

The Behavior of Dam Foundation Reinforced by Stone Columns: Case Study of Kissir Dam-Jijel

Toufik Karech, Abderahmen Benseghir, Tayeb Bouzid

Abstract—This work presents a 2D numerical simulation of an earth dam to assess the behavior of its foundation after a treatment by stone columns. This treatment aims to improve the bearing capacity, to increase the mechanical properties of the soil, to accelerate the consolidation, to reduce the settlements and to eliminate the liquefaction phenomenon in case of seismic excitation. For the evaluation of the pore pressures, the position of the phreatic line and the flow network was defined, and a seepage analysis was performed with the software MIDAS Soil Works. The consolidation calculation is performed through a simulation of the actual construction stages of the dam. These analyzes were performed using the Mohr-Coulomb soil model and the results are compared with the actual measurements of settlement gauges implanted in the dam. An analysis of the bearing capacity was conducted to show the role of stone columns in improving the bearing capacity of the foundation.

Keywords—Earth dam, dam foundation, numerical simulation, stone columns, seepage analysis, consolidation, bearing capacity.

I. INTRODUCTION

CONSTRUCTION of stone columns under an embankment is a common economical ground improvement method [1], [2], numerical modeling is a simple and efficient alternative for the approach of real behavior of soils reinforced by stone columns [2]. Representing a 3D stone column grid in 2D is usually represented by a series of parallel trenches. The stiffness as well as the permeability of both soft soil and coarse grained inclusion needs to be adapted in order to model the deformation behavior and drainage conditions for consolidation correctly [1]. The purpose of this work is to verify if the results obtained by calculating at the level of the reinforced foundation by ballasted columns are concordant with those measured in situ after the commissioning of the dam and to reduce the settlements and the risk of liquefaction [3].

II. PROJECT DESCRIPTIONS AND GENERAL DESIGN

Kissir Dam is located 14 km west of Jijel (Algeria); it is intended to increase the resources of drinking water, industrial and irrigation, ensuring a supply of 48 million m³ per year [4]. The levee consists of upstream and downstream refill of alluvium, a central clay core protected by a filter upstream and a filter and a drain downstream. It is based on a thick alluvial filling of low mechanical characteristics [4]. The typical cross-section of the levee is shown in Fig. 1.

The foundation of the dam consists of a thick alluvial filling

of low mechanical characteristics, which required treatment by stone columns [3].

The alluvial foundation is treated with stone columns under the influence of the dike to consolidate the soil in order to:

- improve the bearing capacity of the foundation of the dam.
- increase the equivalent mechanical characteristics of the reinforced soil.
- reduce horizontal and vertical deformations of the foundation of the dam.
- accelerate the consolidation of the main layer, thanks to the vertical draining effect of stone columns.
- eliminate the liquefaction phenomenon of fine sand in case of seismic solicitation.

The treatment has affected a total area of 28,000 m²; the theoretical depth of columns is 22.5 m for the central part, and 17 m for the sides [5]. The water tightness of the alluvial foundation is provided by a diaphragm wall; the sealing edge is provided by a grout curtain.

The levels and main dimensions of the dam are as [3]:

- Peak level: 48.25 NGA,
- Highest water level: 47.90 NGA,
- Full water level: 44.5 NGA,
- Maximum height on natural terrain: 48 m,
- Peak width: 8 m,
- Crest length: 368 m.

III. GEOTECHNICAL DATA

The levee is an earth dike with a symmetrical clayey core; its height is 50 m; the outer slopes vary from 2H/1V to 3H/1V upstream and 2H/1V to 2.5H/1V downstream.

The alluvial foundation is composed of generally fine materials, whose thickness may reach 40 m. Analysis of surveys and tests led to the consider the foundation as a homogeneous material [3]. The alluvial foundation is treated by a network of stone columns) with a treated surface of 28.000 m², the columns are arranged in a hexagonal mesh with a space step of 3 m. The theoretical depth of columns is 22.5 m (or refusal of substrate) for the central part and 17.5 m (or refusal of substrate) for the side parts. Fig. 2 presents a geotechnical cross-section of the foundation.

The mechanical properties of the foundation and fill materials are summarized in Table I [3].

IV. CONSTRUCTION SEQUENCE

The works began in June 2006 and were completed in late 2010. The levee was constructed between May 2008 and July 2009.

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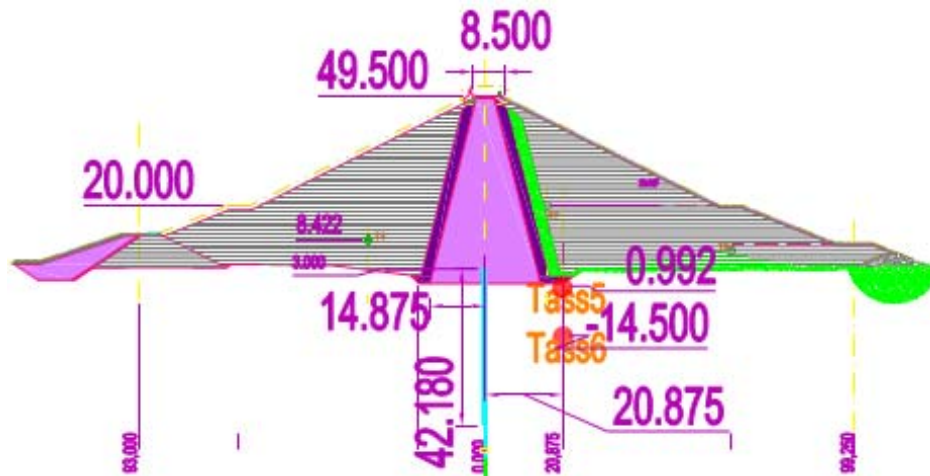


Fig. 1 Typical cross-section of the levee (PK 200.000)

TABLE I
MECHANICAL PROPERTIES OF THE FOUNDATION AND FILL MATERIALS

| | Specific weight γ (kN/m ³) | Friction angle ϕ' (degree) | Cohesion C' (kPa) | Young's modulus E (MPa) | Poisson's ratio ν | Hydraulic conductivity K (m/s) |
|---------------------|--|------------------------------------|----------------------|----------------------------|--------------------------|-----------------------------------|
| Alluvial foundation | 20 | 25 | 0 | 10 | 0.3 | 10 ⁻⁵ |
| Alluvium refills | 21 | 38 | 0 | 40 | 0.3 | 10 ⁻³ |
| Clayey core | 19.50 | 25 | 0 | 20 | 0.3 | 10 ⁻⁹ |
| Filter | 21 | 38 | 0 | 40 | 0.3 | 10 ⁻³ |
| Drain | 21 | 38 | 0 | 40 | 0.3 | 10 ⁻² |
| Rockfill | 21 | 38 | 0 | 40 | 0.3 | 10 ⁻² |
| Stone columns | 21 | 38 | 0 | 60 | 0.3 | 10 ⁻² |

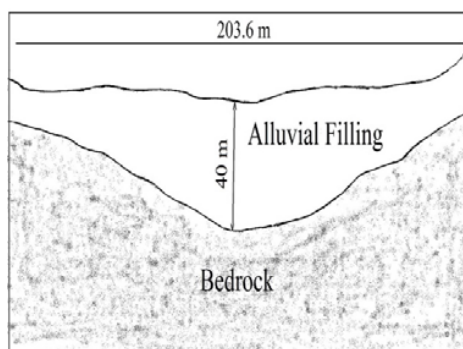


Fig. 2 Geotechnical cross-section of the alluvial foundation

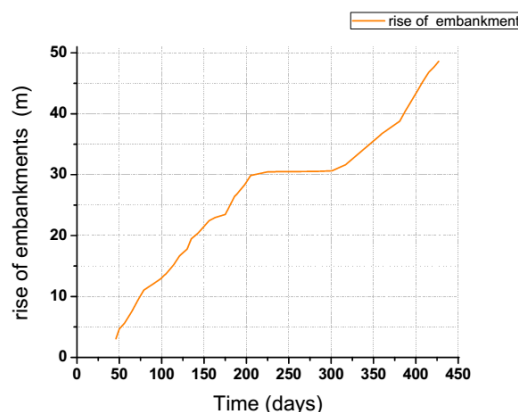


Fig. 3 Rise of embankment

Alluvial embankments were compacted by 50 cm thick layers; core clay materials were compacted by 30 cm thick layers. The rise of the embankment is synthesized in Fig. 3. Water filling began in October 2009 and the normal water level was reached in May 2010 [4].

The volume of the columns used is measured in the Free State without compaction. According to the specifications of DTU 13.2 [6], the volume must be at least 1.5 times the theoretical volume of the column used. This is the case for 90% of columns as measured in the field [4]. Moreover, due to the ballast expansion at the end of compaction, a mean increase of the column diameter of the order of 10% is observed. This increase of diameter is taken into account in the numerical model.

Besides the reinforcement due to the characteristics of the stone columns, the vibratory action during the installation of the columns densifies the soil and causes an increase of the mechanical properties of the surrounding soil and consequently the reduction of settlements [2], [5], [7], [8]. As this last effect is not yet clearly quantified, it is not taken into account in the numerical model.

V. NUMERICAL ANALYSIS

A. Presentation of the 2D Numerical Model

The numerical models are built according to the typical

cross-section of the dam, considering two cases for the foundation: untreated foundation and foundation treated with stone columns. The alluvial foundation is modeled by a 40-m

thick layer; the stone columns are 22.5 m long and 0.88 m thick, with a spacing of 3 m.

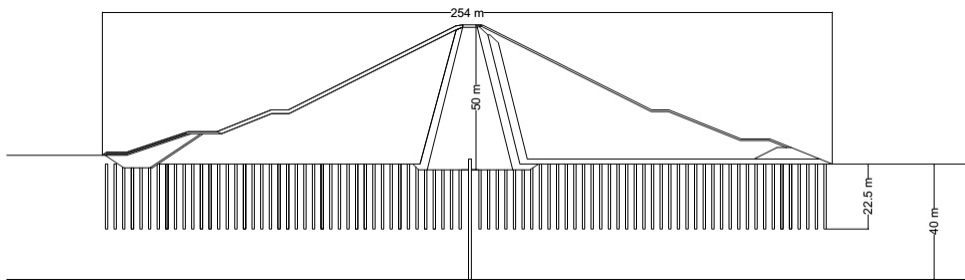


Fig. 4 2D Numerical model

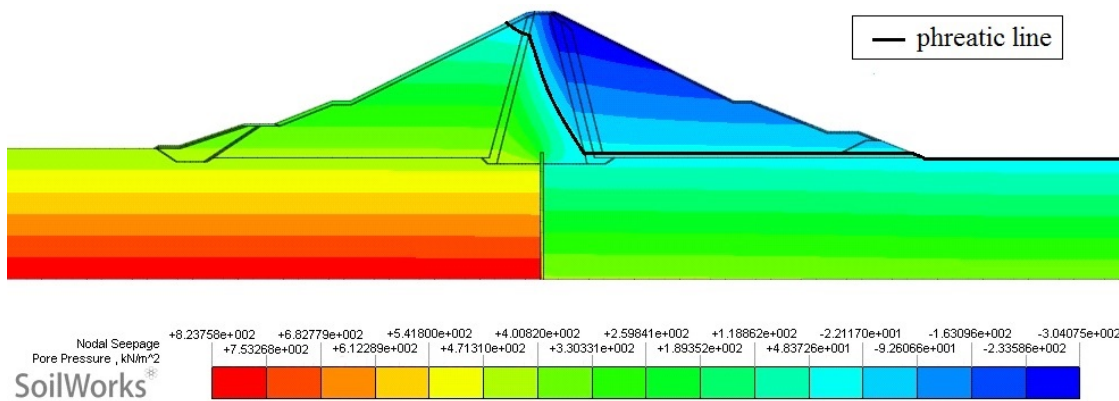


Fig. 5 Steady state – pore pressure

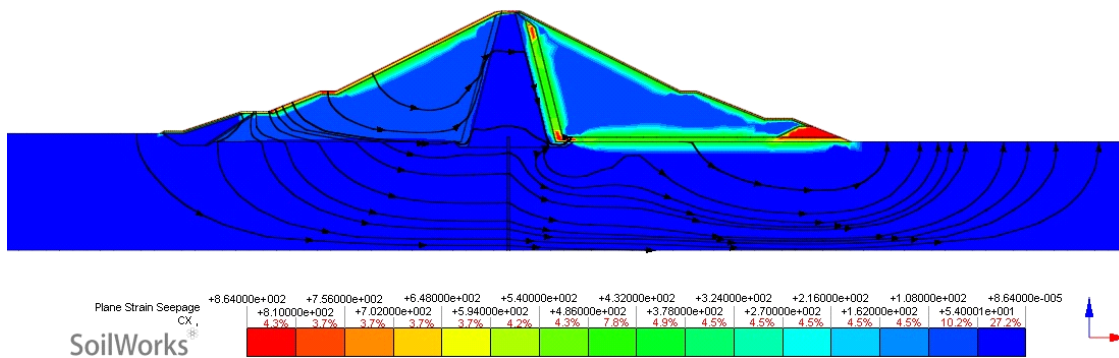


Fig. 6 Steady state - flow network - untreated foundation

Due to 2D modeling of the 3D network of stone columns, the substitution ratio is increased from 8.6% in 3D case to 29.33% in 2D model. One way of proceeding without affecting the 2D geometry is to reduce the rigidity of the columns in the ratio of $8.6/29.33 = 3.4$ to restore the rigidity equivalent to 3D.

The numerical analyzes were performed using the software MIDAS SoilWorks® for determining the position of the water line and the software PLAXIS® for the hydro-mechanical analysis.

The 2D numerical model is shown in Fig.4.

B. Seepage Analysis

This analysis allows to assess the pore pressures and the determination of the saturation line and flow network in steady state of full water level 44.5 m using Darcy's law to express the phenomenon of infiltration in saturated and unsaturated soils. Fig. 5 shows the pore pressure field and the position of phreatic line.

Figs. 6 and 7 show the steady flow network and the permeability coefficient in the case of untreated foundation and the treated foundation.

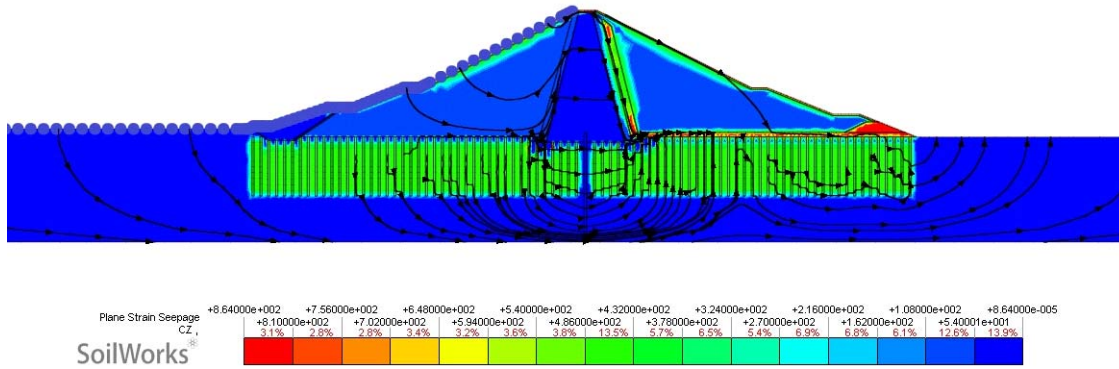


Fig. 7 Steady state - flow network - treated foundation

It is observed that the flow takes the trajectories where permeability is significant (stone column, drain), namely, the flow through the foundation is led by the stone columns.

C. Consolidation Analysis

A consolidation calculation was performed for the typical cross-section. These analyzes were performed using the elasto-plastic model of Mohr-Coulomb, by a simulation of the actual construction of the levee, taking into account the time of realization.

The results of the analysis are compared with the site measurements from the settlement gauges implanted in the dam. The cut PK-200,000 -where the depth of the alluvial foundation is maximum- contains five gauges: TASS4, TASS5, TASS6, TASS7 and TASS8.

- Settlement gauges: TASS4, TASS5, TASS8 are located at the contact dam-foundation.
- Settlement gauge: TASS6 is located in the foundation.
- Settlement gauge: TASS7 is located in the levee.

Note that the settlement gauges: TASS7, TASS8 have unstable measurements, then they were not taken into consideration.

Settlement gauges: TASS4, TASS7, TASS8 do not make part of the comparison.

The position of settlement gages is shown in Fig. 1. Figs. 8 and 9 show the deformed mesh of both models (with and without stone columns). It is observed that displacements are higher in the case of the untreated foundation; maximum vertical displacements are on top of the models, giving 3.85 m in untreated model and 3.42 m in treated model.

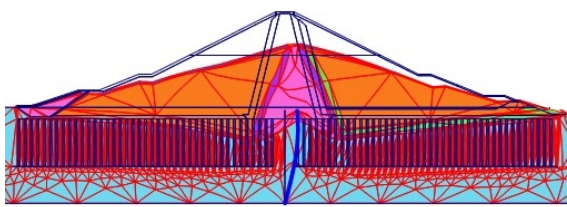


Fig. 8 Deformed mesh with stone columns

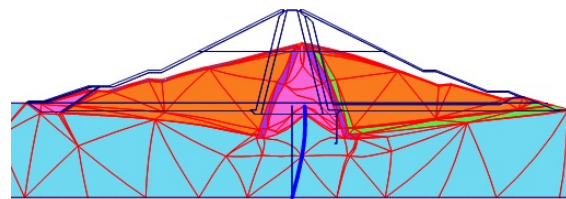


Fig. 9 Deformed mesh without stone columns

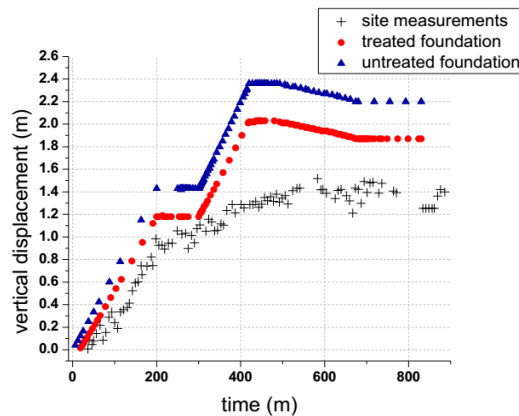


Fig. 10 Vertical displacement TASS5

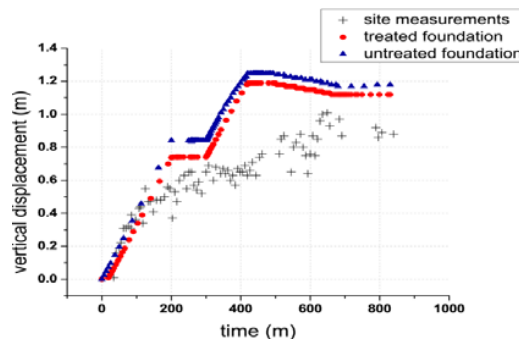


Fig. 11 Vertical displacement TASS6

Figs. 10 and 11 represent the vertical displacement curves of numerical models and the actual measurements from different settlement gauges. Measured settlements are lower

than the numerical ones with stone columns; this difference is due to the installation effect of the columns which increases the characteristics of the soil around the columns, and consequently reduces the settlements. In Fig. 11, it is observed that the settlement difference is reduced in depth.

D. Bearing Capacity Analysis

An analysis of the bearing capacity of the foundation was performed for both foundation models (reinforced or unreinforced). Fig. 12 shows the load-displacement curve that shows the evolution of displacement according to the load parameter. It is observed that treatment with stone columns increased the bearing capacity of the foundation for a very important value.

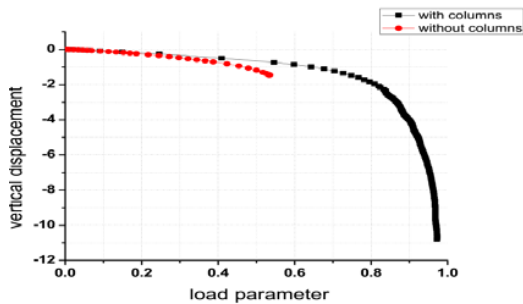


Fig. 12 The load-displacement curve

VI. CONCLUSIONS

Using 2D modeling to represent a 3D grid of stone columns needs an adjustment of parameters in order to preserve the correct conditions and behavior of the foundation. The displacement is more important in the case of untreated foundation; this difference was not observed when the depth increases. Settlements in real cases are less than the numerical model with stone columns; this difference is due to the installation effect of columns. Taking into account the effects of setting up columns in the numerical model requires further study.

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