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# DESIGNING SOIL NAILED WALLS CONSIDERING PROBLEMATIC ISSUES DURING EXECUTION AND SERVICE LIFE BY THE AMHERST WALL.

Julio Garzón-Roca[a,*], Vicente Capa[b], F. Javier Torrijo[c], Julio Company[d]

[a] Assistant Professor, Department of Geotechnical Engineering, Universitat Politècnica de València, Camino de Vera s/n, 46022, Valencia, Spain.

[b] Graduate Student, Department of Construction Engineering and Civil Engineering Projects, Universitat Politècnica de València, Camino de Vera s/n, 46022, Valencia, Spain.

[c] Associate Professor, Department of Geotechnical Engineering, Universitat Politècnica de València, Camino de Vera s/n, 46022, Valencia, Spain.

[d] Assistant Professor, Department of Geotechnical Engineering, Universitat Politècnica de València, Camino de Vera s/n, 46022, Valencia, Spain.

[*] *Corresponding author*. Tel.: +34 963 877 583; fax: +34 963 877 569; E-mail address: jugarro@upv.es

## Abstract

Soil nailing is a technique commonly used as temporary or permanent earth retention system in soft soils. Habitually the design of a soil nailing focuses on its performance at failure, computing a safety factor, and thus neglecting ground deformations. In this paper, an analysis and comparison of the convenience of the use of the limit equilibrium method and the finite element method for designing a soil nailing is conducted. The assessment considers both the suitability of an easy and fast design process, and the necessity to take into account issues such as ground deformations to avoid problematic consequences that can arise during their execution phase and service life. For performing the analyses a numerical study of the "Amherst wall", a full-scale soil nailed wall built to be an experimental test in last years of the twentieth century, is carried out. A two-step process for designing soil nailed walls is proposed. The first step involves the use of limit equilibrium methods to define the main parameters. The second step deals with the development of a finite element model to consider ground deformations as well as to determine nail forces. An approach based on the use the Mohr-Coulomb model for simulating materials more similar to granular soils and the Hardening Soil model for simulating materials more similar to cohesive soils is also presented in the paper as an answer to numerically model soil-nailed walls in ground situations where the soil is neither pure cohesive nor pure granular.

**Keywords:** Soil nailing; Performance at failure; Ground deformations; Execution phase; Service life; Numerical simulation; Finite element modelling; Limit equilibrium method.

## Introduction

Soil nailing is a low cost, easy construction and common technique used all over the world for enhancing the stability of retaining walls, slopes, and excavations, and it is usually used for temporary or permanent earth retention (Yuan et al. 2003; Holman and Tuozzolo 2009; Ghareh, 2015). Basically, soil nailing is an in situ ground reinforcement technique which consists of introducing a series of reinforcing elements, i.e. nails, into the existing ground with the aim of adding the resistance of those elements to the shear strength of the in-situ ground (Junaideen et al. 2004; Pradhan et al. 2006; Yin and Su 2006; Su et al. 2007; Xue et al. 2011; Seo et al. 2014).

In contrast to anchors and tiebacks (Ehrlich and Silva 2015), nails are passive elements which are not post-tensioned after installation and only mobilize their reinforcing effect when ground movements occur (Sheahan 2000; Sheahan and Ho 2003). Nails are subjected mainly to tension forces but may develop shear forces and resist bending moments when are oriented counterclockwise to the normal direction of the slip surface (Fan and Lou 2008; Ghareh 2015). Reinforcing bars, i.e. rebar, are normally used as nails, but hollow steel section or even solid bars could also be used.

Installation of a soil nailing in an existing ground typically involves conducting a relatively small excavation which remains stable without any retaining system, drilling a row of holes in the slope face, inserting the nail elements and then bonding the nails and the ground by grouting. A shotcrete is normally casted at the slope face to unify the nails, create a uniform surface and work as a retaining system at local scale. This procedure may continue until reaching the total depth of the excavation, building a “soil nailed wall”.

Designing of a soil nailed wall, generally focuses on its performance at failure, which is commonly translated into fulfilling a given value of the safety factor (defined as the ratio of the forces that contribute to stabilize the system to the forces that tend to make it unstable). However, ground deformations are not taken into account in the referred safety factor. Ground deformations may give rise to problems in surrounding buildings and structures (Tan et al. 2015) and may also endanger the safety of the own excavation due to excessive movements of the slopes. Consequently, even though a soil nailed wall fulfills a safety factor it does not necessarily means that the system is properly designed.

This paper presents an analysis and comparison of the use of limit equilibrium methods and finite element methods for designing a soil nailed wall. Even though different numerical methods were developed (Kim et

al. 1997; Sivakumar Babu et al. 2002; Yuan et al. 2003; Liu et al. 2016), nowadays limit equilibrium methods and finite element methods are most commonly used when analyzing this kind of earth retention structures (Smith and Su 1997; Zhang et al. 1999; FHWA 2003; Yuan et al. 2003; Fan and Luo 2008; Singh and Sivakumar Babu 2010; Wei and Cheng 2010; Ghareh 2015). The assessment conducted considers both the suitability of an easy and fast design process, and the necessity to take into account issues such as ground deformations to avoid problematic consequences that can arise during their execution phase and service life.

For doing this, a numerical study of the soil nailed structure "Amherst wall" is carried out (Sheahan 2000). This is one of the few existing full-scale soil nailed walls built to be an experimental test, along with the test conducted in 1986 for the French national research project "Clouterre" (Plumelle et al. 1990). There exist some examples of instrumented soil nailed walls (Holman and Tuozzolo 2009; Wood et al. 2009), but these are normally real infrastructures, so failure is generally not occurring. On the other hand, the Amherst wall was led deliberately to failure, which enables the study of a soil nailed wall at collapse. Two models of the Amherst wall using the finite element method and limit equilibrium method, respectively, are built, and an analysis in terms of performance at failure, load distribution patterns developed in the nails and ground deformations issues is undertaken so as to establish the convenience of using each of the methods under study.

## The Amherst wall

### *General description*

The "Amherst wall" was built in 1997 at the University of Massachusets-Amherst (UMass-Amherst) National Geotechnical Experimentation Site (NGES), near the town of Amherst, Massachusetts. This was a full-scale soil nailed wall built as an experimental test carried out to increase the knowledge about the soil nailing technique.

The Amherst wall reproduced a typical excavation in a soft soil using the soil nailing technique. Excavation of the ground was made in three main phases. The first excavation phase reached around 2.5 m; the second excavation phase reached a total depth of 3.6 m. After each excavation phase, a series of nails were installed in the ground, the vertical slope face was shotcreted and the head of the nails were hand-tightened. The third excavation phase corresponded to an overexcavation (no nail or shotcrete was installed), and was carried out

until the failure of the wall. Works were developed from August 18th, 1997 to September 7th, 1997 (Oral and Sheahan 1998). More information about the construction process can be found in Sheahan (2000).

**Figs. 1 and 2** show the wall's front elevation (**Fig. 1a**), plan view (**Fig. 1b**) and side cross-section (**Fig. 2a**) of the Amherst wall. As observed, two rows of nails were installed. Both rows had a total of 5 nails separated 1.5 m from each other. The first row of nails was located 1.2 m from the top of the slope. The second row was located 1.5 m from the first row (so at a depth of 2.7 m). Nails consisted of 19-mm-diameter steel bars of 414 MPa of yield stress, placed with an inclination of 20° below horizontal. The shotcrete facing was 100 mm thick and 100 mm x 100 mm welded wire fabric was used as reinforcement. Besides, some instrumentation was installed in the Amherst wall, especially a total of 7 vertical inclinometers reaching a depth of 9.1 m and which were used to monitor lateral ground deformations. It is important to mention that no significant groundwater was observed during the excavation (Sheahan 2000).

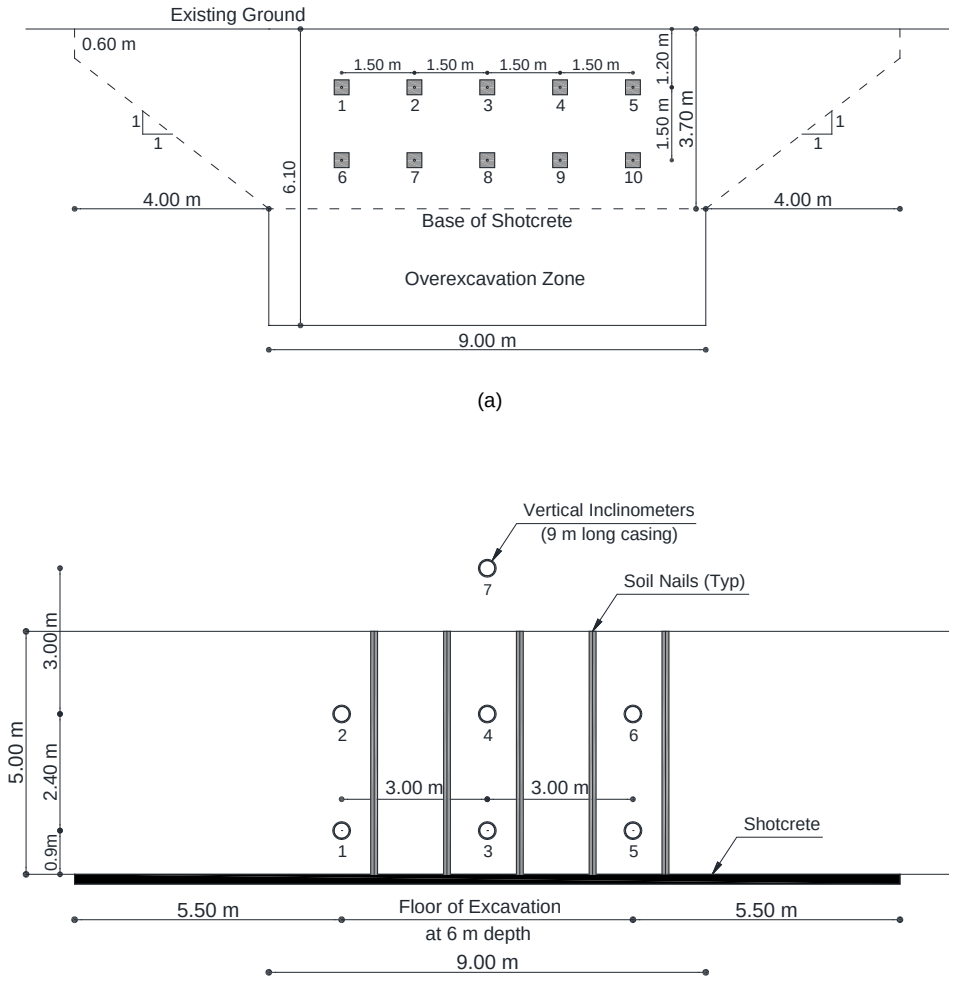
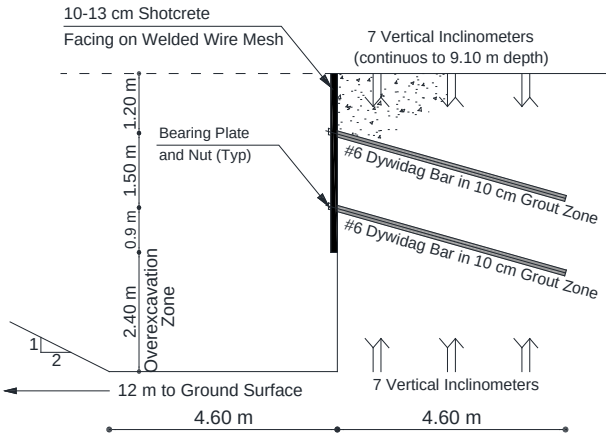
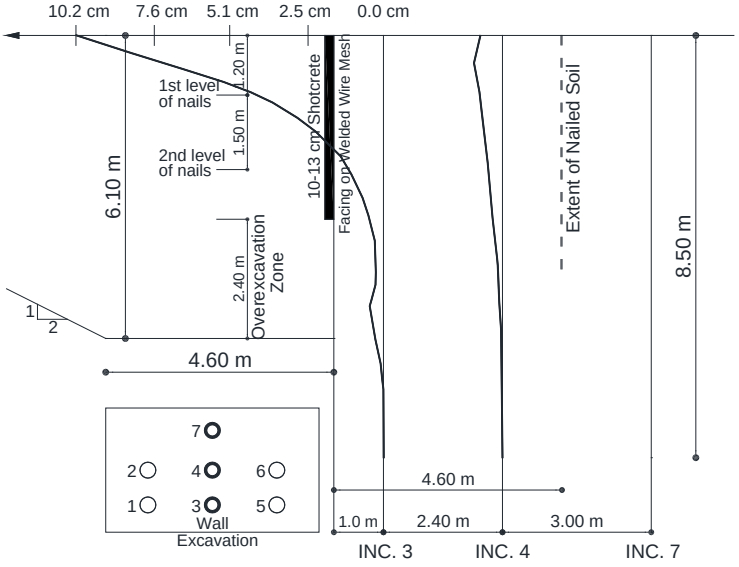



*Fig. 1. The Amherst wall front elevation (a) and plan view (b). Modified form Sheahan 2000.*




(a)




(b)

*Fig. 2. Amherst wall cross-section (a) and lateral deformation (b) recorded by the inclinometers placed at the middle section of the wall, i.e. inclinometers 3, 4 and 7. Modified form Sheahan 2000.*

*Geological-geotechnical characterization of the area*

The site is situated within the Connecticut River Valley (**Fig. 3a**), approximately 1.5 km within the shore of the ancient Glacial Lake Hitchock, the large pro-glacial lake system that occupied the length of the upper Connecticut River Valley in central New England (Stone and Ashley 1995; DeGroot and Lutenegger 2005).

A thick deposit of varved lake-bottom silts and clays (Ridge et al. 2012), locally referred to as Connecticut Valley Varved Clay (CVVC), was deposited on the floor of the glacial lake basin, meanwhile coarse deposits (i. e. sand and gravels) and meteoric-stream-fed deltas were graded to its shorelines. The postglacial deposits

partly overlie the Paleocene glaciolacustrine ones. They consist of stream-terraces, floodplain and swamp deposits, fluvial-estuarine channel-fills, eolian dunes, beaches, and marine delta deposits (Stone et al. 1998; Thorson et al. 2014). At present, the Connecticut River generally follows the course of the ancient Lake Hitchcock so, as expected modern fluvial deposits also cover the glaciolacustrine lake-bottom sediments (**Fig. 3b-c**).

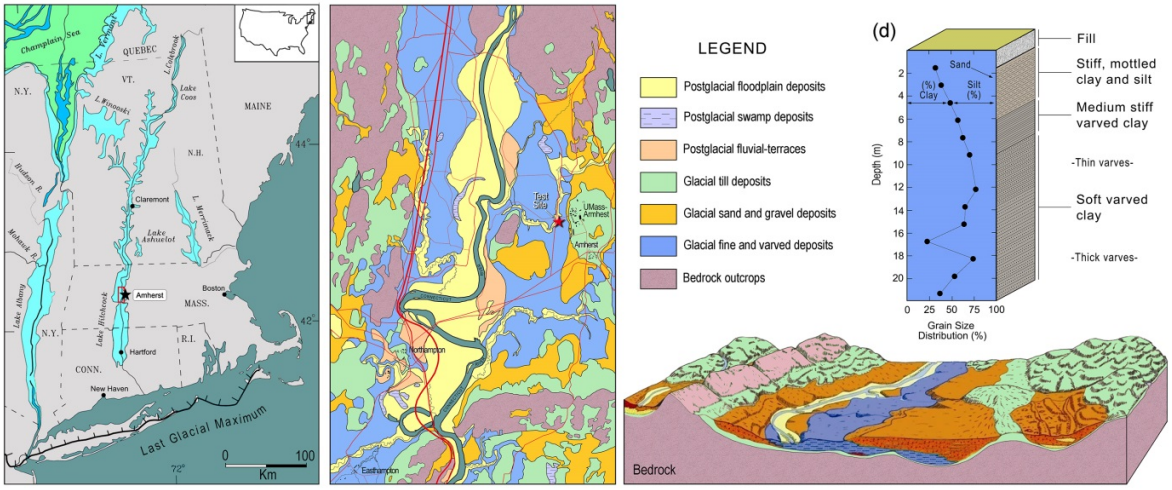



*Fig. 3. Geological setting of the Amherst test wall: (a) Location of Glacial Lake Hitchcock in New England and nearby contemporaneous glacial lake systems (rectangle shows the position of the figure 2b); (b) Simplified geological map of the Connecticut River Alluvial Lowland showing the location of the UMass Armhest NGES (star). Block diagram illustrating the distribution of glacial and postglacial deposits overlying bedrock in the Connecticut River Valley; (d) General soil profile and grain distribution at the UMass Amherst NGES site ((a), (c) and (d) modified from Ridge and Larsen, 1990; Stone et al., 1998; and Woods, 1995; DeGroot and Lutenegger, 1995, respectively).*

The soil stratigraphy at the UMass Amherst NGES test site is well-documented. Cores drilled to bedrock in the clayey soils reached a maximum depth of exploration of about 24.4 m (Woods 1995). However, nearby borings have yielded a 33 m-thick sequence. In both cases, samples are mainly dominated by the lake-bottom varved deposits of the former Glacial Lake Hitchcock (Brigham-Grette et al. 2000).

The sedimentary sequence at the test site begins with an uppermost compacted crust of miscellaneous fine-grained and granular fill, altered by soil development, extending from the surface to about 1.2 to 1.5 m. The fill consist of CVVC placed about 30 years ago after excavations at the Amherst Wastewater Treatment plant, which is adjacent to the site (Beim and Luna 2012). At 1.5 m depth, there is a gradational transition

from the abovementioned stiff silty-clay fill to a new layer of thin (< 1 cm), poorly defined varved clays. This transitional unit (about 20 cm thick) commonly also contains thin (1-2 mm) discontinuous, interbedded clay sheets. These upper horizons are underlain by more than 30 m of CVVC. The upper 5 to 6 m of this deposit is overconsolidated. Below the varved silt and clays are lightly to normally consolidated, increasing consolidation with depth (DeGroot and Lutenegger 1994). **Fig. 3d** shows a simplified vertical profile of the site.

A vertical anisotropy of the varved deposit have undergone significant changes as a result of pedogenic alteration, human activity and weathering. This zone, commonly known as a crust, extends to about 5 to 6 m below ground surface, near the maximum depth of the excavation of the Armhest test wall. According to Sheahan (2000), in that area plasticity index varies from 17 to 22% while undrained strength ranges from 80 kPa to less than 40 kPa.

The consistence properties of the varved clays also varies seasonally in the upper portion of the profile: the ground water table at the UMass Amherst NGES test site usually occurs in the upper 2-3 meters below ground surface, but varies as much as 1 to 2 m throughout the year according with seasonal groundwater recharge (DeGroot and Lutenegger 1994, 2005).

*Failing of the wall and lateral deformation*

Failure of the Amherst wall due to the overexcavation occurred after overexcavating 2.4 m, i.e. when a total depth of 6 m was attained. Global failure took place during the night of September 7th, apparently initiated by soil sloughing from the overexcavation face (Sheahan 2000). **Fig. 2b** displays the lateral deformation recorded by those inclinometers placed at the middle section of the wall (inclinometers 3, 4 and 7 in **Fig. 1b**) at failure.

## Numerical study

*General approach*

The analysis of the geological-geotechnical characteristics of the soil where the Amherst wall was built revealed a very heterogenic soil, formed by sand, silt and clay, with different proportion of each component at different depths. Besides, geotechnical parameters were also very variable, specially undrained shear strength which ranged from 80 kPa at the upper layers to 40 kPa or less at the lower layers.

Nevertheless previous analytical studies based on limit equilibrium methods (Sheahan 2000; Sheahan and Ho 2003) considered that soil to be a uniform undrained cohesive material. However, when this kind of material is tried to model by the finite element method (see **Fig. 4**), it is easy to observe that deformations of the nailed wall are completely different as in reality (rotation of the wall by its upper part occurred in a numerical model instead of rotation by its lower part seen experimentally).

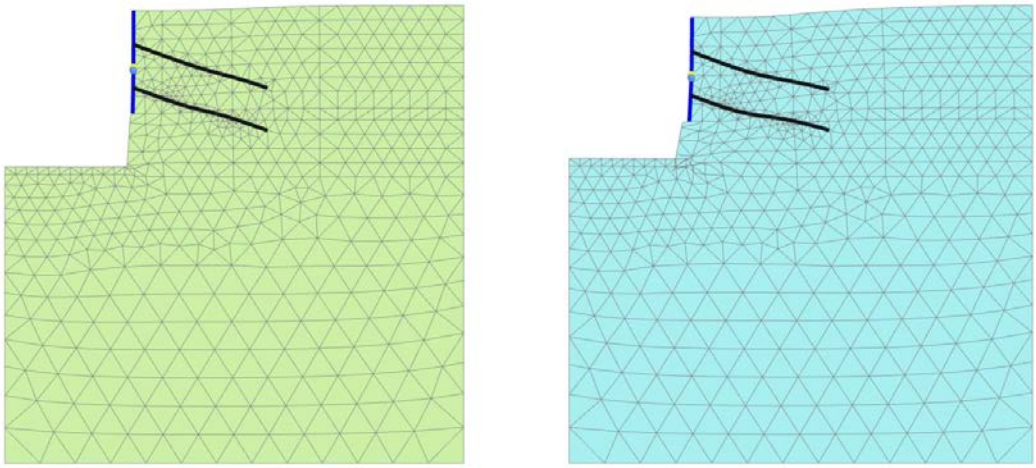



*Fig. 4. Deformed shapes of the Amherst wall obtained by a finite element simulation considering a unique undrained cohesive soil (approach followed previously by Sheahan 2000 and Sheahan and Ho, 2003). Soil was modelled using the Mohr-Coulomb model (a) and the Hardening Soil model (b). It is observed that in both cases rotation of the wall occurs by its upper part instead of rotation by its lower part, as seen experimentally. The finite element simulations where conducted following the same approach as that described in this paper.*

Hence, a back-analysis was conducted in this paper in order to reproduce as close as possible the behavior observed in the Amherst wall. Both a finite element model and a limit equilibrium method model were developed. Due to the natural variation observed in the ground characteristics at the depths where the wall was built, the ground was numerically stratified each meter until reaching 5 m (i.e. one meter before the total excavation). From that point, a uniform material was considered. In both cases, nails and shotcrete were placed at the same positions as they were located in the experimental test and a uniform value of 18.9 kN/m$^3$ was taken as unit weight of the soil. Water table did not appear during the excavations performed. Thus, no water table was introduced in the numerical models (i.e. pore water pressure was null).

*Finite element modelling*

The finite element commercial software Plaxis 2D v.2016 (PLAXIS 2016) was used to build a numerical model of the Amherst wall. A generic cross section of the wall was selected to be simulated in plane strain (Shiu et al. 2006; Fan and Luo 2008; Singh and Sivakumar Babu 2010). The model developed tried to simulate the Amherst wall behavior at failure, taking into account the construction process.

A total area of 16x16 m (see **Fig. 5a**) was modelled to avoid any disturbance in the results due to the proximity of the model boundaries to the area of interest. The nailed wall was located 4.6 m from the left border of the model, leaving the same excavation space as in reality (see **Fig. 2a and Fig. 5b**). More than 10 m of ground from the wall to the right border was left, space enough to introduce the nails and to study its influence. **Fig. 5** shows the numerical model developed.

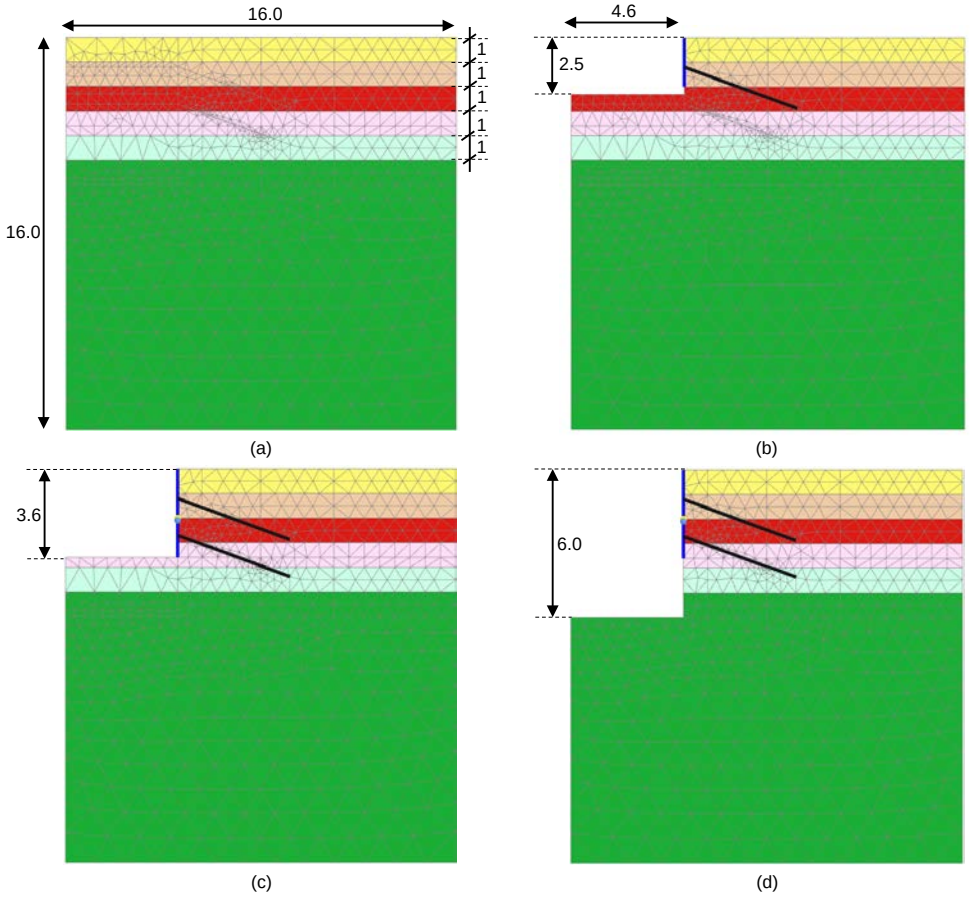



*Fig. 5. Finite element model developed for studying the Amherst wall (dimensions in m): (a) Model prior to any excavation; (b) Model after conducting the first excavation and installing the first row of nails (and shotcrete); (c) Model after conducting the second excavation and installing the second row of nails (and shotcrete); (d) Model after conducting the overexcavation.*

The finite element software Plaxis v.2016 allows defining different stages to simulate the construction process of an infrastructure. It is possible, in any stage, to remove ground clusters to reproduce an excavation or add structural elements such as the nails and the shotcrete face. Hence, to study the Amherst wall a total of six stages were considered: (i) initial state (no excavation takes place, ground is in its natural state); (ii) excavation of the first 2.5 m; (iii) installation of the first row of nails and building of the shotcrete face from the top until the bottom of the excavation; (iv) excavation until reaching a depth of 3.6 m; (v) installation of the second row of nails and building of the shotcrete face until the bottom of the new excavation; (vi) overexcavation until reaching a depth of 6 m.

Soil was meshed using 6-node triangular isoparametric elements with three Gauss points for each element, while nails and shotcrete were introduced as plate elements, with flexural rigidity and normal stiffness, and meshed with 3-node elements, with three degrees of freedom per node (two translational and one rotational). A connection element was introduced between the shotcrete casted before and after the construction stage (v), which allowed free rotation (no bending moment resisted). That was conducted trying to reproduce the clear change of deformation seen experimentally (see **Fig. 2b**) in the nailed wall.

Both the nails and the shotcrete were modelled using an elastic behavior. Mechanical values used for nails and shotcrete were obtained based on the material properties and the geometry of these elements, and are given in **Table 1**. Due to their discrete nature, nails were modelled using an ‘‘equivalent plate model’’ approach (Al-Hussaini and Johnson 1978; Unterreiner et al. 1997; Singh and Sivakumar Babu 2010), replacing them by a plate extended to one unit width. Particularly, normal stiffness (*EA*) and flexural rigidity (*EI*) of the nails were computed as:

$$EA = \frac{E_{eq}}{s_h} \cdot \left( \frac{\pi \cdot D_{dh}^2}{4} \right) \tag{1}$$

$$EI = \frac{E_{eq}}{s_h} \cdot \left( \frac{\pi \cdot D_{dh}^4}{64} \right) \tag{2}$$

where $s_h$ is the horizontal spacing of soil nails, $D_{dh}$ the diameter of the drill hole and $E_{eq}$ the equivalent elastic modulus of the grouted soil nail, which is given by:

$$E_{eq} = E_n \cdot \left( \frac{A_n}{A} \right) + E_g \cdot \left( \frac{A_g}{A} \right) \tag{3}$$

with:

$$A = 0.25 \cdot \pi \cdot D_{dh}^2 \quad (4)$$

$$A_n = 0.25 \cdot \pi \cdot d^2 \quad (5)$$

being $A$ the total cross-section area of grouted soil nail, given by the diameter of the drill hole, $D_{dh}$; $E_n$ and $A_n$ the elastic modulus and cross-section area of the steel nails (with $d$ the diameter of the steel bar), respectively; and $E_g$ and $A_g$ the elastic modulus and area of the grout material (note that $A_g = A - A_n$).

*Table 1. Mechanical characteristics of the shotcrete and nails as plate elements.*

| **Element** | ***Weight*** **(kN/m / m)** | ***E*** **(GPa)** | ***EA*** **[1,2]** **(kN / m)** | ***EI*** **[1,2]** **(kN·m² / m)** | ***ν*** |
|---|---|---|---|---|---|
| Shotcrete | 2.70 | 34.5 | $3.45 \cdot 10^6$ | 2875.0 | 0.2 |
| Nails | 0.13 | 200 | $117.9 \cdot 10^3$ | 76.67 | 0.3 |

*[1] Values of EA and EI for shotcrete were computed as:*

$$EA = E_c \cdot t_s \; ; EI = E_c \cdot \frac{t_s^3}{12}$$

*Being $E_c$ the elastic modulus of the shotcrete and $t_s$ the shotcrete thickness. Values for those parameters were taken based on the work of Sheahan (2000) and Sheahan and Ho (2003).*

*[2] Values of EA and EI for nail were computed as:*

$$EA = \frac{E_{eq}}{s_h} \cdot \left( \frac{\pi \cdot D_{dh}^2}{4} \right); EI = \frac{E_{eq}}{s_h} \cdot \left( \frac{\pi \cdot D_{dh}^4}{64} \right) \; with \; E_{eq} = E_n \cdot \left( \frac{A_n}{A} \right) + E_g \cdot \left( \frac{A_g}{A} \right); A = 0.25 \cdot \pi \cdot D_{dh}^2 \; ; A_n = 0.25 \cdot \pi \cdot d^2$$

*Being $s_h$ the horizontal spacing of soil nails, $D_{dh}$ the diameter of the drill hole, d the diameter of the steel bar (nail) and $E_n$ and $E_g$ the elastic modulus of the nail and the grout material that fills the drill hole, respectively. Values for those parameters were taken based on the work of Sheahan (2000) and Sheahan and Ho (2003).*

Regarding the soil elements, two possible mechanical behaviors were taken into account in the modelling process: the well-known Mohr-Coulomb model (Kim et al. 1997; Smith and Su 1997; Zhang et al. 1999; Sivakumar Babu et al. 2002; Fan and Luo 2008) and the Hardening Soil model (Liew and Khoo 2006; Singh and Sivakumar Babu 2010). The former consists of an elastic part based on Hooke's law of isotropic elasticity (defined by the elastic modulus, $E$, and the Poisson ratio, $\nu$) and a plastic part based on the Mohr-Coulomb failure criterion, which depends on cohesion, $c$, and the angle of friction, $\phi$, and formulated in a non-associated plasticity framework (the plastic potential function also depends on the dilatancy angle, $\psi$). The latter is based on the observed hyperbolic relationship between the deviatoric stress and the axial deformation in a cohesive soil element, where the Mohr-Coulomb failure criterion (defined by $c$ and $\phi$) set the maximum deviatoric stress withstood by the soil element. In this model, rigidity is not constant but it

depends on the stress level, so the elastic modulus $E_{50}$ corresponding to the stiffness of the soil when deviatoric stress is half the maximum one is used. Additionally, the oedometric modulus, $E_{oed}$, is taken into account and rigidity of the soil under unloading-reloading is considered by the modulus $E_{ur}$. Relation between volumetric plastic deformation and shear plastic deformation is considered by the dilatancy angle, $\psi$. The normally consolidated earth pressure at rest coefficient $K_{0,nc}$ is the last parameter needed by the Hardening Soil model. More information about this model along its verification may be found in Schanz et al. (1999).

Both the Mohr-Coulomb model and the Hardening Soil model allow an undrained or drained behavior of the soil. The Mohr-Coulomb model represents the behavior of geotechnical materials in a general way and it is optimal to model granular material. Unfortunately, it does not include stress-dependency, stress-path dependency, strain dependent stiffness, or anisotropic stiffness, which may be a downside when modelling cohesive materials. Conversely, the Hardening Soil model is a more advanced model for the simulation of soil behavior and described the soil stiffness much more accurately. This is a good model for simulating cohesive materials but it involves a total of 11 parameters instead of the 6 parameters used in the Mohr-Coulomb model.

*Limit equilibrium methods*

The commercial software Slide v.7 (RocScience 2017) was used to perform a stability analysis of the Amherst wall by limit equilibrium methods. These methods consider only the static mechanical laws to define the stability of a soil slope, neglecting the deformation of the ground, and assuming that the shear strength of the soil is totally and simultaneously developed along the sliding surface. The most common way of applying limit equilibrium methods is by the method of slices, which divides the sliding mass into a number of vertical slices to solve the problem, assuming that failure of the soil is governed by the Mohr-Coulomb criterion, slices behave as rigid bodies and no stresses exist inside each slice.

Safety of the slope is defined as the ratio between the available shear strength in the sliding surface, $S_m$, and the needed one to keep a strict equilibrium of the sliding mass. That ratio is known as safety factor, $F$. Hence, since failure follows Mohr-Coulomb criterion, value of available shear strength on each slide is given by:

$$S_m = \frac{c \cdot l}{F} + N \cdot \frac{\tan \phi}{F} \quad (6)$$

being $c$ the material cohesion and $\phi$ its angle of friction, $l$ the length of the sliding surface and $N$ the total reaction orthogonal to the sliding surface. Behavior of the soil can be considered undrained or drained. In the first case, undrained material values are assumed, i.e. cohesion equal to the undrained shear strength and null angle of friction. In the second case, effective values for cohesion and angle of friction are considered and $N$ transforms into $N'$ to indicate effective total reaction.

The method of slices requires previously defining the sliding surface (normally assumed to be circular). Since that is unknown, normally a grid of centers is defined and the factor of safety is computed for each of the resulting circle and the slope safety factor will correspond with the minimum value. Moreover, the equation system obtained once the equilibrium is established on each slice is normally solved assuming different simplifications and hypotheses. The main ones typically used in Geotechnical Engineering are Bishop (1955) and Janbu (1954) methods. The former establishes the equilibrium of vertical forces and bending moments, assumes a horizontal resultant of the interslice forces and does not take into account interslice shear forces. The latter establishes the equilibrium of both horizontal and vertical forces, assumes a horizontal resultant of the interslice forces, and uses an empirical correction factor to account for interslice shear forces. Normally both methods are computed and the lower safety factor obtained is selected.

The described method can be easily extrapolated to a soil consisting of several horizontal materials just by adding the available shear strength developed by each material crossed by the sliding surface.

The model built for studying the slope stability of the Amherst wall by limit equilibrium methods is showed in **Fig. 6**. It should be noted that the model was geometrically similar to the one used in the finite element simulation. An area of 16x16 m was modelled, placing the nailed wall at 4.6 m from the left border of the model. Nails were introduced in the model as passive elements, with no tension force applying on them, and located at their real position, with an inclination of 20º and with an out-of-plane spacing equal to 1.5 m. Tensile capacity was set to 118 kN (equal to the maximum axial load of a bar of diameter 19 mm and yield stress 414 MPa) and bond strength was left as default value (15 kN/m). The shotcrete face was not modelled, as it is usual done when working with limit equilibrium methods, since its contribution to the stability of the

slope is normally negligible. As done in the case of the finite element simulation, the ground was stratified each meter until reaching 5 m and then considered uniform.

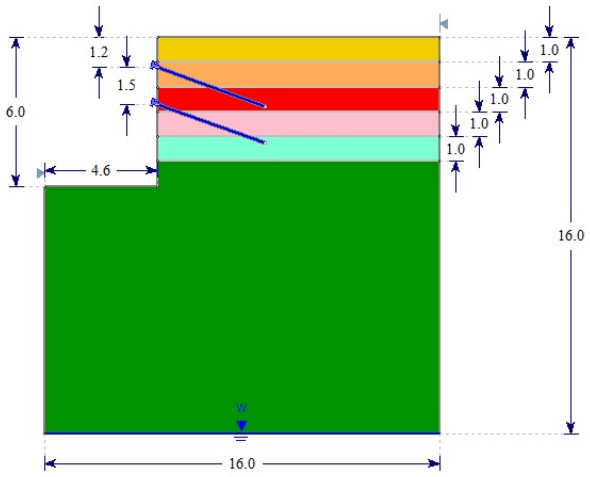



*Fig. 6. Limit equilibrium method model developed for studying the Amherst wall (dimensions in m).*

*Results*

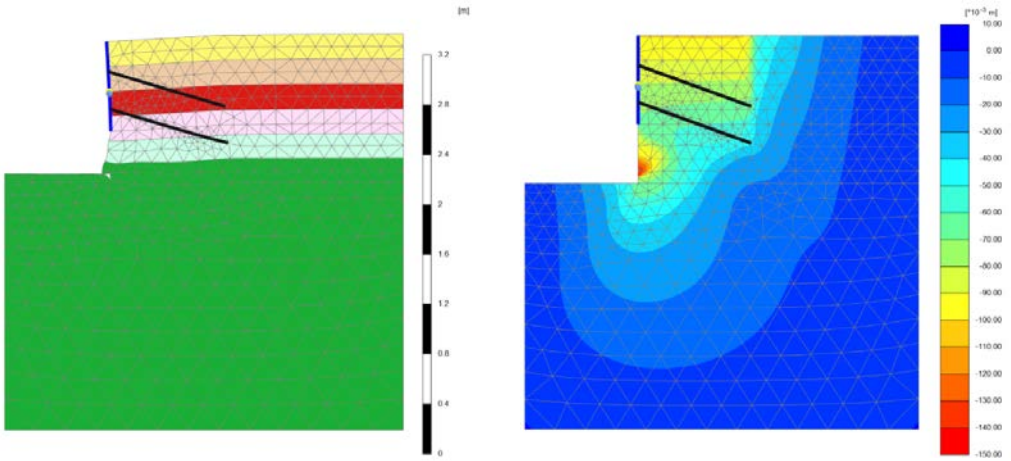
**Table 2** shows the mechanical characteristics of the different ground materials layers obtained after a trial and error process in the finite element simulation, so as to attain a behavior of the numerical nailed wall similar to the experimental one observed.

*Table 2. Mechanical characteristics of the ground materials.*

| **Stratum depth (m)** | **Mechanical model** | **Drain Behavior** | $s_u$ **(kPa)** | $c'$ **(kPa)** | $\phi'$ **(º)** | $\psi$ **(º)** | $E$ **(kPa)** | $\nu$ | $E_{50}$ **(kPa)** | $E_{ur}$ **(kPa)** | $E_{oed}$ **(kPa)** |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 – 1 | Mohr-Coulomb | Drained | - | 10 | 20 | 0 | 30000 | 0.33 | - | - | - |
| 1 – 2 | Mohr-Coulomb | Drained | - | 100 | 25 | 0 | 50000 | 0.33 | - | - | - |
| 2 – 3 | Mohr-Coulomb | Drained | - | 80 | 25 | 0 | 38000 | 0.33 | - | - | - |
| 3 – 4 | Mohr-Coulomb | Drained | - | 50 | 25 | 0 | 30000 | 0.33 | - | - | - |
| 4 – 5 | Hardening Soil | Undrained | 39 | - | - | 0 | - | - | 26000 | 52000 | 26000 |
| 5 – | Hardening Soil | Undrained | 27 | - | - | 0 | - | - | 16000 | 32000 | 16000 |

In order to ensure the failure of the slope in the finite element model, the overexcavation stage was divided into two stages: one stage where all but the last 0.25 m were excavated and a second stage where those last 0.25 m were excavated. Based on this, with the proposed geotechnical materials of **Table 2** a numerical instability of the solving procedure occurred when trying to excavate the final 0.25 m, not finding convergence and registering a negative total stiffness of the model. This means that failure of the wall took

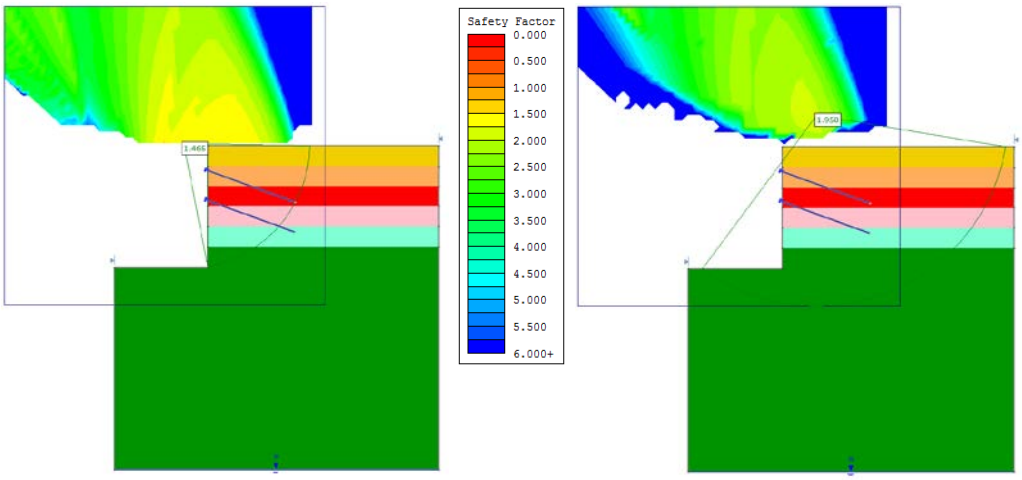
place. **Fig. 7** shows the deformed shape (**Fig. 7a**) and lateral displacements (**Fig. 7b**) obtained for this final stage. As can be seen, displacement of the wall follows the same behavior observed in the experimental test and the lateral displacements at the top of the model are about 10 cm, as recorded by the inclinometers experimentally.



(a) (b)

*Fig. 7. Numerical study of the Amherst wall by a finite element model: (a) deformed shape; (b) lateral displacements.*

The limit equilibrium method model developed was also run using the proposed geotechnical materials. **Fig. 8** shows safety factor obtained for both the Bishop (1955) and Janbu (1954) methods (**Figs. 8a and 8b**, respectively), being the lowest one that obtained by the Bishop method and equal to 1.47.




(a) (b)

*Fig. 8. Numerical study of the Amherst wall by limit equilibrium methods: (a) factor of safety based on Bishop method; (b) factor of safety based on Janbu method.*

## Analysis of the results and discussion

*Ground modelling*

Results obtained in the back analysis of the Amherst wall showed that numerically the ground can be separated in two different zones: an upper zone, corresponding to the first 4 m, and a lower one, which extends below, from a depth of 4 m.

The upper zone contains the first four ground numerical strata, and approximately corresponds to the area where the shotcrete face existed. In this zone ground materials were modelled as drained materials working according to the Mohr-Coulomb mechanical model. This drained behavior may be explained by two main aspects: (a) the experimental displacement observed on the wall, which rotated by its lower point towards the excavation, a typical performance of granular materials; and (b) the geotechnical characteristics of the upper layers of the soil according to the geotechnical investigation of the area which is composed by a mix of sand, silt and clay, with a slightly high OCR. Besides, it should be mentioned that since geological-geotechnical profile indicated that surface material corresponded to a fill, mechanical characteristics of the first numerical layer were selected based on typical values of a fill (Sánchez-Alciturri et al. 1993).

The lower zone comprises the last two ground strata, and corresponds to the area where the overexcavation was conducted. In this zone ground materials were modelled as undrained materials working according to the Hardening Soil mechanical model. This numerical approach considers that materials in this zone behave as cohesive ones, which is consistent with the geological-geotechnical characterization of the Amherst wall area, composed by a mix of silt and clay from a depth of around 4 m. Besides, since excavation was carried out fast in terms of time (no more than 15 days passed between the start of excavation of the top part of the wall and the failure of the nailed wall due to the overexcavation) an undrained behavior may be expected. It should be clarified that materials in the upper zone (modelled by the Mohr-Coulomb mechanical model) may also be working under an undrained behavior according to this, but since those materials were found to numerically behave as granular materials, a drained behavior was selected.

Having found a better performance of the use of the Hardening Soil mechanical model to simulate the cohesive soil strata of the ground which are located on the bottom of the excavation, is in accordance with the work of Singh and Sivakumar Babu (2010) who compared the use of the Mohr-Coulomb and the Hardening Soil models in simulating a soil nailed wall and observed that the former overestimated (Callisto

et al. 1999; Brinkgreve et al. 2006) the base heave of the excavation face to almost twice as that predicted by the latter. This phenomenon is of importance, since, as stated by the FHWA (2003), basal heave may be a failure mode of soil nailed walls caused by the unbalanced forces that appear on the bottom of the excavation which may result in a bearing capacity failure of the ground.

All the aforementioned concerns regarding the convenience of the use of a certain mechanical model for defining the geotechnical materials and thus simulating as close as possible the real behavior of the nailed wall are the keystone of a good finite element model. However, it is interesting to note that in terms of a design based on the limit equilibrium method, those aspects are of a lower importance. As showed above, the objective of limit equilibrium methods is computing the safety factor, which only involves those forces that contribute to stabilize the system and those that tend to make it unstable.

This is a great advantage of the limit equilibrium methods, since (aside parameters regarding to the nails) only ground strength properties are needed, i.e. angle of friction and cohesion (or undrained shear strength when simulating an undrained cohesive soil). That simplifies greatly the problem and allows a fast and efficient designing of soil nailed walls, avoiding the necessity of using geotechnical properties like the elastic moduli of the ground materials, which are normally difficult to obtain. Nevertheless, one should keep in mind that the real behavior of the soil and the wall is neglected and that the whole performance of a nailed wall is reduced to a unique number (the safety factor), which may be problematic in some cases, especially when excessive ground deformations are developed.

*Load distribution on nails*

An important aspect when designing a soil nailing wall is related with the load distribution on nails. As seen in the literature (Fan and Luo 2008; Wei and Cheng 2010), nails located in the lower part of a soil slope have a considerable influence on the overall stability of the system, and consequently they tend to absorb more loads. In those lower rows, the highest values of shear forces and bending moments are normally located at sections close to the nails ends.

Those issues were captured by the numerical simulation developed by the finite element model. **Fig. 9** shows load distribution on nails at the moment of failure of the soil nailing wall. As can be observed, axial loads clearly reached higher values in the lower row of nails: a value about 22.5 kN/m was obtained in the upper row (**Fig. 9a**) while more than three times, nearly 69.0 kN/m, was obtained in the lower row (**Fig. 9b**). A

similar phenomenon was observed for the shear loads, with a value of about 0.65 kN/m recorded as maximum shear load in the upper row (**Fig. 9c**) and approximately a value of 5.5 kN/m in the lower row (**Fig. 9d**). Finally in terms of bending moments, maximum value in the upper row was about 0.14 kN·m/m (**Fig. 9e**), and 1.75 kN·m/m in the lower row (**Fig. 9f**). Thus, higher loads were developed in the lower row of nails, and in that row the load distribution graphs show that, as expected, highest values of shear forces and bending moments occur at a great distance from the wall (in the vicinity of the end of the nails).

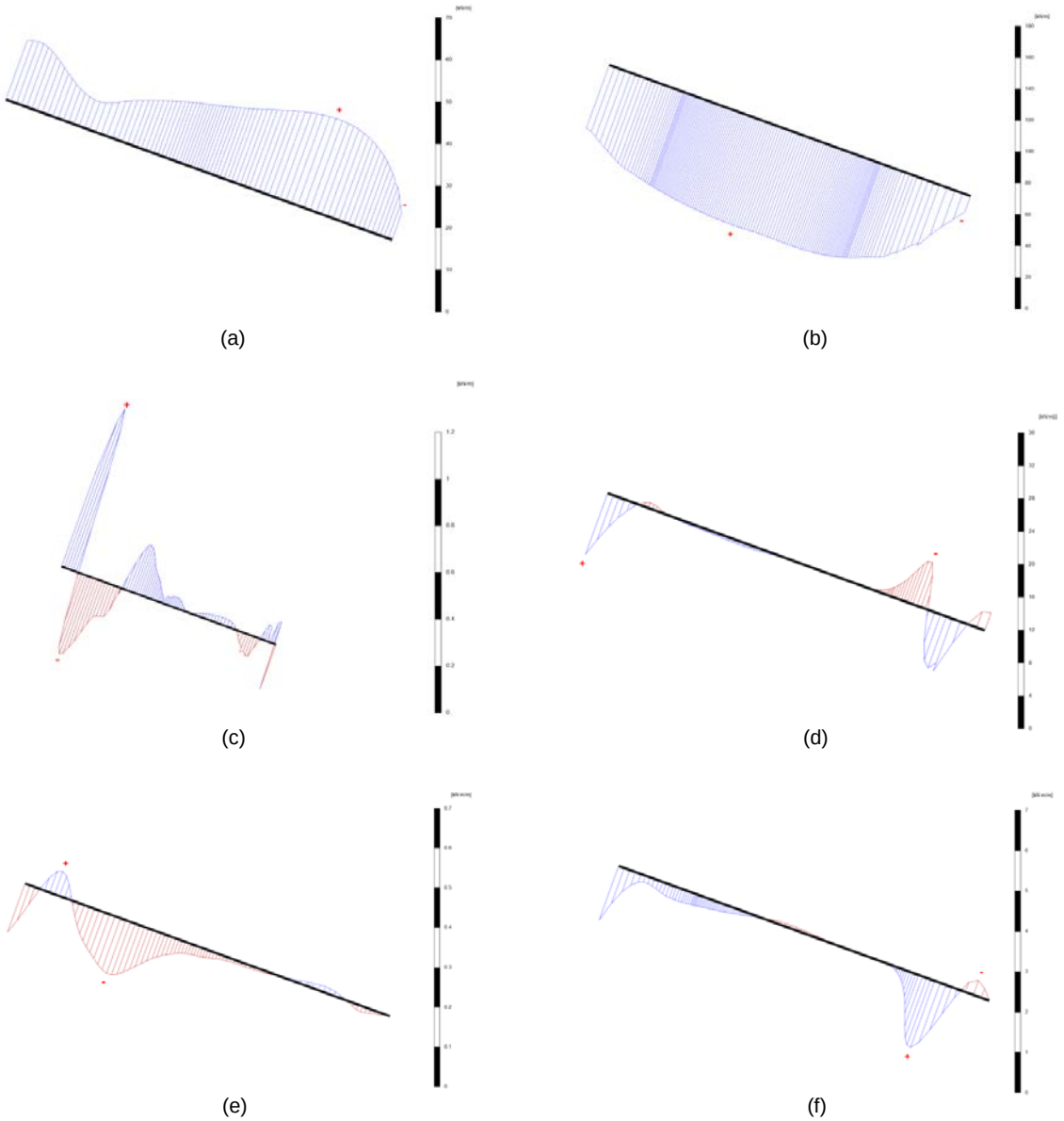



*Fig. 9. Load distribution on the nails obtained by the finite element model developed in the moment of failure: (a) axial load in the upper row of nails; (b) axial load in the lower row of nails; (c) shear force in the upper row of nails; (d) shear force in the lower row of nails; (e) bending moment in the upper row of nails; (f) bending moment in the lower row of nails.*

When using limit equilibrium methods, maximum tension within nails is normally considered to be at the intersection of such elements with the failure surface (Wei and Cheng 2010). Even though this is a clear simplification, it is interesting to observe that the potential slip surface defined by limit equilibrium methods run in this work (see **Fig. 8**) crosses the lower row of nails near their end. Besides, the axial load distribution obtained for those nails (**Fig. 9b**) shows that axial load tend to achieve a certain value along a part of the nail and that value is kept constant until reaching the vicinity where the highest values of shear forces and bending moments appear (near the end of the nails).

Thus, the use of the classical approach of many practitioners, who assume that the maximum tensile force line matches the potential sliding surface obtained by limit equilibrium methods, may be used to estimate the loads of nails. Those elements can therefore be designed considering that the mass of soil inside of the sliding surface (close to the wall) tends to pull the nail out of the ground, while the mass out of the sliding surface tends to restrain the nail from being pulled out (FHWA 1996; Wei and Cheng 2010).

*Failure and ground deformations*

Strictly speaking, if the safety factor obtained by a limit equilibrium method is above 1.0 the slope may be considered not to be in a failure state. The lowest safety factor obtained in the Amherst wall model was 1.47, so, at first one may think that the system is in a safe situation. However, as the finite element model demonstrated, the system is in a failure condition.

**Figs. 10a** and **10b** display tension cut-off points and failure points given by the finite element model developed at the step when no convergence of the model was attained and a negative total stiffness was obtained, i.e. when failure was considered to occur. As can be observed, failure points occur mainly at the undrained cohesive materials located in the lower zone of the soil nailed wall, while tension cut-off points are generally located in the upper zone of the wall and to a certain distance to it, being a consequence of the lateral movement experimented by the wall (which rotates by its lower point towards the excavation).

Hence, assuming that in a soil nailed wall a safety factor equal to 1.0 means no failure may be not correct. In fact, in slope stability, due to the own nature of the calculation methods (especially the limit equilibrium methods) and the reliability of the geotechnical parameters, according to geotechnical codes, a safety factor lower than 1.2 - 1.5 (the value may depend on issues such as the permanent or temporary conditions of the work and the contemplation of the seismic action) should be considered an unsafe situation. Soil nailing can

be seen as a particular case of slope stability, so taking into the results obtained in the numeric simulation undertaken, a safety factor of at least 1.5 should be ensured when using limit equilibrium methods to design a soil nailed wall.

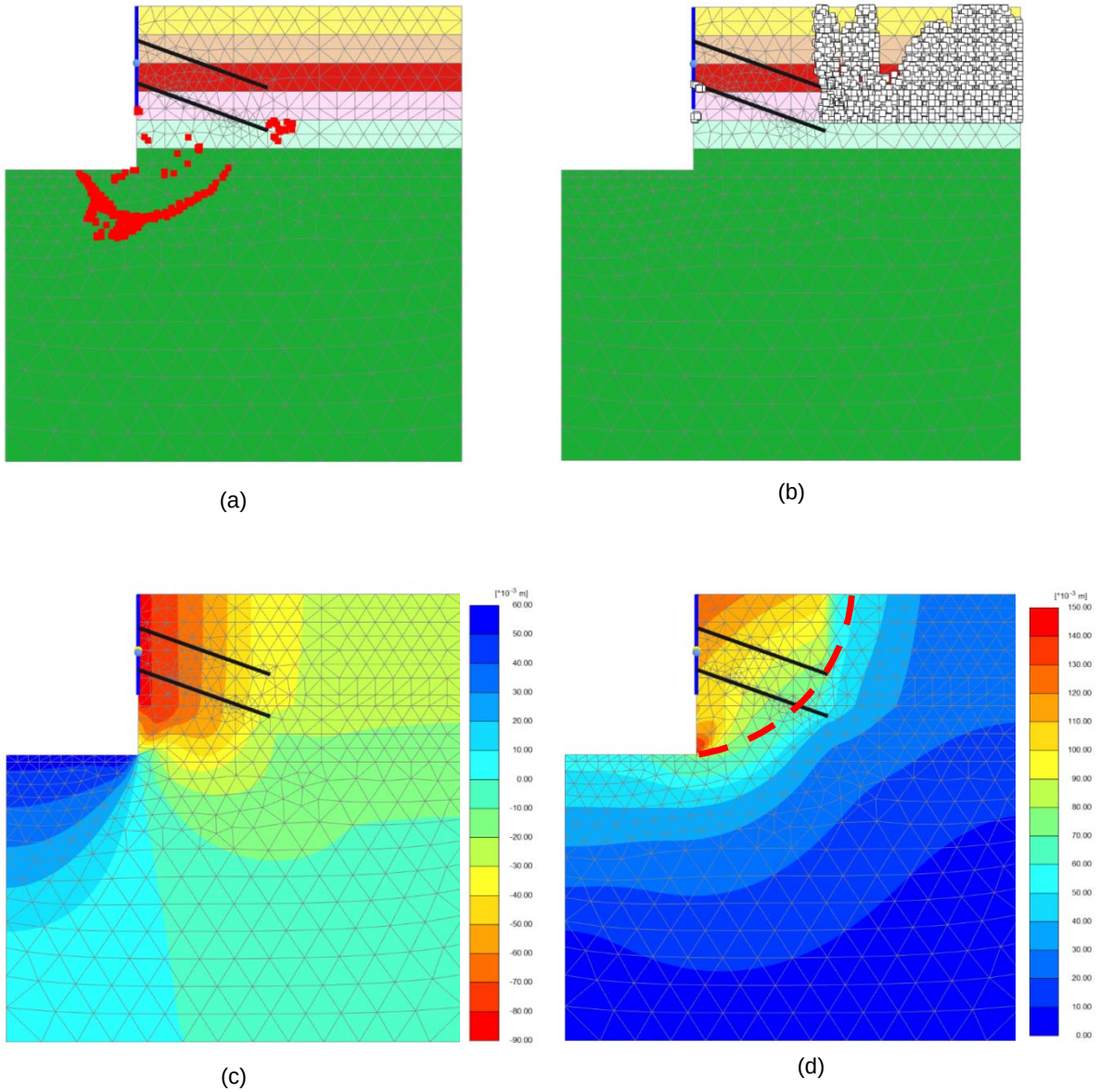



*Fig. 10. Finite element model results: (a) Tension cut-off points; (b) Failure points; (c) vertical ground deformations; (d) total ground deformations (depicted in red Bishop's failure circle according to the limit equilibrium method developed).*

Besides mentioned issues concerning failure of the system, as was indicated above, limit equilibrium methods neglect deformations, which may be problematic if excessive ground deformations are developed. **Fig. 10c** shows vertical deformations of the ground obtained in the finite element model developed prior to failure. Values exhibited in the vicinity of the excavation are larger than one inch (2.54 cm), the value

typically considered to be the maximum settlement that does not induce any deformation problems to infrastructures and buildings. **Fig. 10d** displays ground total deformations. Values of more than 10 cm are recorded, which likely may lead to compromise the own stability of the excavation.

Thus, both graphs demonstrate that in the case under study excessive ground deformations take place, which can never be detected by just using limit equilibrium methods. However, it should be noted that limit equilibrium methods are able of capturing with relative approximation the failure surface (see **Fig. 10d**). That, together with the advantages showed above for those methods (easy to build models, not many geotechnical properties involved, fast and simple estimation of nail forces) make limit equilibrium methods a very useful tool to design soil nailed walls. However, in a second phase, once the main parameters of the design are defined (such as excavation phases and nail strength properties) a finite element model should be developed in order to verify the good performance of the soil nailed wall and especially to check the possibility of having excessive ground deformations.

## Conclusion

This paper has tried to establish the convenience of the applicability of limit equilibrium methods and finite element models for designing soil nailed walls, considering both the suitability of an easy and fast design process, and the necessity to take into account issues such as ground deformations to avoid problematic consequences that can arise during their execution phase and service life.

For doing this, a numerical study of the Amherst wall was performed and subsequently analyzed. The Amherst wall is a full-scale soil nailed wall built as an experimental test at the Amherst National Geotechnical Experimentation Site, near the town of Amherst, Massachusetts, and it intentionally failed due to an overexcavation.

A finite element model and a limit equilibrium model (using the method of the slices) were developed. Nails and shotcrete were placed at the same positions as where located in the experimental test and, as a consequence of the variability of the ground characteristics at the depths where the wall was built, the soil was numerically stratified each meter until reaching one meter before the total excavation. A back-analysis procedure was used to define the mechanical properties of each ground stratum, validating the values with the behavior of the Amherst wall experimentally observed.

In the numerical model, the ground is divided into an upper zone, corresponding approximately to the area where the shotcrete face existed, and a lower one, where the overexcavation was conducted. The upper zone was found to behave similar to a granular material, causing the nailed wall to rotate by its lower end and producing the highest value of lateral deformation at its top part, while the lower zone was found to behave as an undrained cohesive material. Thus, this paper has also shown an approach to numerically model soil-nailed walls in ground situations where the soil is neither pure cohesive nor pure granular, but a mix of them. Materials more similar to granular soils have demonstrated to behave according to a Mohr-Coulomb model (with drained analysis), while materials more similar to cohesive soils are better reproduced using the Hardening Soil model (with undrained analysis).

Limit equilibrium methods have showed to be fast and efficient when designing soil nailed walls, presenting several advantages of their use. The main one may probably be the fact that they are focused on computing the safety factor, which involves only forces, simplifying greatly the problem and avoiding the necessity of using geotechnical properties difficult to obtain such as the elastic moduli of the ground materials (for limit equilibrium methods only ground strength properties, i.e. angle of friction and cohesion are needed). Moreover, they allow a good estimation of the tensile stresses to which the nails are subjected by assuming that the maximum tensile force line matches the potential sliding surface obtained by those methods.

Design of a soil nailed wall by limit equilibrium methods should be addressed with some caution, however. Firstly they do not take into account ground deformations, so they do not detect the development of excessive ground deformations. Secondly a safety factor of 1.0 or a little higher may not be a reliable indicator of a good performance of the wall. It is recommendable to reach at least a safety factor of 1.5 in order to consider that the system designed is in a safe situation.

On the other hand finite element methods allow a good determination of both performance of the nailed wall and development of ground deformations. They also give more information about stresses developed in the nails, not limiting the output value to axial loads, but delivering the value of shear loads and bending moments that can appear in those elements. Nevertheless finite element methods require more parameters than limit equilibrium methods to be properly defined (both strength and deformability properties of ground materials are needed) as well as the selection of the appropriate mechanical model (e.g. Mohr-Coulomb or Hardening Soil) to correctly simulate the soil behavior. That results in more complex model, but if they are

adequately built they will provide good information about the expected behavior of the soil both in the excavation and its surroundings.

After having conducting the analysis and comparison of both limit equilibrium methods and finite element methods, a designing process for soil nailed walls based on two steps or phases is proposed. In a first phase limit equilibrium methods may be used to define the main parameters of the design such as the depth of excavations, the nail strength properties and a global safety factor fulfilment. Then, in a second phase, a finite element model should be developed to verify the good performance of designed system, to account for the real forces expected to be developed in the nails (axial loads, shear loads and bending moments) and to check the possibility of having excessive ground deformations that can give rise to problems in surrounding buildings and structures as well as endanger the safety of the own excavation.

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