# NUMERICAL ANALYSIS OF LATERALLY LOADED LIME/CEMENT COLUMNS

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**Abstract**

*Stabilization of soft clayey soils with lime/cement columns has become a common ground improvement method in Europe, USA and Japan. Yet, the current design methods for stability analyses show limitations, in that they only consider shear failure of the columns. Previous experimental and theoretical studies have showed that failure by bending is more probable. In this work shear-box tests on reinforced clay are numerically reproduced with non-linear finite element modeling. A concrete material model is used for lime/cement, which makes it possible to observe cracks within the columns. The results obtained show good agreement with the shear-box experiments, and confirm the general conclusions regarding failure of the columns.*

**Key words**

Deep mixing soil stabilization, finite element method, lime/cement, clay, damaged plasticity, shear-box test

## 1 INTRODUCTION

Deep stabilization using lime/cement columns is the most common ground improvement method in Sweden. They are frequently used in the construction of roads and railway embankments. Their main effects are to accelerate construction by reducing the consolidation times, decrease settlements and improve embankment stability. It is often an economical solution compared with other soil improvements methods such as excavation and replacement and embankment piles.

Design codes in Sweden assume that the columns fully interact with the unstabilized soil between them, that is, the reinforced soil behaves like a composite material ([9], [10]) with weighted average undrained shear strength. The columns are assumed to fail in shear. However, several studies have showed that failure in bending is more probable, which means that the strength of the embankment might be overestimated ([2], [3]).

Shear box tests have been conducted by the Swedish Deep Stabilization Research Centre ([4], [5], [6]) on lime/cement columns stabilized kaolin clay, with several configurations (isolated columns, shear walls, arching columns). The results showed that bending failure occurred and that the shear resistance was lower than indicated by the design codes. The purpose of this work is to numerically reproduce some of those tests with the non-linear finite element method, in order to study more closely the failure pattern. It was carried out at the Royal Institute of Technology of Stockholm (Kungliga Tekniska Högskolan).

## 2 METHODS AND MATERIALS

### 2.1 Shear box tests

The preparation of kaolin, the installation of lime/cement columns, the test setup and the results are described exhaustively in [5].

Dry kaolin clay powder was mixed with tap water to obtain uniform slurry, and was then consolidated vertically using pressurized air. The consolidation pressure was increased in stages up to 60 kPa. The properties of the soil obtained are gathered in Table 1.

*Tab. 1 Properties of the consolidated kaolin clay.*

| Property | Value |
|---|---|
| Specific gravity [$t/m^3$] | 2.6 |
| Water content [%] | 48-51 |
| Liquid limit [%] | 54 |
| Plastic limit [%] | 30 |
| Coefficient of consolidation [$m^2/s$] | $3.10^{-7}$-$10.10^{-7}$ |
| Shear strength [kPa] | 4-6 (drop cone test) |

Lime/cement was installed in the clay with the dry method. The composition of the binder was 30% lime and 70% cement, and the quantity was 150 $kg/m^3$. After the columns were installed, a 50 mm thick layer of clay at the top was replaced by sand. A normal pressure of 15 kPa was applied after one day. The shearing tests were performed two weeks after the installation of the lime/cement columns. The shear box is showed in Figure 1.

The lower part of the steel barrel was fixed while the upper part underwent a traction force. For each test the load – horizontal displacement curve was recorded.

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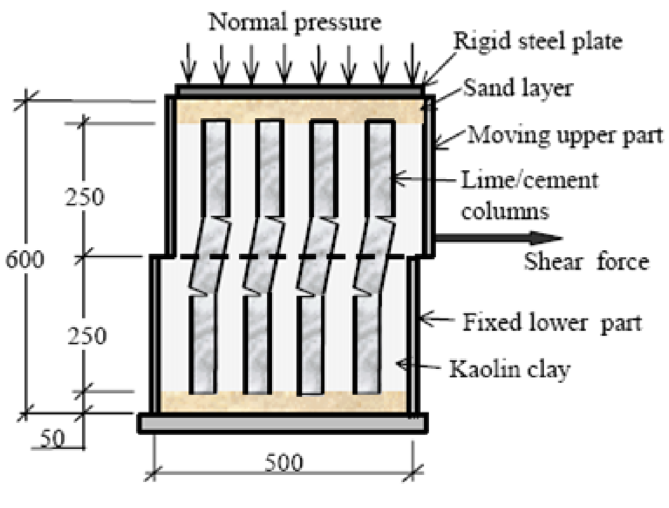


*Fig.1 Schematic of the shear box test. From [6].*

Some of the most important observations were that:

- The clay flows around the columns.
- Plastic hinges appear for both single columns and shear walls, approximately 10 cm apart. This corresponds to a bending failure, that is, the columns fail when the moment capacity is exceeded.
- The cracks resemble those in concrete.

### 2.2 Material modeling

The numerical analysis were carried out with the finite element program ABAQUS ([1]). Two configurations were reproduced – unstabilized soil and 12 isolated columns. The clay was modeled as a Mohr-Coulomb elasto-plastic material. As the pore water was not simulated, the clay was simulated in an undrained manner, that is, the angle of internal friction was taken as equal to 0°. The cohesion was measured from the shear box experiments as equal to 4 kPa. The linear elastic properties of the clay – Young's modulus and Poisson's ratio – were investigated through numerical analysis on unreinforced soil.

As the cracks in the columns were similar to those in concrete, the lime/cement improved kaolin columns were modeled using a recent material model – the concrete damaged plasticity model ([7], [8]). It is primarily intended for reinforced concrete but can also be used for quasi-brittle materials. The concrete damaged plasticity model in ABAQUS consists of a combination of non-associated multi-hardening plasticity and isotropic damaged elasticity.

The most important feature of this model is its ability to take cracks into account. This is done through an isotropic reduction of the elastic stiffness as the deformation goes on. For instance, the strain- stress relationship for uniaxial tension is:

$$\sigma_t = (1 - d_t)E_0(\varepsilon_t - \tilde{\varepsilon}_t^{pl}) \qquad (1)$$

where,

$\sigma_t$ is the uniaxial tensile stress (Pa)
$\varepsilon_t$ is the plastic strain in tension
$\tilde{\varepsilon}_t^{pl}$ is the equivalent plastic strain in tension,
$E_0$ is the initial (undamaged) elastic stiffness of the material, and
$d_t$ is the damage variable in tension.

This means that if the material is unloaded from any point on the strain softening branch, that is, when the tensile failure stress is exceeded, the elastic stiffness is reduced by the amount (1-$d_t$). The tensile parameter itself is a growing function of the deformation.

The uniaxial compressive behavior is defined in a similar way. The concrete damaged plasticity model also requires the definition of the uniaxial cyclic behavior, a yield function and a plastic potential. One important feature of the model is that, compared to the uniaxial case, the material is stronger under biaxial and triaxial compression.

As lime/cement is a much weaker material than concrete, many parameters had to be adjusted. The uniaxial compressive strength was obtained from unconfined compressive tests on the columns ([5]) and taken as 100 kPa. The other parameters were either kept at their default value, or lowered owing to the lower strength of lime/cement.

Density, Young's modulus and Poisson's ratio for the clay and lime/cement are gathered in Table 2.

*Tab. 2 Basic mechanical properties.*

| Material | Property. | Value |
|---|---|---|
| Clay | Density [kg/m$^3$] | 1500 |
| | Elastic modulus [MPa] | 0.5 - 1 |
| | Poisson's ratio | 0.45 - 0.48 |
| | | |
| Lime/cement | Density [kg/m$^3$] | 1500 |
| | Elastic modulus [MPa] | 20 |
| | Poisson's ratio | 0.15 |

### 2.3 Numerical analysis

The finite element model reproducing the shear box had three parts: the clay, the lime/cement columns and the sand layer. The steel barrel was simulated through boundary conditions. The contact between the soil and the lateral surface of the columns was defined with friction Coulomb model for tangential behavior (friction coefficient equal to 0.05), and the default "hard" contact for normal behavior (contact pressure between the surfaces is transmitted only if their nodes are in contact). The lower surfaces of the columns were tied to the bottom of the clay. A vertical pressure (15 kPa) was applied during the first step of the simulation. In the second step, an increasing displacement was applied to all the upper nodes of the shear box, while the lower nodes were restrained from any displacement.

## 3 RESULTS AND DISCUSSION

### 3.1 Finite element analyses on unstabilized soil

These analyses have been used to adjust Young's modulus and Poisson's ratio of the clay. They were varied in order to obtain the best fit with the experimental curves (see Figure 2). However, Young's modulus was finally chosen lower to account for actual decrease of stiffness when the clay undergoes deformation.

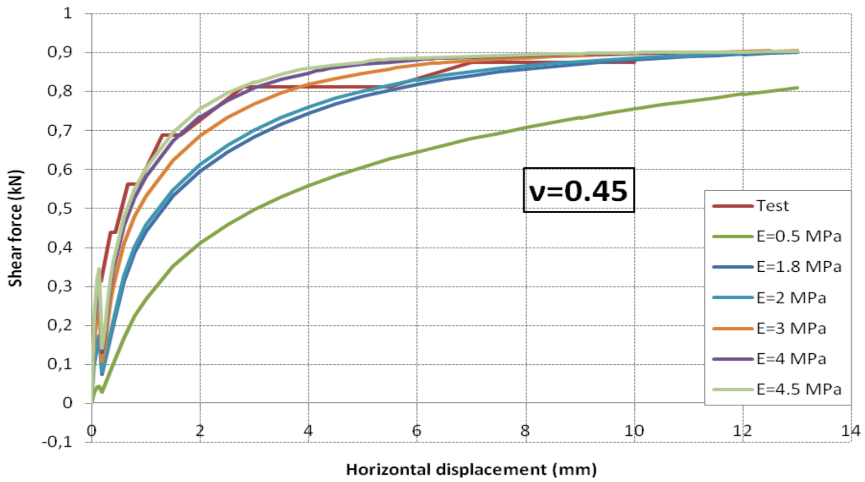


*Fig.2 Experimental and numerical load-displacement curves for unreinforced clay.*

### 3.2 Finite element analyses on stabilized soil

Numerical analyses were performed with 12 single columns, corresponding to an area replacement ratio of 12%. The horizontal displacement field after shearing is showed in Figure 3. It is seen that the columns have helped reduce the displacement of the soil between the columns.

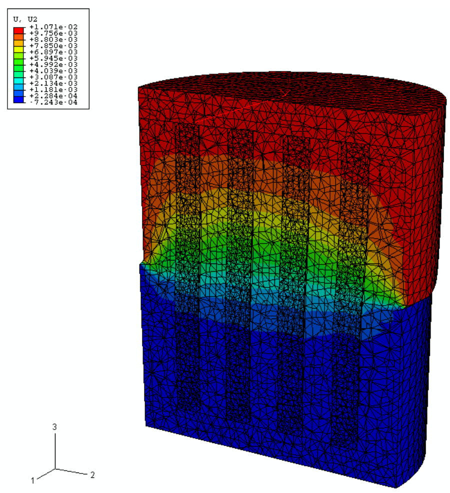

*Fig. 3 Horizontal displacement field of the stabilized soil after shearing.*

The experimental and numerical load-displacement curves are showed in Figure 4. The results obtained from the finite element analyses display good agreement with the experimental results. The ultimate load is about the same value – between 1 and 1.2 kN i.e. between 5 and 6 kPa, whereas the value according to the design codes is 11 kPa. The design method overestimates the strength of the reinforced soil.

Convergence problems occurred when the shearing displacement reached 7 - 8 mm. This is thought to be due to yielding of the materials, especially cracking and stiffness degradation of the lime/cement columns. High values of the tensile damage parameter $d_t$ correspond to a degradation of the stiffness, which indicates the formation of cracks. As pointed out in previous studies ([2], [3], [6]), two plastic hinges appeared below and above the shear plane, which means that the columns fail by bending (see Figure 5).

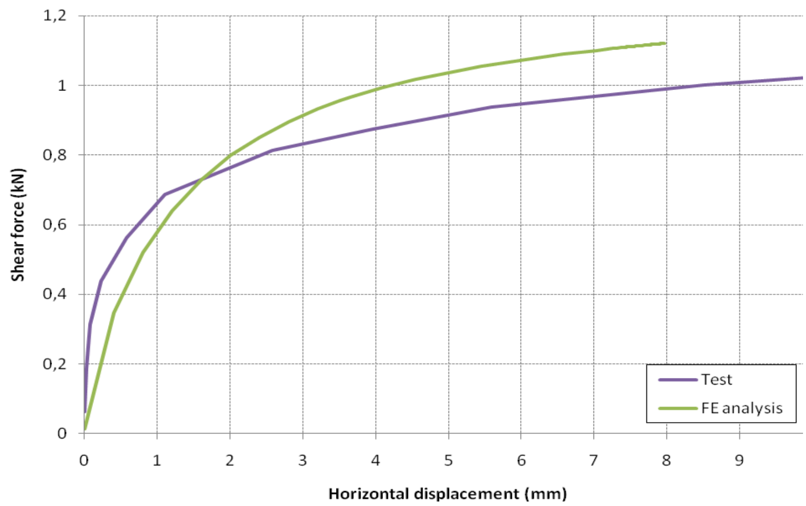


*Fig.4 Experimental and numerical load-displacement.*

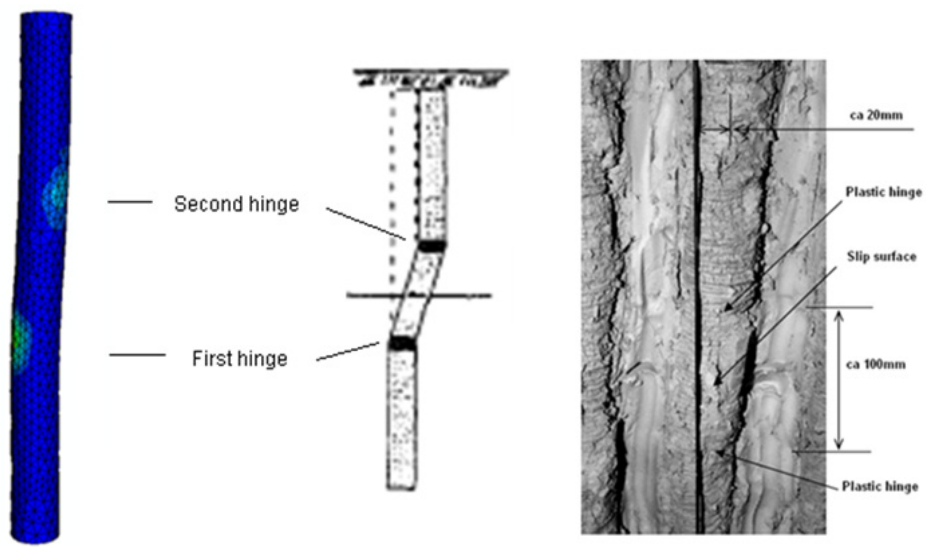


*Fig.5 Location of the plastic hinges. Results from ABAQUS, [3] and [5].*

The dowel force at the slip surface can be estimated (see [3] and [6]). There is a good agreement between the values from the shear box experiments, theoretical studies and this numerical analysis.

## 4 CONCLUSIONS

Numerical analyses were carried out with the finite element program ABAQUS. Analyses on unreinforced soil were used to calibrate the elastic properties of the clay. For analyses on reinforced soil, the lime/cement columns were modeled with the concrete damaged plasticity model for its ability to simulate cracks. The failure mode observed was bending. This is in agreement with the shear box experiments – which were the basis for this work – and other studies. The ultimate load was also close to the value obtained experimentally and lower than indicated by the design codes. This means that the design codes can overestimate the strength of the reinforced soil under the embankment because they do not consider bending failure, which is more probable than shear failure.

### Acknowledgements

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