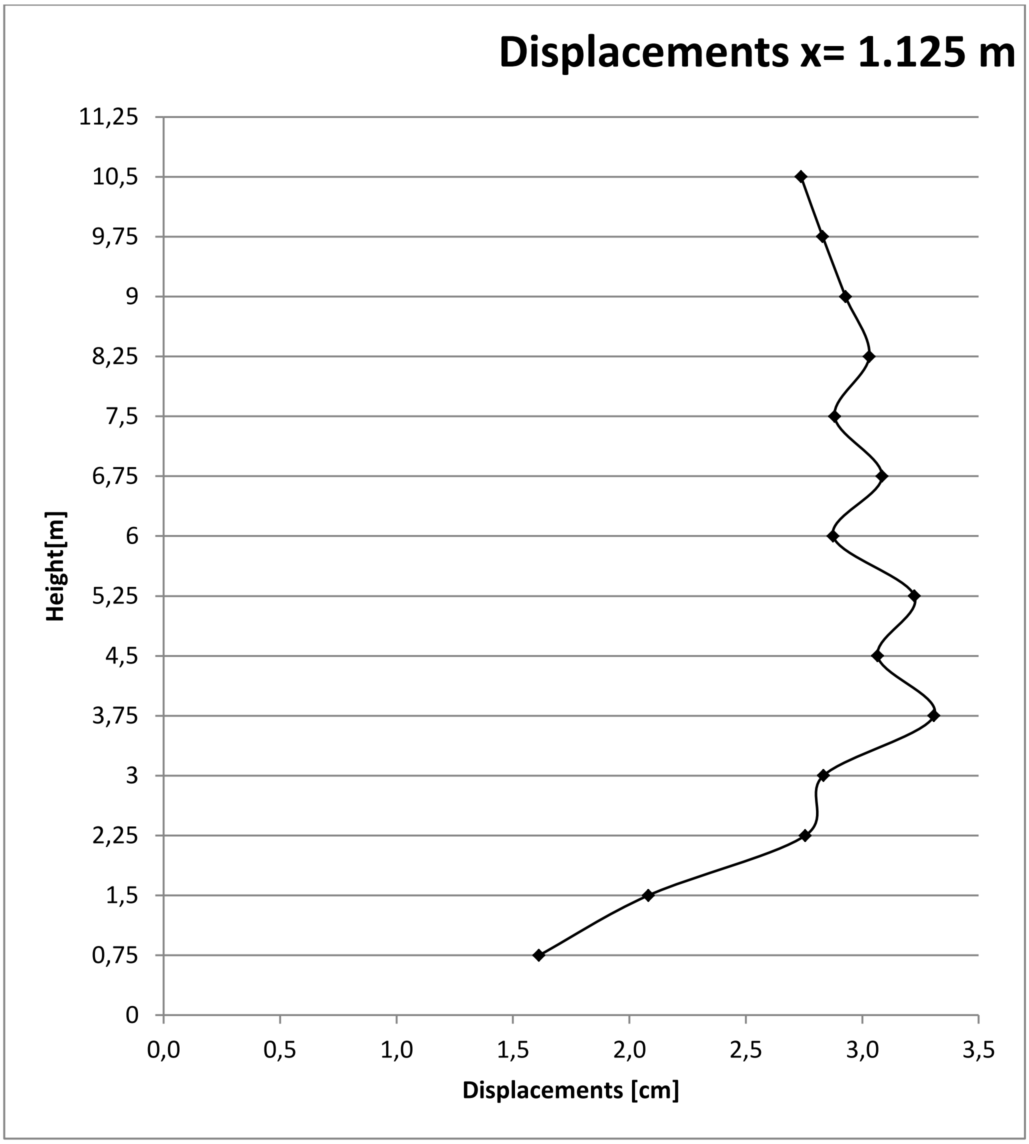


Fig. 4.42 – Displacements between the two rows of nails (1.5 m)

This diagram confirms that the displacements is not increasing with the height in soil nailed walls flexible facing but they depend on the force acting on the surface and on the facing's properties.

4.4.3 Displacements occurring with a spacing of 2.0 m

The data written in paragraph 4.4.1 show that displacements occurring with a spacing of 1.5 m are not that high. Therefore a comparison with a spacing with a value of 2.0 m was conducted to demonstrate that the use of a higher value of spacing is possible in structure with a low-medium value of the angle of inclination to the horizontal.

For this comparison, the script 7 was run and the output data of the displacements are shown in figure 4.43:

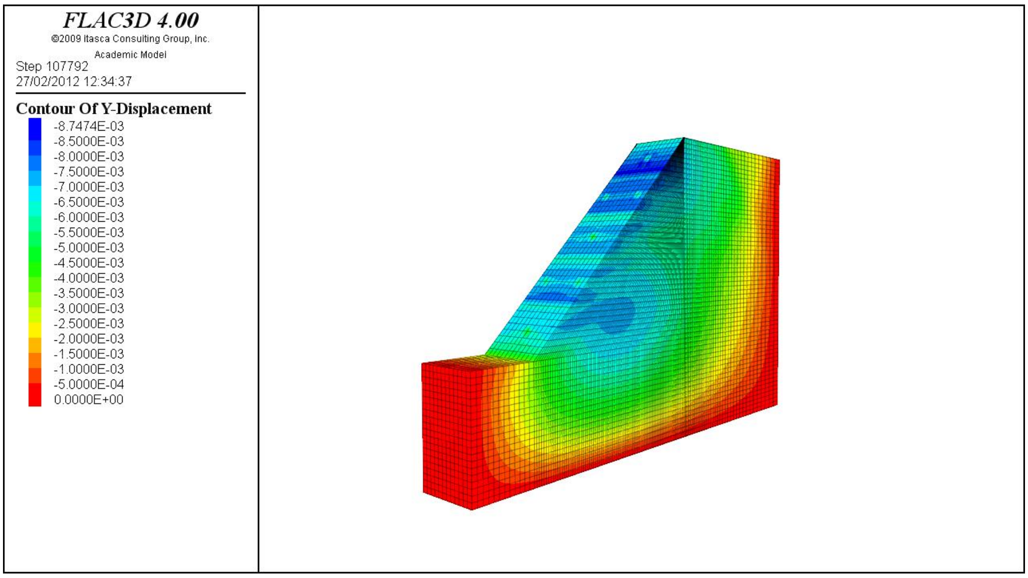


Fig. 4.43 Displacements occurring with a spacing of the nails of 2.0 m (45°)

As for the previous scripts, data were collected at the heights that correspond to the places where the nails were positioned and in the middle of two different rows of nails, both horizontally and vertically.

The results show that an increment of the spacing brings the stricter to higher displacements that are, however, contained in a little range and always under the value of 1 cm. This script demonstrates that in a sand-granular slope with a low angle of inclinations the use of higher space is possible. That would decrease significantly the cost of the whole structure itself.

The same implementation was run with an inclination of the slope of 60°. Data result are shown in fig.4.44:

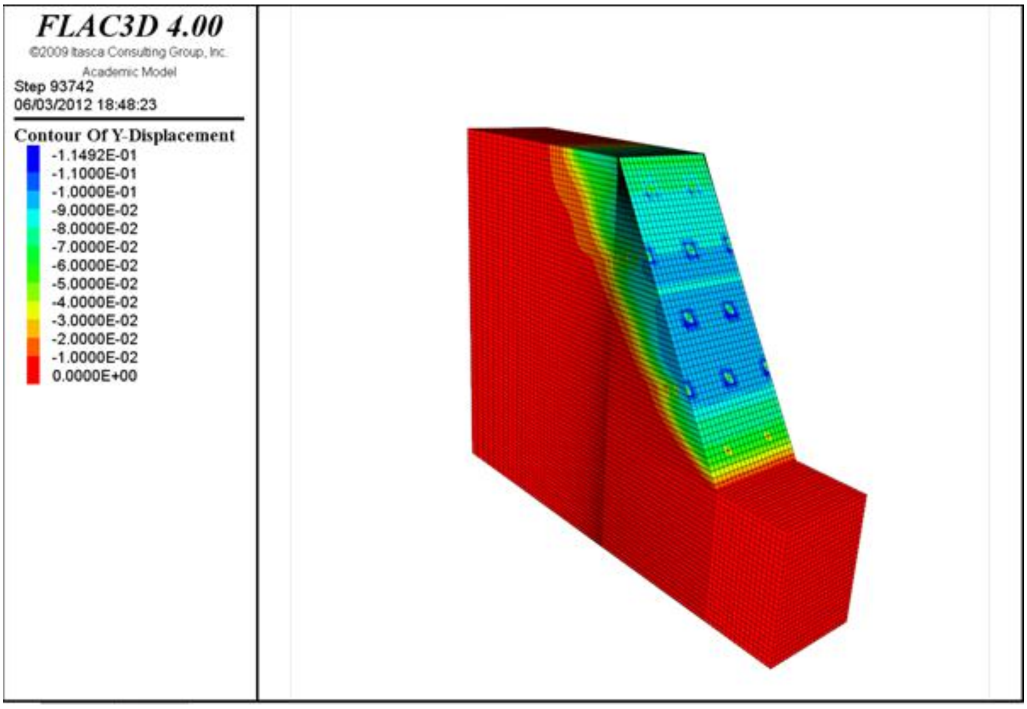


Fig. 4.44 Displacements occurring with a spacing of the nails of 2.0 m (60°)

It shows high values of the displacements compared to ones found with a spacing between the nails of 1.5 m. This picture demonstrates the possibility to use a larger span of the nail to build a cost effective structure.

4.5 Study of the coupling spring stress acting on the facing in slopes with different inclinations

As written above, the facing was modeled as a *geogrid* elements. That is due to the fact that the facing is acting like a membrane and it does not resist neither to bending moment stress nor compressive forces. The *geogrid* element is the only that resists only to tensile forces.

Results of the first two script, with an inclination of the slope of a value, respectively, of 45° and 60° are shown in the following paragraphs.

As for the displacements the data were collected in correspondence of the two rows of nails, with *x*-coordinate from the global point system, respectively of 0.75 and 1.5 m, and a third row of points was collected between these two rows of nails with x-coordinate of 1.125 m.

The first vertical column has the nails at a height of: 2.25, 5.25 and 8.25 m. The second vertical column of nails presents the nail at a height of: 0.75, 3.75, 6.75 and 9.75 m.

4.5.1 Coupling spring stress in the first model – 45°

The coupling spring stress resulting in the first output data are shown in fig 4.45

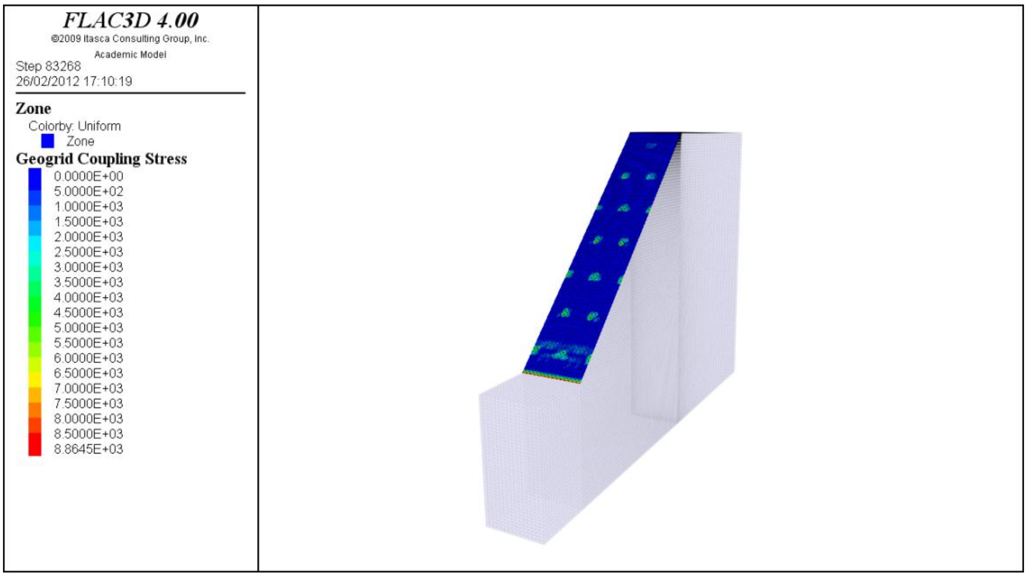


Fig. 4.45 – Coupling spring stress in the first slope

These data will be used to developed the second model: the micro-scale model, obtained with the FEM software *Straus7*. Data show that the maximum value of the coupling stress is acting in correspondence of zone at the top of the nail . That means that there is the risk of puncturing in the zone nearby the head of the nails. The value registered is about 7.5 kPa in the “border” nails (at a height of 2.25 and 5.25m) and about 6 kPa in the nail placed in the middle of them (at a height of 3.75 m). Between two rows of nails the stress acting is not increasing with the displacements hence the soil pressure, in fact the data registered are contained in a range between 1 and 1.5 kPa. That is due for the higher stiffness of the nails compared to the stiffness of the wire mesh. That confirm the distribution of the stress depends on the stiffness of the element and it also means that in this case, with very low values of displacements the steel wire mesh is not providing a structural function.

4.5.2 Coupling spring stress in the second model – 60°

The coupling spring stress resulting in the first output data are shown in fig. 4.46.

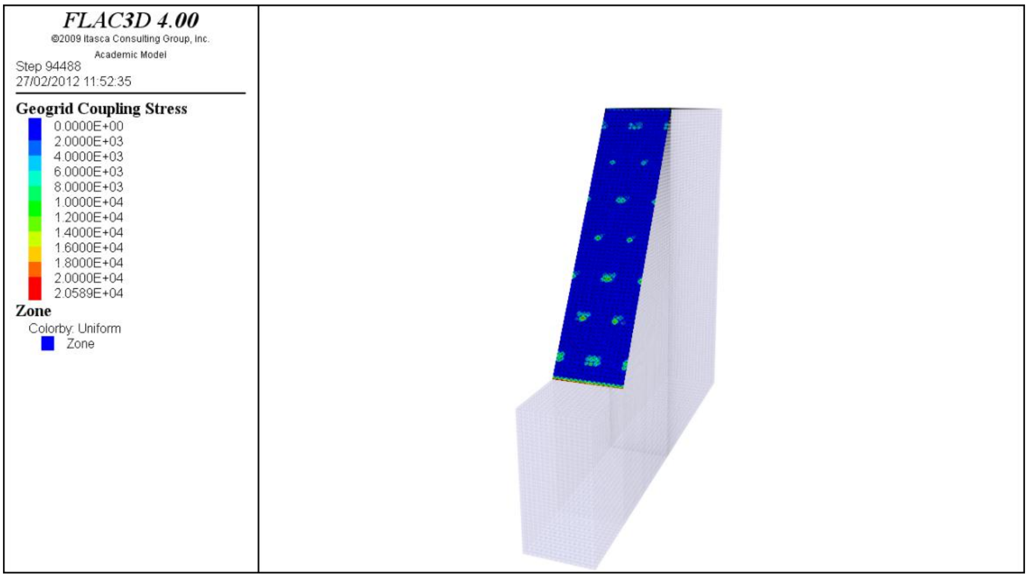


Fig. 4.46 – Coupling spring stress in the second slope

Also in this case data show that the maximum value of the coupling stress is acting in correspondence of zone at the top of the nail . A risk of puncturing in the zone nearby the head of the nails is still possible. The value registered is about 21 kPa in the “border” nails (at a height of 2.25 and 5.25m) and about 20 kPa in the nail placed in the middle of them (at a height of 3.75 m). Between two rows of nails the stress acting is not increasing with the displacements hence the soil pressure, in fact the data registered are contained in a range between 2 and 4 kPa.

4.6 Comparison between different type of facing with numerical analysis

4.6.1 Comparison between hard and flexible facing

In this paragraph a comparison between the slope with hard facing and the slope with flexible facing is studied. The inclination of the slope is of 75° and the spacing between two different rows of nails is of 1.5 m.

The scripts run for this study are the script 3 and the script 5. The data of the script 3 has been studied in the previous paragraphs and they show that a slope with an inclination of 75° with a spacing of 1.5 m is going to a failure since the fourth step of excavation.

At the third step of excavation the output displacements are shown in fig. 4.47.

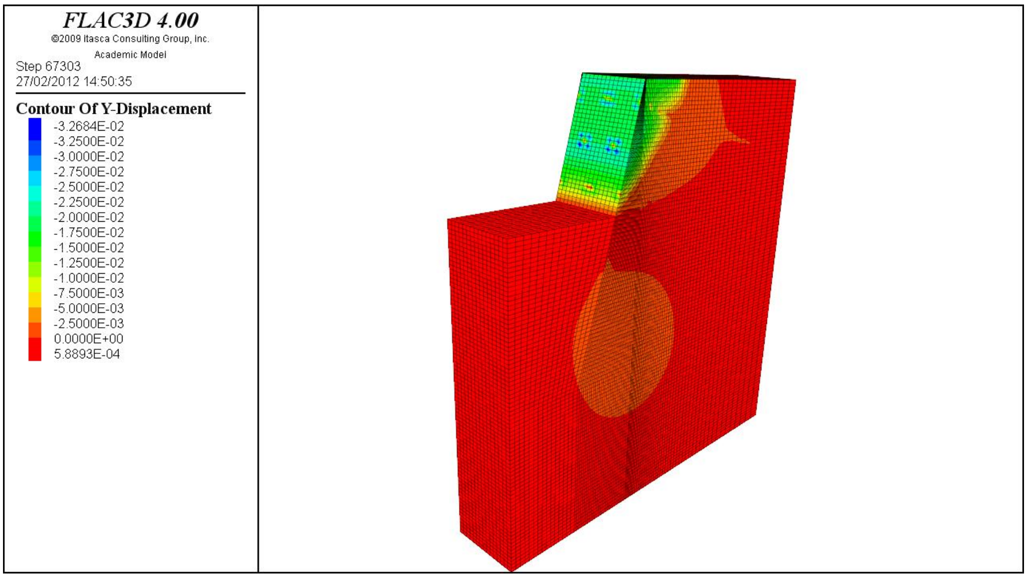


Fig. 4.47 – Displacements at the third step of excavations in script 3

As it is possible to see, the value of the displacements occurring at the third step of excavation, equal to a cut with an height of 4.5 meters, are already higher than 3 cm and. In fact, at the fourth step of excavation, when the slope is not achieving anymore a state of equilibrium, the maximum displacements occurring in the slope has a value of about 18 cm and it is developing in the zone nearby the third row of nails (fig 4.48).

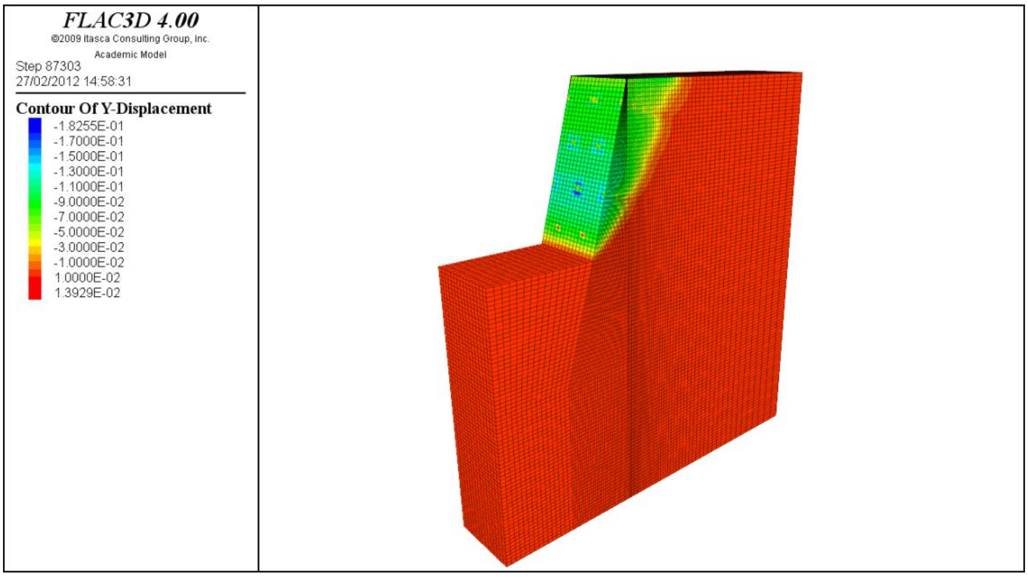


Fig. 4.48 – Displacements at the fourth step of excavations in script 3

If these data are compared with those resulted from the study of the same slope but with a hard structural facing made with *shotcrete* (fig. 4.49), the magnitude of displacements is highly different. That confirm the effective structural function of the hard facing for high values of the inclination of the slope.

The maximum displacement amount now of value of about 8 mm. This confirm that if the displacements has to be controlled because other structure are situated in the proximity of the soil nailed wall, a structure with a hard facing has to be built.

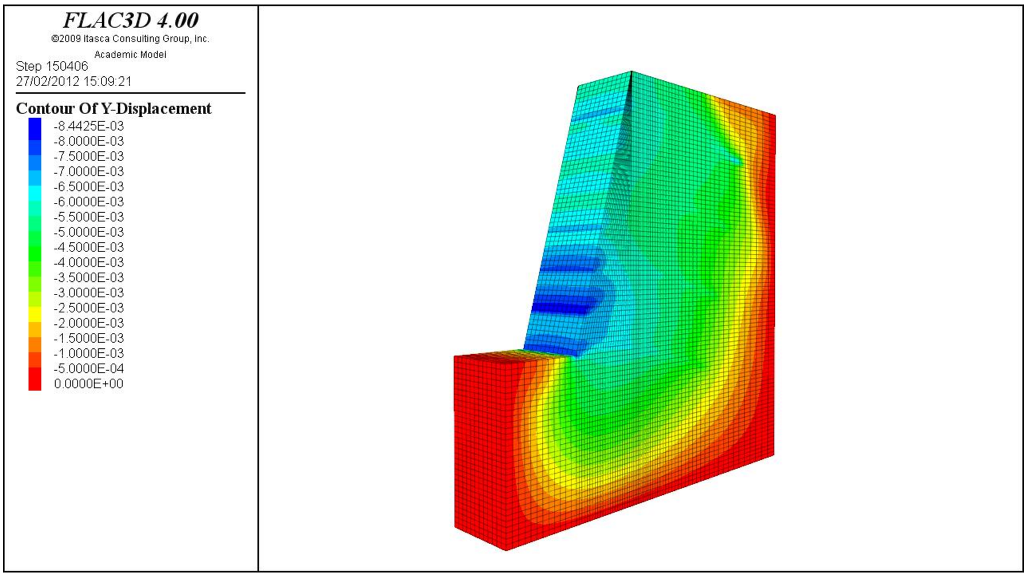


Fig. 4.49 – Displacements occurring in script 5

With this comparison is possible to note that the order of magnitude of the stress occurring in the facing is twice higher than that occurring in a flexible facing structure. That is due, of course, for the high inclination and the higher value of the stiffness of the *shotcrete*, however, it means that this type of facing can achieve to a high level of stress.

The stress developing in the highest row of nails is shown in fig. 4.50.

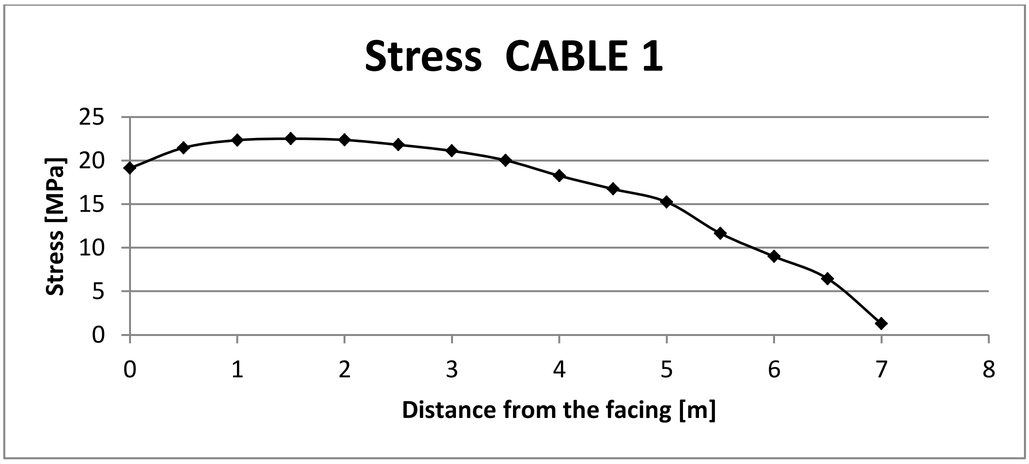


Fig. 4.50 – Stress developing in the first row of nails

From this graph is possible to notice that the maximum value of the stress acting along the nail is lower than that occurring in a soil nailed wall with a flexible facing built with a lower inclination. That is due, obviously, to the distribution of the stress by the different stiffness of the elements composing the structure, but that means that the role of the nails in structures with this kind of facing is not that significant with this value of the spacing between the nails.

It is also notable that the stress along the nail starts with a higher value. This is the bond effect of the hard structural facing. In a structure with flexible facing this effect is achievable only with the use of steel plates like head of the nails.

4.6.2 Comparison between soft and flexible facing

Slope inclination of 45°

In this paragraph a comparison between the slope with flexible facing and the slope with soft facing is studied. The inclination of the slope is of 45° and the spacing between two different rows of nails is of 1.5 m.

The scripts run for this study are the script 1 and the script 6. The data of the script 1 have been studied in the previous paragraphs and they show that a slope with an inclination of 45° with a spacing of 1.5 m is reaching the equilibrium very easily.

For that reason the comparison with a slope without facing was conducted to understand whether the facing act an important role in the sustainability of the structure itself or it only provides a function of erosion control.

The cross section of this structure is shown in fig 4.51:

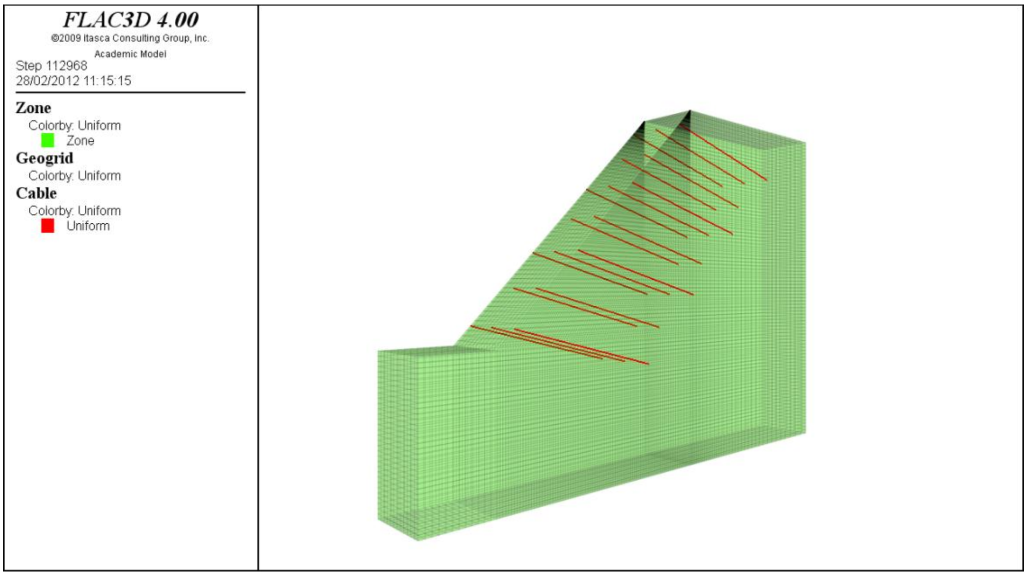


Fig. 4.51– Cross section of the structure with soft facing

A comparison of the displacements developing along the surface was made and it is possible to see in fig 4.52 and fig 4.53 how the variation of the displacements between the two systems with different type of facing is insignificant.

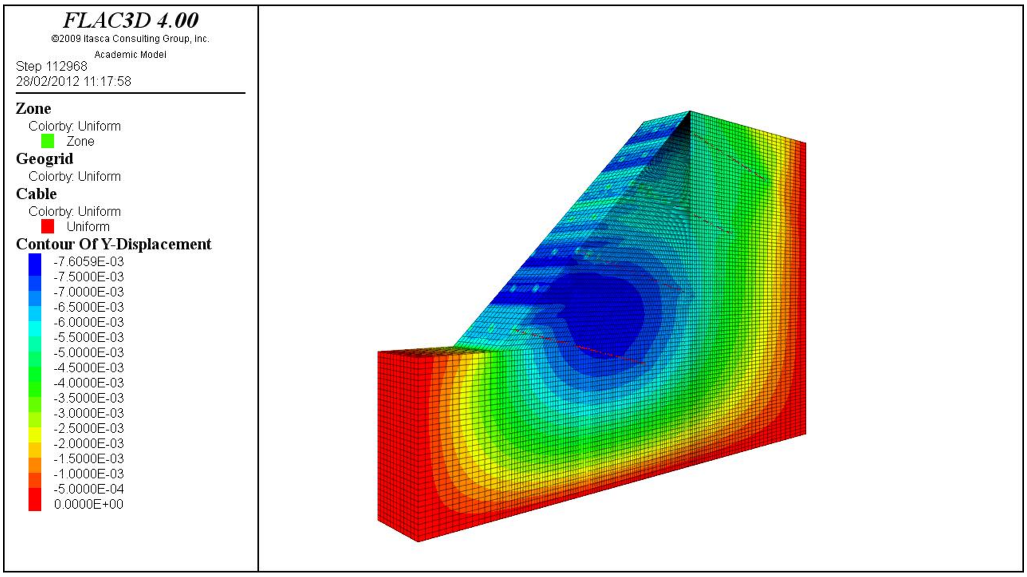


Fig. 4.52 – Deformations occurring with soft facing system

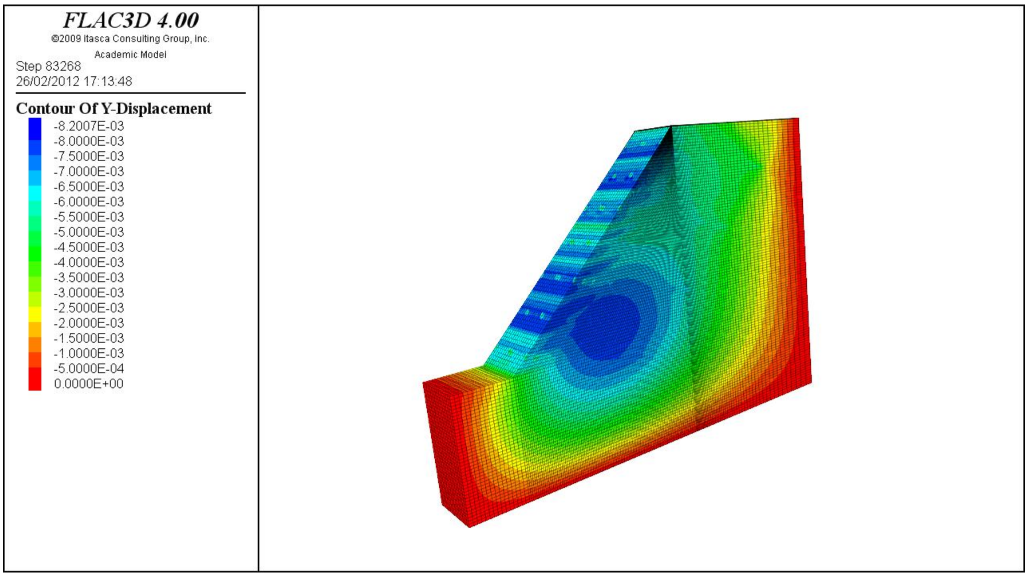


Fig. 4.53 – Deformations occurring with flexible facing system

Of notable importance is to say that the higher values of the deformation developing in the model with flexible facing is due to the higher degree of refinement of the mesh use for the first script compared to the refinement factor used in the soft facing model.

Even a comparison between the stresses acting along the nails in the different models were conducted to understand how the different use of different type of facing acts. Results are shown in fig 4.54. As it is possible to see, the stress acting along the nails in a structure with an inclination of the slope of 45° is approximately the same. That means that a structure with characteristics of the soil and elements of those kind, it is possible to use a soft type of facing. So the flexible facing is doing very little in the 45 degrees case, and for these soil properties, is not required for structural reasons.

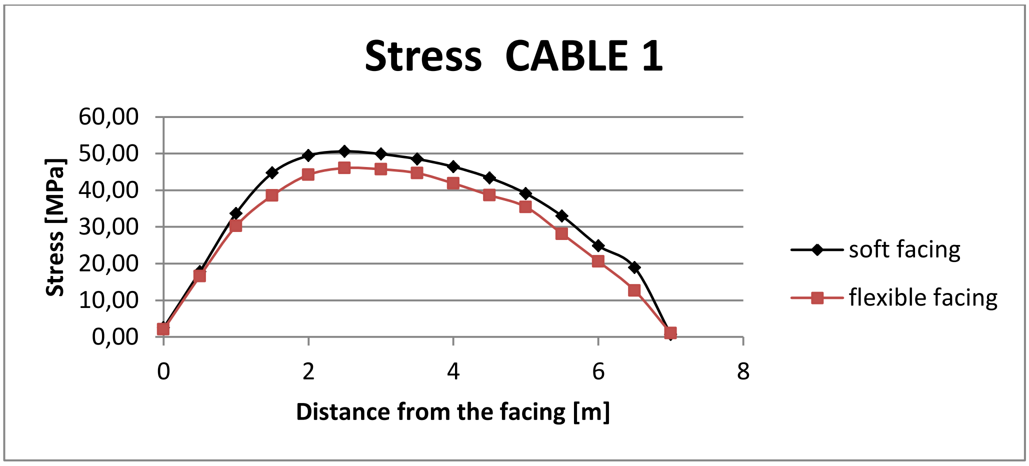


Fig. 4.54 – Stress acting along the nails with different type of facing

Slope inclination of 60°

As seen above there is not a big difference with a low inclination of the slope. Now, the different behavior of these two different types of facing with an inclination of the slope of 60° is studied. In the following figure (4.55) the displacements occurring with a soft facing are shown.

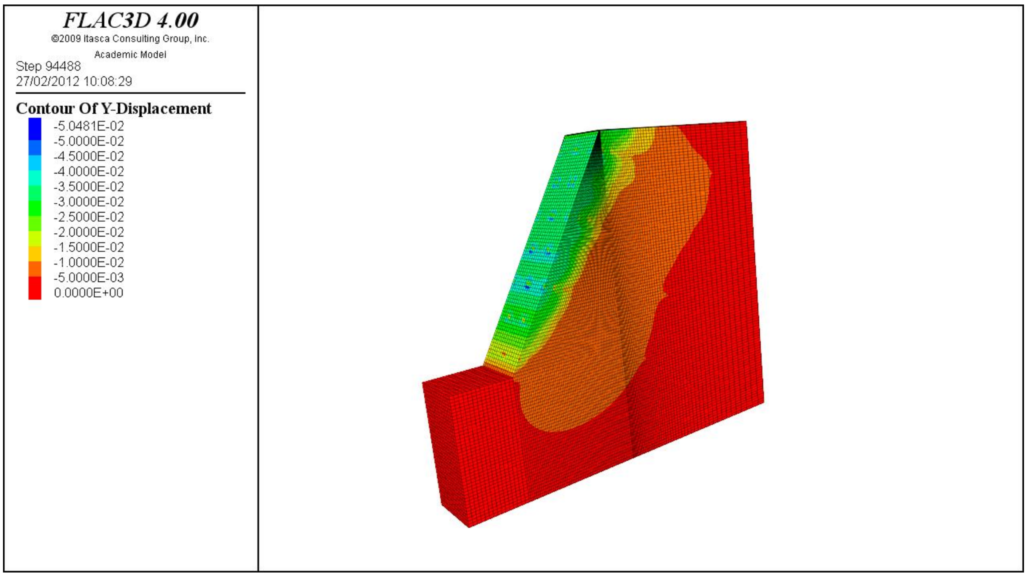


Fig. 4.55 – Displacements in soft facing system with an inclination of 60°

These results compare with the ones output in the second script (fig 4.56) show that with the use of a steel wire mesh as flexible facing, a big reduction of the displacement along the wall surface occur.

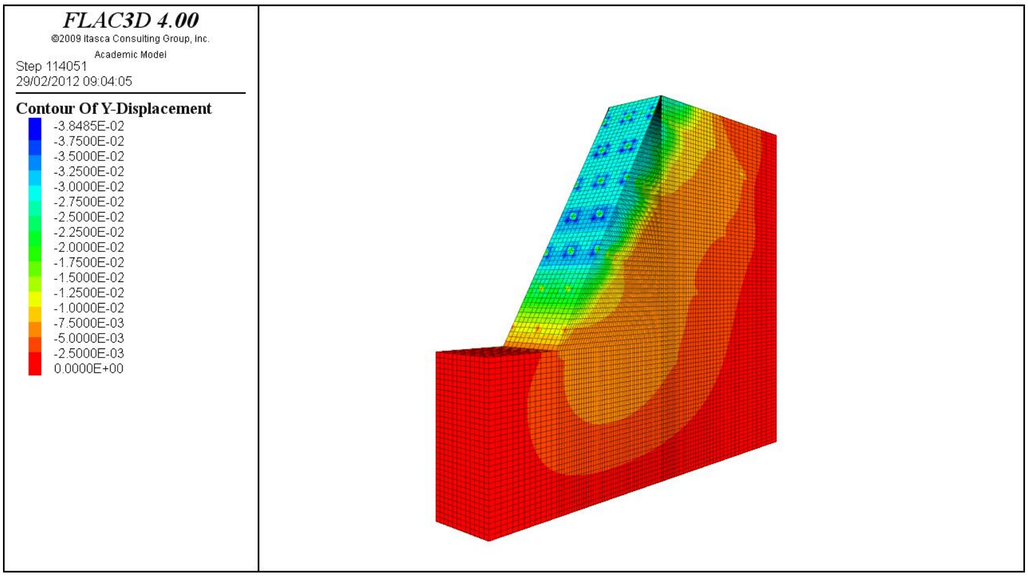


Fig. 4.56 – Displacements in flexible facing system with an inclination of 60°

Even a comparison between the stresses acting along the nails in the different models were conducted. Results are shown in fig 4.57. As it is possible to see, the stress acting along the nails in a structure with an inclination of the slope of 60° is bigger with the use of a soft facing systems.

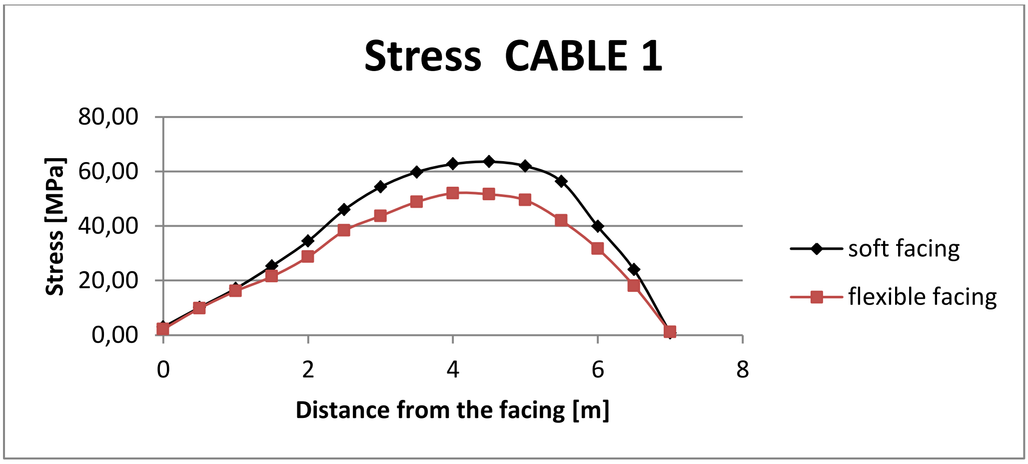


Fig. 4.57 – Stress developing with different types of facing

5. NUMERICAL ANALYSIS OF THE STRESS ACTING IN STEEL WIRE MESHES.

In *FLAC^{3D}* the facing is modeled as a *geogrid* element with six degrees of freedom. That means the *geogrid* is acting like a membrane element. It resists membrane but does not resist bending loading.

The element is considered like an homogeneous plate and it is not possible to understand how the steel hexagonal wire mesh is resisting to this action.

The aim of this chapter is to compare the membrane stress acting in the *geogrid* element with the nominal tension strength of the wire mesh. To do this comparison *Maccaferri Rocknetfall*'s properties are considered: the tension strength for this element is about 350 – 500 N/mm^2.

To study how this stress is acting in the different elements composing the wire mesh, a model with the FEM software *Straus7* has been developed. A simplified model of the behavior of the wire mesh is obtained by the employment of a multi-scale model technique: starting from the macro-scale behavior it is possible to study the micro-scale behavior and compare the results obtained in the macro-scale model with the results obtained in the micro-scale one.

In the macro-scale model, developed with *FLAC^{3D}*, is possible to read the membrane stress in some observation points, defined before the implementation. In the scripts with a nail spacing of 1.5 meters , these point where collocated on the nails and in the points in the middle of these two rows.

5.1 Micro-scale model

For the micro-scale model, *beam* elements were used to define the hexagonal elements of the mesh and this is one of the limits of this model. In fact, it is not possible to study these elements with *cable* or *trus* elements, which have a deformational behavior closer to the real behavior, but the calculation of the stress acting in the elements is denied because of the lability of the model itself. Using *beam* element is possible to compare the stress acting in the macro-scale *geogrid* with distributed forces acting on the edge of the mesh in the micro-scale model.

Three different types of wire mesh dimension were designed and modeled. This choice was made relating to the most used type of wire mesh in soil nailing technique system. These three types are:

- 60 x 80 mm;

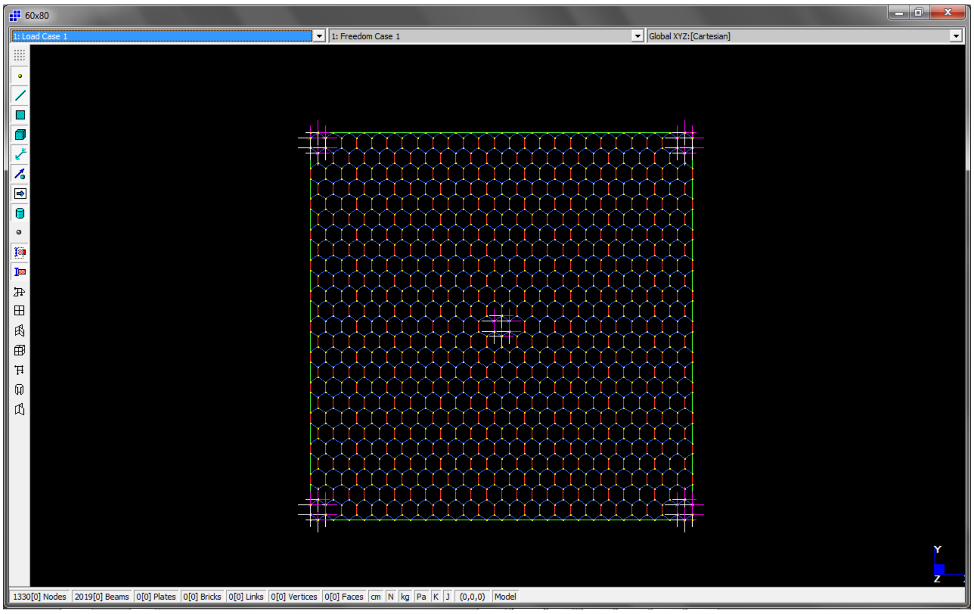


Fig. 5.1 60x 80 mm wire mesh

- 80 x 100 mm;

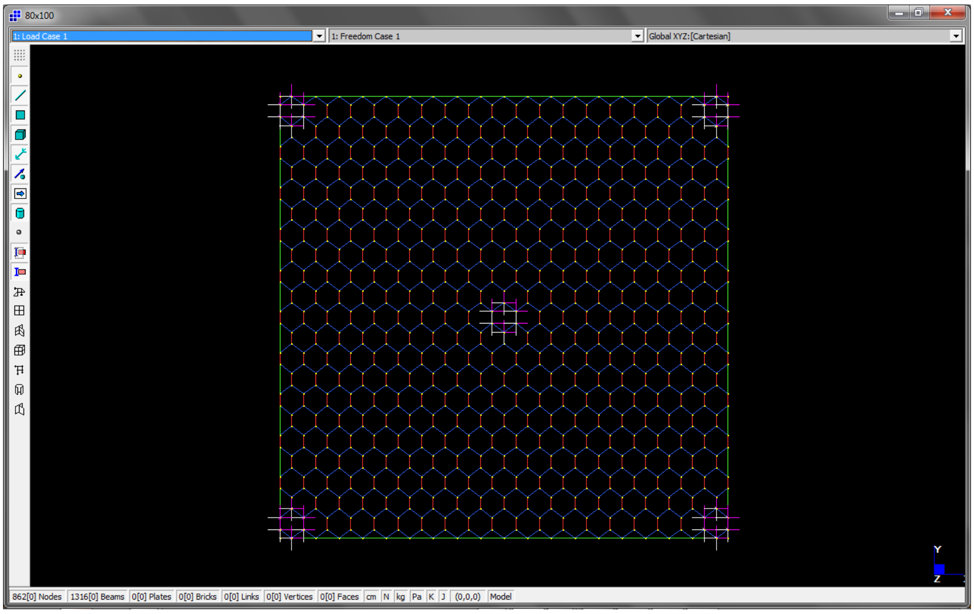


Fig. 5.2 80 x100 mm wire mesh

- 100x120 mm;

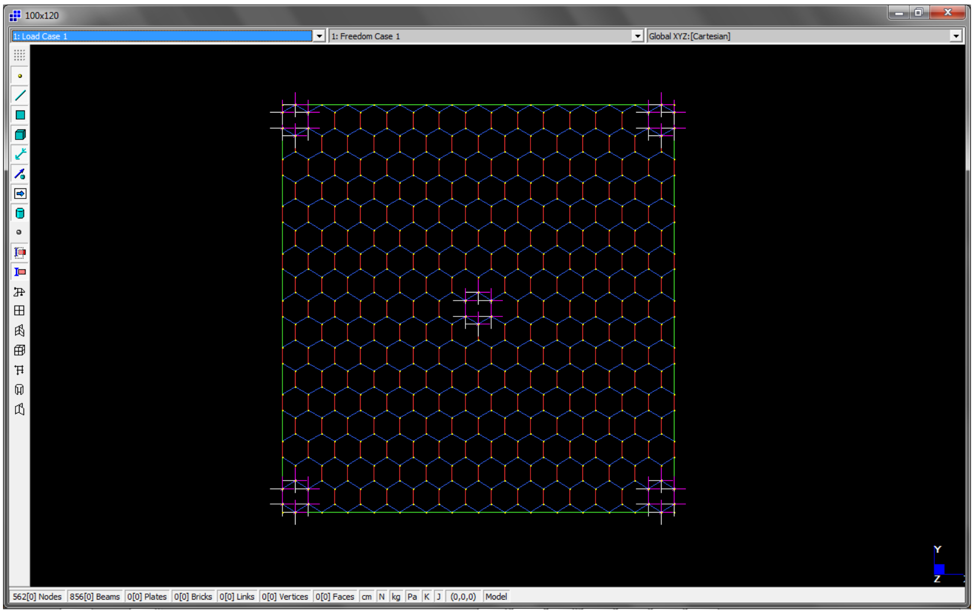


Fig.5.3 100 x120 mm wire mesh

As written above the considered area corresponds to a steel wire mesh area of 1.5 x 1.5 m, exactly the distance between two rows of nails. In fact, the point where the nails are installed are fixed in the model. A further place in this area was fixed that corresponds to the nail installed in the intermediate row of nails.

It is important to say that the calculation of the stress acting in the single elements of the wire mesh in the micro-scale model are considered as *total fiber stress*. Although in the macro-scale model developed in $FLAC^{3D}$ the stress considered was a *membrane stress,* to consider the real mechanism of behavior of the wire mesh element and to compare the stress acting along a single part composing this wire mesh with the nominal tensile strength of the element itself, this assumption must be taken.

It is possible to describe this behavior in this manner: in the macro-scale behavior the steel wire mesh is acting as an homogeneous element, as a sheet of paper, not resisting to bending moment forces. For that reason the wire mesh was modeled as a membrane structure in the macro-scale model. But if the micro-scale behavior is studied, it is possible to considered, the bond within every part of the steel wire mesh as rigid. In this way the single elements are supposed to resist to bending moment forces. In this case the *total fiber stress* acting in the elements must be taken in account .

5.2 Study of the stress acting in the steel wire mesh

The analysis of the stress developing in the steel wire mesh was conducted for all the scripts modeled with a flexible type of facing. Results of these analysis are shown in the following paragraphs.

5.2.1 Stress acting in the wire mesh of the structure modeled in script1

The membrane stress acting in the first implemented model is shown in fig 5.4 This structure has an inclination of the slope of 45° and a spacing of the nails of 1.5 m.

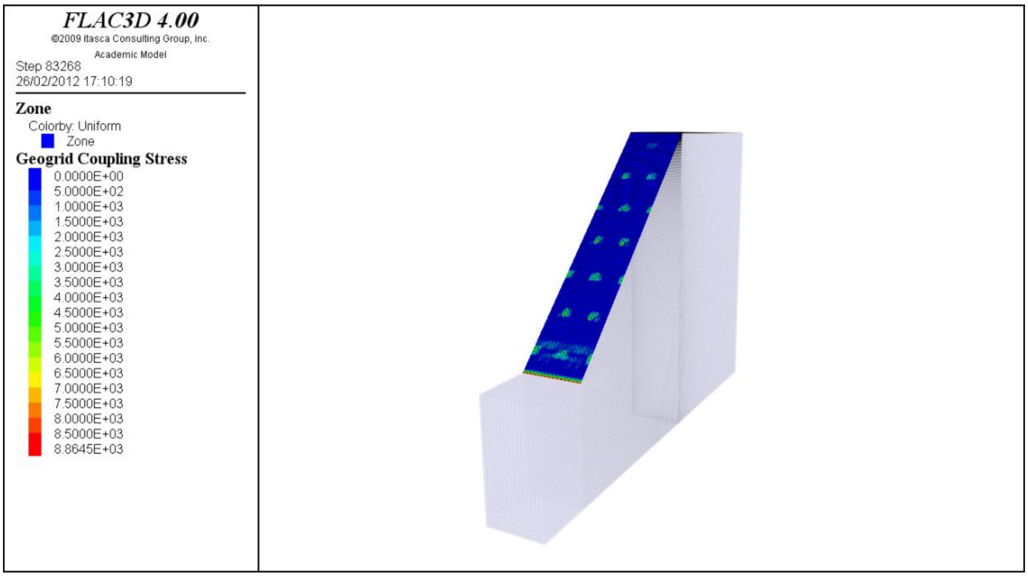


Fig. 5.4 – Coupling spring stress in the first slope

The values of the coupling stress were studied in correspondence of the points were nails were installed and between two different rows of nails.

In the first structure, the coupling stress acting along the wire mesh is about 9 kPa nearby the head of the nails and about 2 kPa between two different rows of nails. These data were input in the *Straus7* models.

The following result of the first script, in this case, were carried out using a mesh with dimensions 60 x 80 mm (fig.5.5)

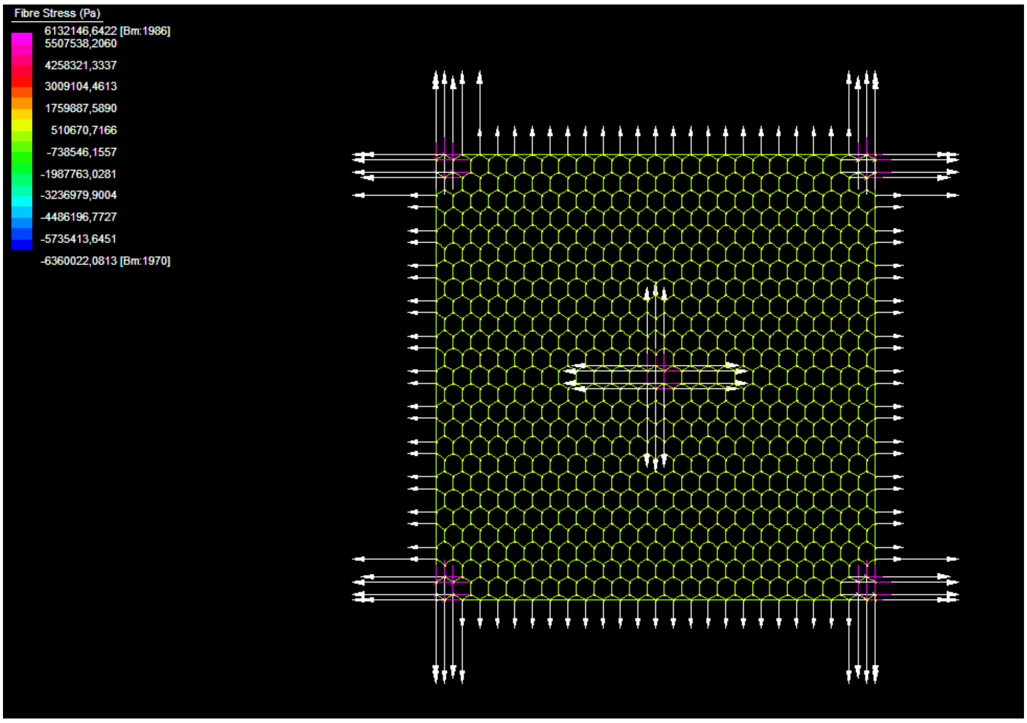


Fig. 5.5 – Total fiber stress in the steel wire mesh with dimension 60x80 mm

In this case the maximum *total fiber stress* has a value of $6\ \frac{N}{mm^2}$.

This value compared with the tensile strength of the wire mesh that has a value of 350-500 $\frac{N}{mm^2}$ is largely satisfied.

The following results of the first script , in this case, were carried out using a mesh with dimensions 80 x 100 mm. Results are shown in fig 5.6

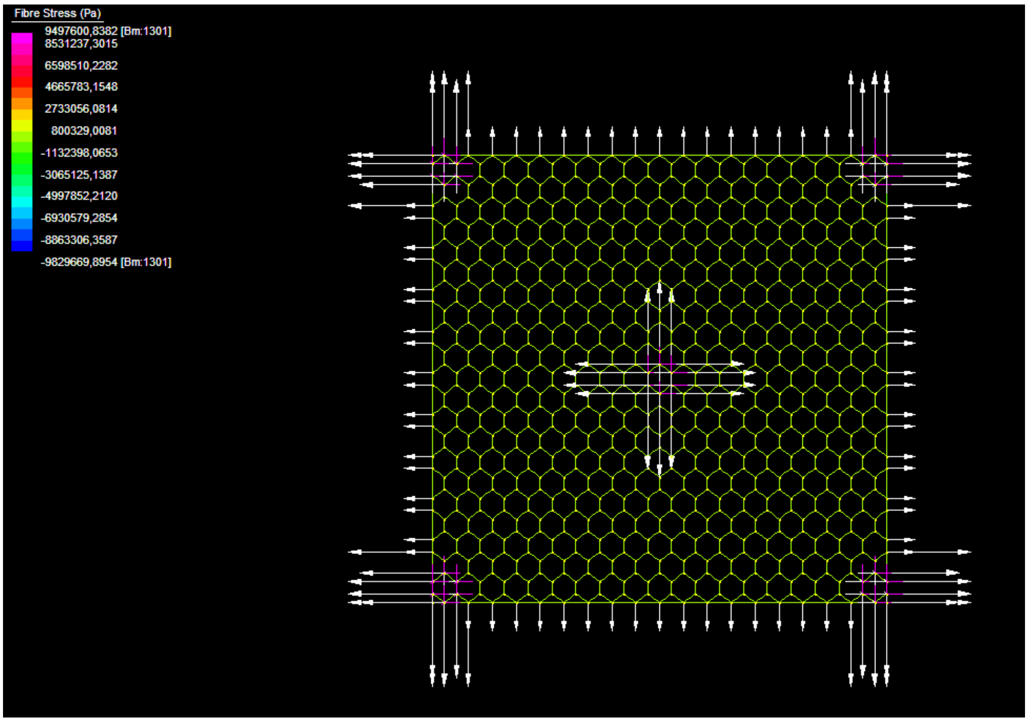


Fig.5.6 Total fiber stress for a 80x100mm wire mesh

In this case the maximum value of the *total fiber stress* has a value of $9 \frac{N}{mm^2}$.

Even in this case value compared with the tensile strength of the wire mesh that has a value of 350-500 $\frac{N}{mm^2}$ is largely satisfied.

The following results of the first script, in this case, were carried out using the wire mesh model with larger dimension: 100 x 120 mm

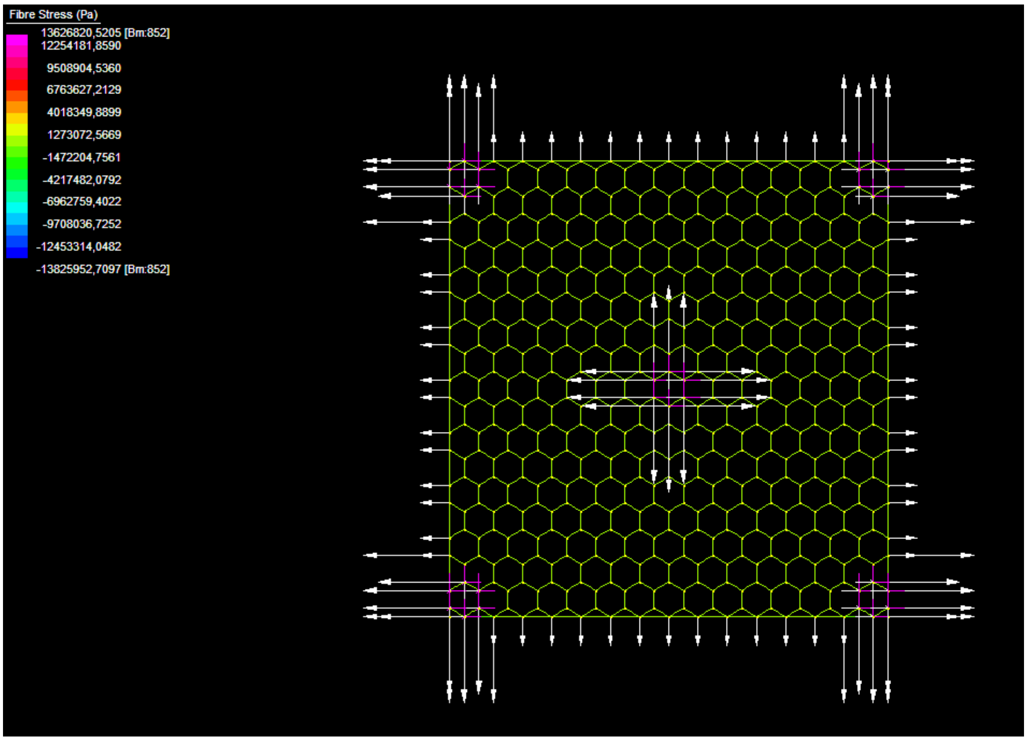


Fig 5.7 Total fiber stress for a 100x120mm wire mesh

The maximum *total fiber stress* output has a value of 13 $\frac{N}{mm^2}$.

Also in this case, the steel wire mesh is not subject to a stress that could bring the element to failure.

This results confirm that the presence of the steel wire mesh in a slope with an inclination angle of low magnitude has not structural function but only a function of erosion control.

5.2.2 Stress acting in the wire mesh of the structure modeled in script2

The membrane stress acting in the first implemented model is shown in fig 5.8. This structure has an inclination of the slope of 60° and a spacing of the nails of 1.5 m.

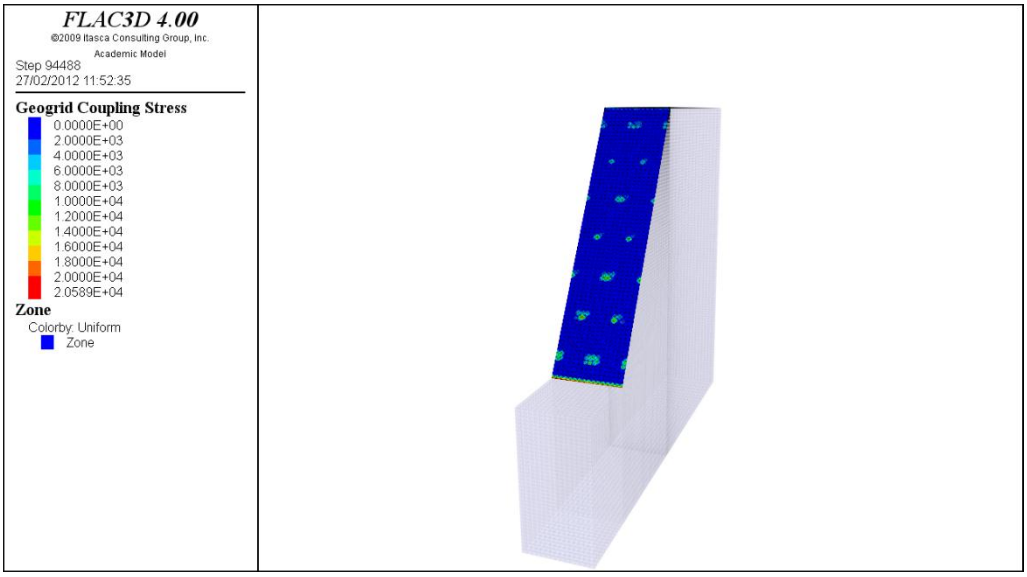


Fig. 5.8 – Coupling spring stress in second slope

The values of the coupling stress were studied in correspondence of the points were nails were installed and between two different rows of nails.

In the second structure, the coupling stress acting along the wire mesh is about 21 kPa nearby the head of the nails and about 5 kPa between two different rows of nails. These data were input in the *Straus7* models.

The following result of the second script, in this case, were carried out using a mesh with dimensions 60 x 80 mm (fig.5.9).

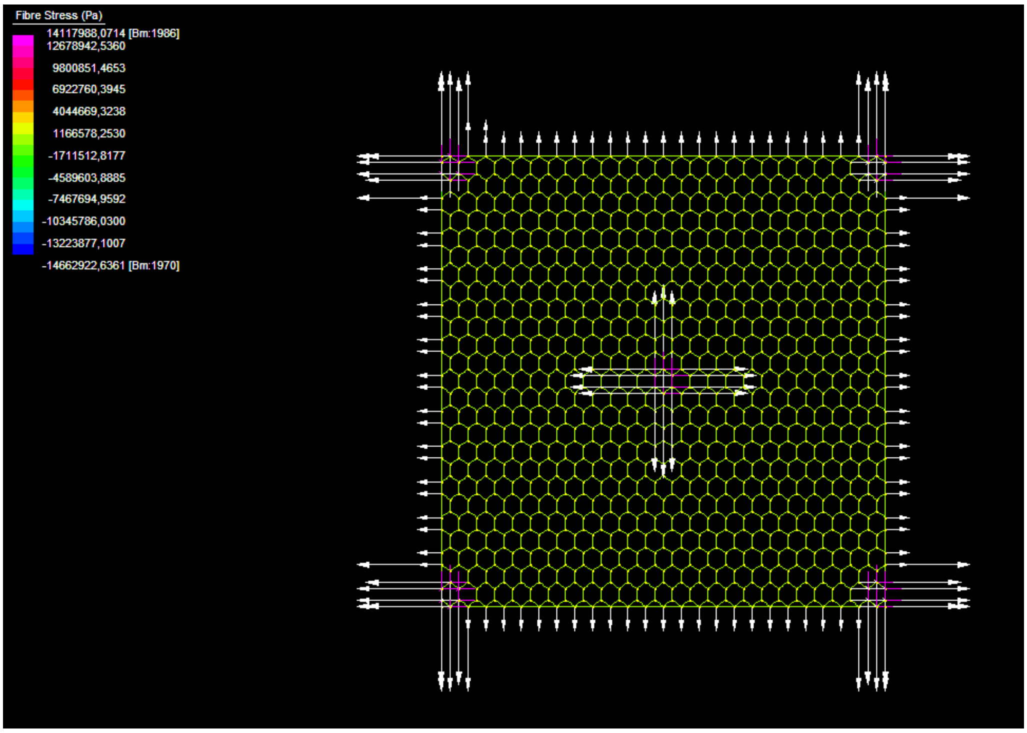


Fig. 5.9 – Total fiber stress in the steel wire mesh with dimension 60x80 mm

In this case the maximum *total fiber stress* has a value of $14\,\frac{N}{mm^2}$.

This value compared with the tensile strength of the wire mesh that has a value of 350-500 $\frac{N}{mm^2}$ is largely satisfied.

The following results of the second script , in this case, were carried out using a mesh with dimensions 80 x 100 mm. Results are shown in fig. 5.10.

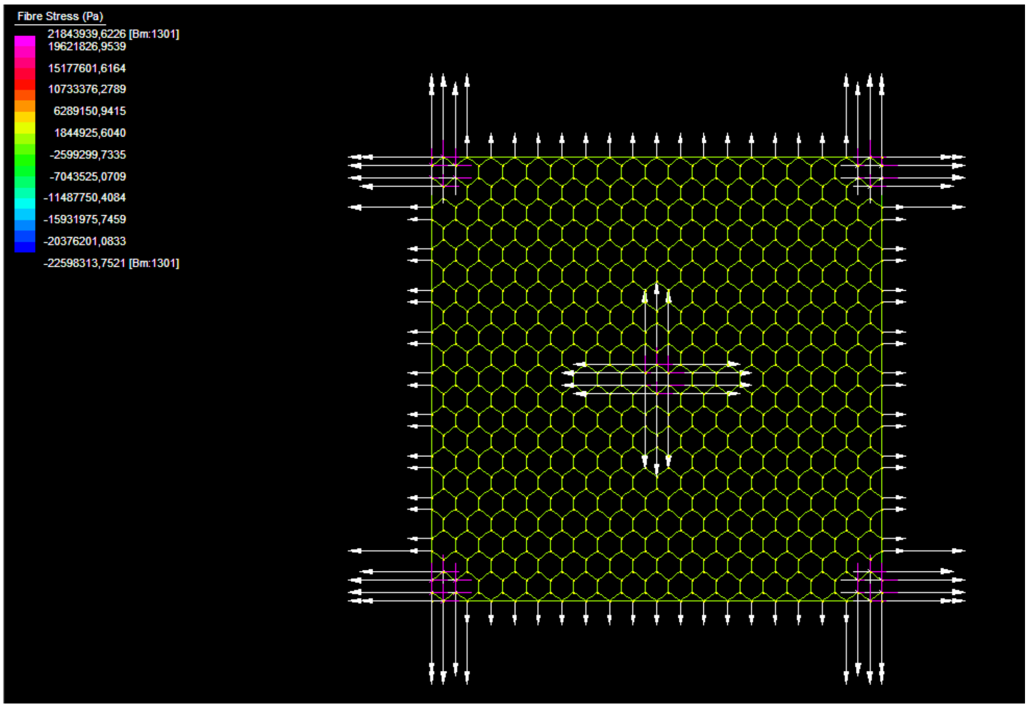


Fig. 5.10 Total fiber stress for a 80x100mm wire mesh

In this case the maximum value of the *total fiber stress* has a value of $22\,\frac{N}{mm^2}$.

Even in this case value compared with the tensile strength of the wire mesh that has a value of 350-500 $\frac{N}{mm^2}$ is largely satisfied.

The following results of the second script, in this case, were carried out using the wire mesh model with larger dimension of 100 x 120 mm. They are shown in figure 5.11.

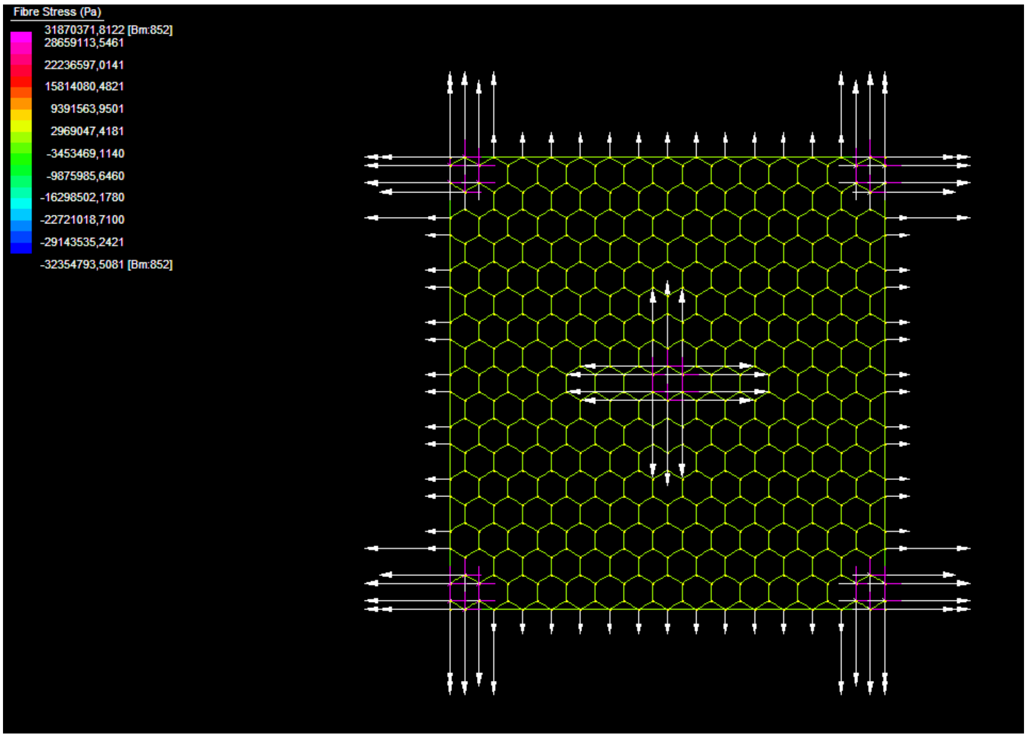


Fig. 5.11 Total fiber stress for a 100x120mm wire mesh

The maximum *total fiber stress* output has a value of 32 $\frac{N}{mm^2}$.

Also in this case, the steel wire mesh is not subject to a stress that could bring the element to failure.

5.2.3 Stress acting in the wire mesh of the structure modeled in script4

The membrane stress acting in the fourth implemented model is shown in fig. 5.12. This structure has an inclination of the slope of 60° and a spacing of the nails of 2.0 m.

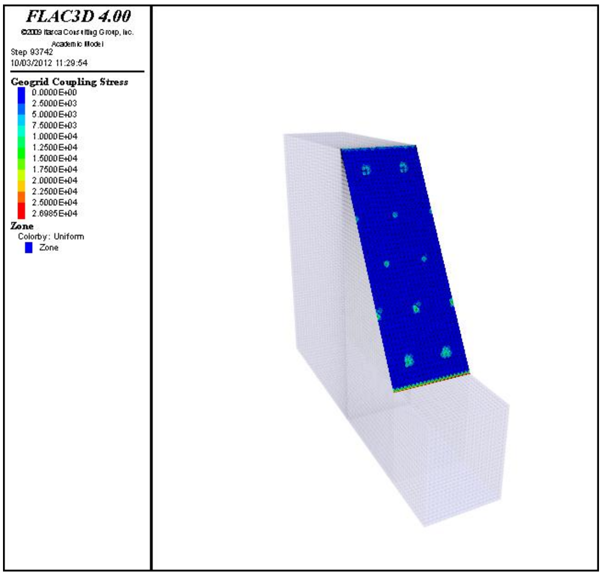


Fig. 5.12 – Coupling spring stress in second slope

The values of the coupling stress were studied in correspondence of the points were nails were installed and between two different rows of nails.

In the fourth structure, the coupling stress acting along the wire mesh is about 30 kPa nearby the head of the nails and about 8 kPa between two different rows of nails. These data were input in the *Straus7* models.

The following result of the fourth script, in this case, were carried out using a mesh with dimensions 60 x 80 mm (fig.5.13).

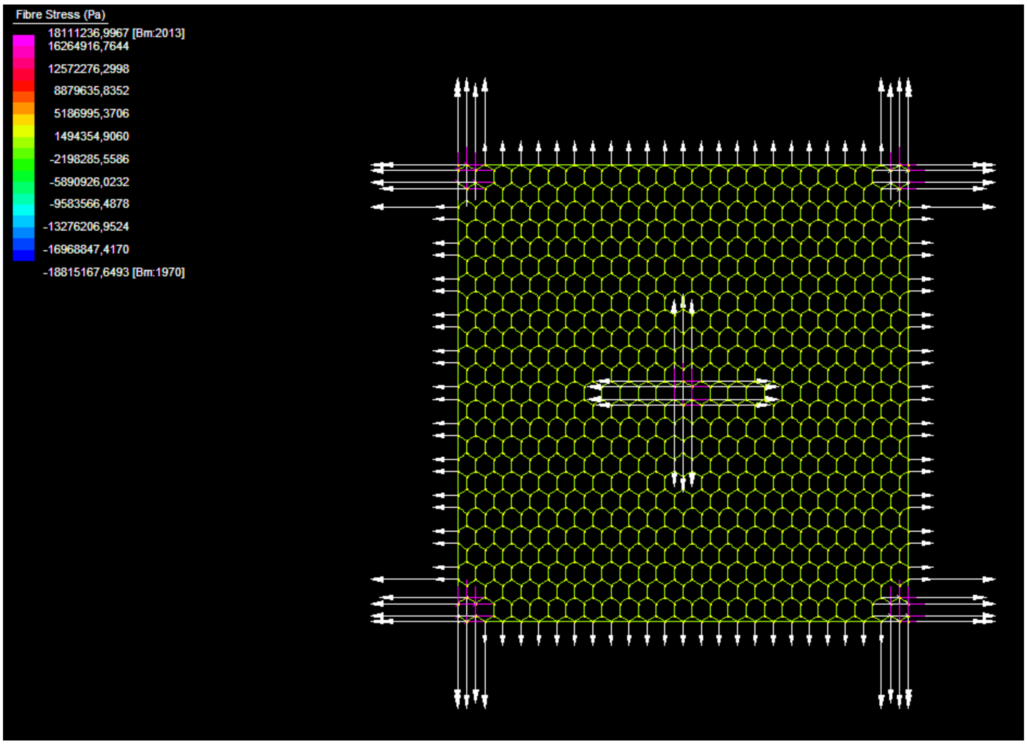


Fig. 5.13 – Total fiber stress in the steel wire mesh with dimension 60x80 mm

In this case the maximum *total fiber stress* has a value of $18 \frac{N}{mm^2}$.

This value compared with the tensile strength of the wire mesh that has a value of 350-500 $\frac{N}{mm^2}$ is largely satisfied.

The following results of the second script , in this case, were carried out using a mesh with dimensions 80 x 100 mm. Results are shown in fig 5.14

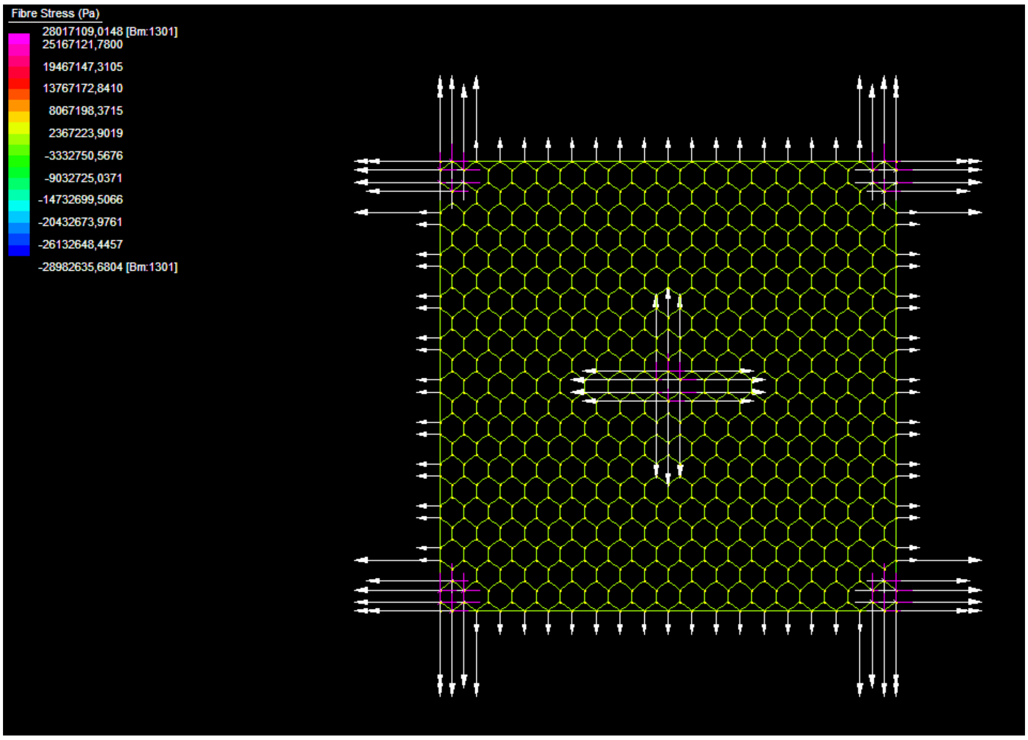


Fig. 5.14 Total fiber stress for a 80x100mm wire mesh

In this case the maximum value of the *total fiber stress* has a value of $28\,\frac{N}{mm^2}$.

Even in this case value compared with the tensile strength of the wire mesh that has a value of 350-500 $\frac{N}{mm^2}$ is largely satisfied.

The following results of the fourth script, in this case, were carried out using the wire mesh model with larger dimension of 100 x 120 mm. They are shown in figure 5.15.

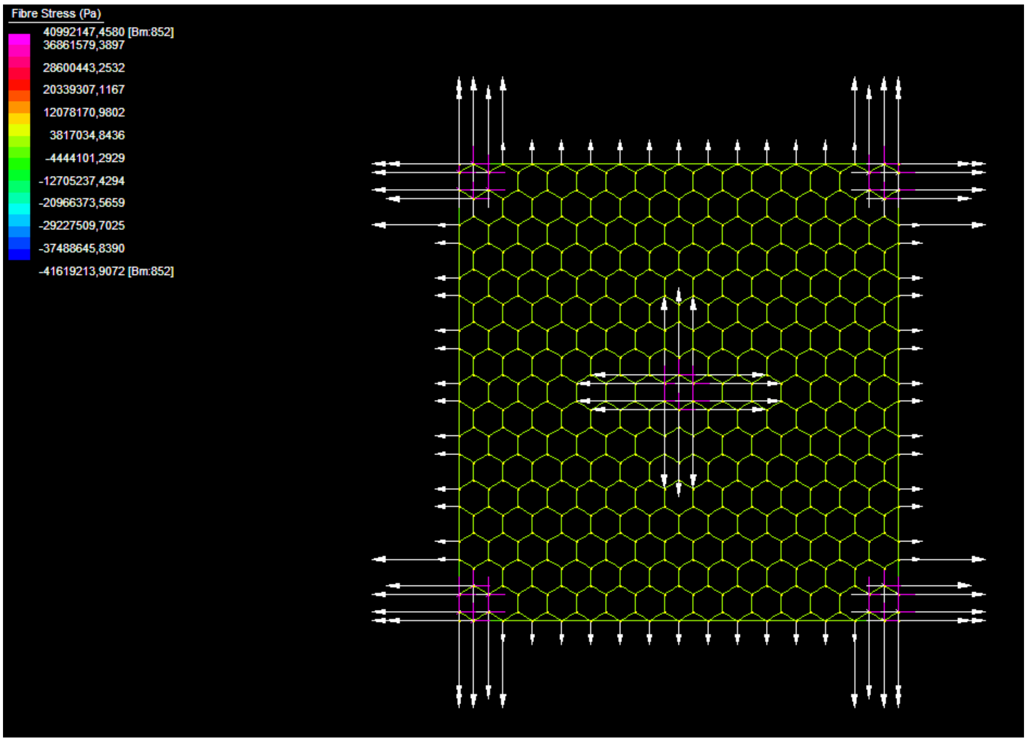


Fig. 5.15 Total fiber stress for a 100x120mm wire mesh

The maximum *total fiber stress* output has a value of 41 $\frac{N}{mm^2}$.

Also in this case, the steel wire mesh is not subject to a stress that could bring the element to failure.

This results show that the presence of the steel wire mesh in a slope with an inclination angle of medium magnitude do not provide the main structural function of the structure but it ensures a control of the displacements and it provide tensile resistance against the punching of the slope.

On the evidence of the shown data it is possible to say that a correct design of the steel wire mesh could permit the construction of a cost effective structure. If the steel wire mesh with structural function is coupled with a non-woven textile *geogrid* with erosion protection function it is possible to use mesh with big spans instead that mesh with small ones.

CONCLUSION

With the aim of study and analyse the behavior of soil nailed structures with flexible facing, numerical models were implemented and run with the three-dimensional finite differences software, *FLAC*^{3D} and with the finite element software *Straus7*. The following conclusions and recommendations were developed based on the finite difference and finite element modeling. Finite difference modeling was effective in predicting the behavior of the soil compared to the data written in the literature. Specifically, the shape and relative magnitude of the deformations and the developing of the stress in the different elements composing the soil nailing structure.

The software *FLAC*^{3D} was used, to implement seven different modeled structures. These structures were studied with different type of facing (hard, flexible, soft) and with different characteristics of the slope (inclination) and the elements (spacing).

Due to lack of available data regarding structures with flexible facing a site with a hard facing system was analysed to compare the model with real slope for which monitoring data was available: that is a structure built in the city of Istanbul with a hard facing technique (*Durgonoglu* et al., 2007). This analysis were conducted to study the sensitivity of the model and the elements which come into play in a numerical model of a soil nailed structure.

Once the model was calibrated the other models were run on the basis of a real structure built by the company *Mott Mac Donald*, in the Hindhead Tunnel Scheme of the highway H3, London-Portsmouth. The parameters used in the implementation are the same used in the construction of the real structure.

The first three models that were implemented and developed correspond to three structures with the same soil and elements parameters (mechanical properties, geometric properties and spacing) but with different inclination of the slope, respectively of 45°, 60° and 75°. With the results obtained in these implementations, a comparison with the literature data was possible. These are the developing of the stress along the nails that was studied on every row of nails and on the first row of nails at every construction step and the deformation occurring on the surface of the structures.

From these data is possible to see that a structure built with a sand with a low value of cohesion is reaching a state of equilibrium for inclination angles of low magnitude but it is not achieving the equilibrium with a steeper inclination (75°). In fact, the structure is already presenting a mechanism of failure at the fourth (out of seven) excavation.

In particular, the first structure, with an inclination of the slope of 45° is reaching a complete state of equilibrium without the clear formation of critical slip surface but only with zones nearby the facing which are achieving a low state of plasticity. Furthermore, the deformations occurring in this structure confirm the very low mobilization of the slope. They show a maximum value less than one centimeter.

In the second structure, with an inclination of 60°, instead, is clear to identify a potential critical slip surface with the possibility of the beginning of a failure mechanism. In this case, the stresses developing along the nails and the deformations occurring at the surface are greater than shallower slope. The deformations are achieving a maximum value of about four centimeters. That means a higher mobilization of the soil behind the surface, hence a higher stress occurring along the nails.

For those reasons two structures with a higher spacing of the nails (2.0 meter) were implemented, to study whether it could bring the structure to maintain the state of equilibrium or not, hence if the use of higher spacing could be a cost effective solution. The analysis conducted with an inclination of 45° and 60° show that there is no significant change of values of the stress and the deformation occurring, hence in the achievement of a state of equilibrium.

To understand the behavior of structures with different type of facing, three other models were run. One of this has a hard facing and the other two present a soft facing system. The structure with hard facing has an inclination of the slope of 75° with a spacing of the nails of 1.5 meter and it was compared to the structure implemented in the third script with a flexible facing system. It is possible to see that with these type of facing, the stress developing in the facing is greater, the deformation are lower than those found in the flexible facing system. Furthermore, an equilibrium state is completely achieved. That confirms that for structure with high inclination and/or for structure in which the value of displacement must be contained in a small range, a hard facing system is required.

In the case of structure with soft facing is possible to see that in shallow slopes the equilibrium state is achieved very easily hence the use of a steel wire mesh has only the function of erosion control, not structural. Instead, with steeper inclination of the slope, the deformation occurring in a soft facing structure is doubled. That means the steel wire mesh, now, has structural function.

To study this structural behavior and to understand how the steel wire mesh is acting in this way, a series of models were implemented with the finite element software *Straus7*. With this software the hexagonal wire mesh was modeled with three different dimensions: 60 x 80 mm, 80 x 100 mm, 100 x 120 mm (these correspond to the typical dimensions used in real structure).

So multi-scale models of hexagonal steel wire meshes were implemented comparing the membrane stresses acting in the whole mesh, studied as homogenous element, with the total fiber stress acting on a single element composing the mesh. In this way, the hypothesis of a macro-scale element not resisting to bending stresses but only membrane stresses was made. Instead, in the micro-scale model, the single elements are considered as rigid (that is due to the mechanical properties of the steel) hence they are resisting both membrane and bending stresses.

The wire mesh did not appear to be overstressed in the simulation and, in this aspect, it performed very well.

The results in the micro-scale models confirm that the steel wire mesh is providing a structural function and requires deformation and mobilization of the slope to become functional. That means that significant deformation is required to mobilize the tensile strength of the facing.

Based on these results, it is recommended that use of soil nail walls with flexible facing in sand be limited to walls with low-medium angle of inclination where significant deformations are tolerable. A flexible facing systems does not provide the main contribute for the stability of the wall, but it does help in this way and with its characteristic of allow the growth of vegetation, hence a lower impact in the environment, it can be preferred to hard facing systems when the environmental conditions permit that.

This research could even represent the start of the study of guide lines for the design of a construction technique that represents a cost effective and a low environmental impact solution.

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