Dynamic analysis of a compressible soil of foundation reinforced by mixed-module columns (CMM) and stone columns

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ABSTRACT

This work presents a numerical analysis of the dynamic behavior of reinforcements of liquefiable loose sand by flexible columns (stone columns) and Mixed Module Columns (CMM). In this study, the numerical simulation was carried out in 3D difference element analysis. Using a Finn model for dynamic Pore-Pressure Generation and the foundation subjected to the variable-amplitude harmonic ground motion of the Boumerdes earthquake (2003). When the height of the rigid part of the CMM has 50% of the column, the behavior of the improvement soil is the same by stone columns under the foundation, also the increase in the rigid part decreases the settlement. Overall, the high recorded accelerations by CMM indicate the largely preservation of the foundation stiffness.
Keywords: liquefiable loose sand; stone columns; Mixed Module Columns; Finn model; shallow foundation.

1. INTRODUCTION

Soil reinforcement with CMM (Mixed Module Column) combines two soil improvement methods, vibro replacement stone columns (short stone column with 1.50 m to 2.00 m long) with rigid inclusions displacement columns benefts and avoids their disadvantages. In the case of soil improvement by rigid inclusions we often have to adapt the shear strength of the upper part due to horizontal stresses linked to wind or sometimes to seismic loads and the bending moments for foundation slab, to avoid this disadvantages a vibro replacement column is installed in the top last meters part of the CMM (Keller document).

The behavior of CMM in seismic zones show that CMM dissipates an important energy (Hatem et al. 2010; Zhang et al. 2011; Lambert et al. 2013) in its flexible upper part much more than in the granular mattress of a system with mattress and rigid

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inclusions (Zhang et al. 2011) and the soil response reinforced by CMM without mattress is very close to that of the soil reinforced by rigid inclusions associated with a mattress of 1.5 m thick, but with the advantage of having less important internal efforts (Hatem et al. 2010). The length of the upper part has an important influence on the response of the lower rigid inclusions (Zhang et al. 2011). The footing bearing capacity with CMM increases (Lambert et al. 2013). In the most situation, the rigid part stays in the elastic domain by the effect of the gravel head of the CMM (Hatem et al. 2010).

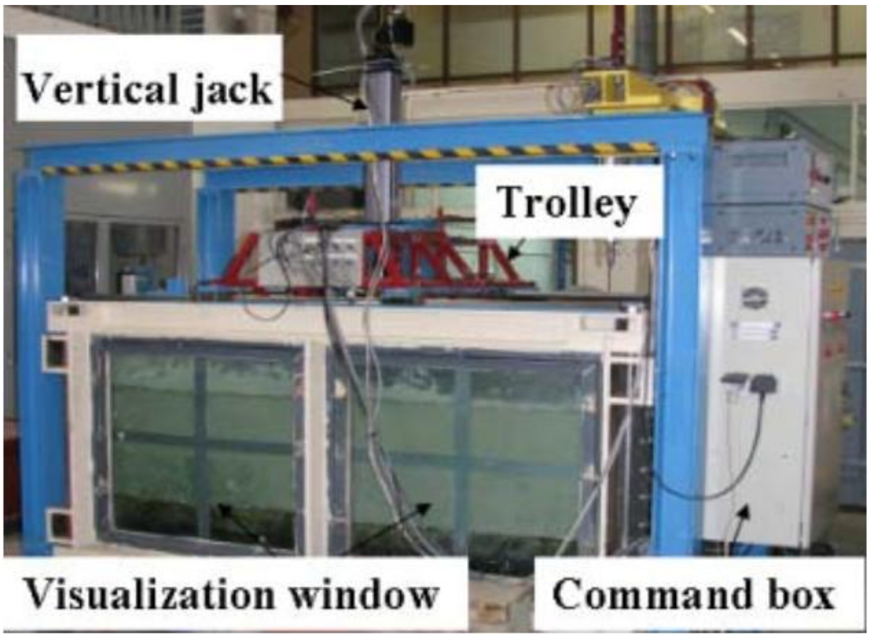
This paper proposes a numerical simulation by the program FLAC3D to design a shallow foundation resting on liquefiable soil of Boudouaou [Boumerdes-Algeria] (Bouafia 2014) with and without reinforced by CMM with a new vision where the length of gravel head (stone column) exceeds 2.0 m and presents most parts of the column. This paper proposes also to evaluate the dissipation of pore water pressure using of the Finn model with the criterion of Byrne (1991) under the foundation and the effect of the length of rigid part in the settlement and the strength of improvement soil.

2. THEORETICAL BACKGROUND

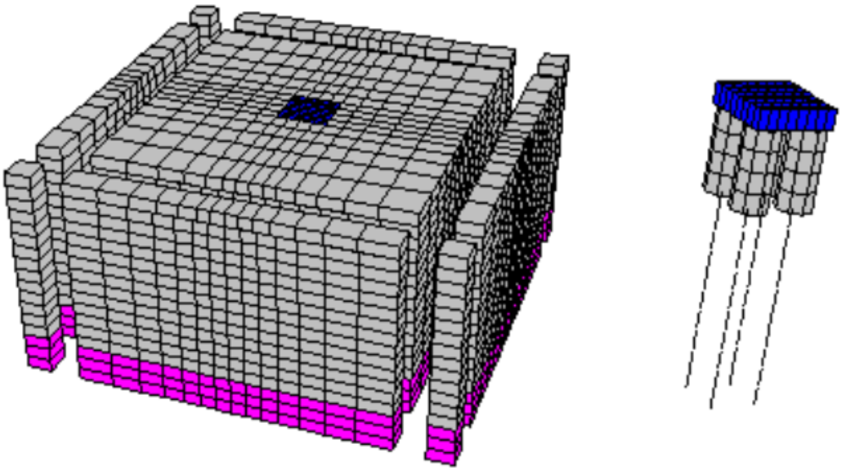
2.1 Seismic performance of mixed module columns (CMM)

Previous work focused on the comparison between seismic behavior of soft soils reinforced rigid inclusions and by mixed module columns. There is a numerical simulation by finite differences (FLAC3D) (Hatem et al. 2010), The work presents some results on the inertial interaction of the reinforced system subjected to a real seismic record (Tabas, IRAN), the results show that In all cases, the seismic forces induced by these two reinforcement systems are still much lower than those induced in equivalent classic piles.

Zhang et al. (2013) presents the study of the response of a square footing with 2 m width embedded in very soft clay reinforced by four CMM and submitted to cyclic horizontal loading with physical models in 2D and numerical simulation (Fig. 1).



(a)



(b)

Fig. 1 (a) ¨Physical models in 2D, (b) The grid of the numerical model of the CMM system.

The results of 2D experiments indicate that the system CMM dissipates an important energy in its flexible upper part, much more than in the granular mattress of a system with mattress and rigid inclusions. Horizontal displacements of the heads of the rigid inclusions were observed in dynamic loading. The 3D simulation indicate that the length of the upper flexible stone columns has an important influence on the response of the lower rigid inclusions by the high decreasing in the shear forces.

in situ tests performed by IFSTTAR and from a laboratory experimental study performed in Laboratoire 3S-R (Grenoble) of a shallow foundation with a group of Mixed Columns (Fig. 2) (Lambert et al. 2013).

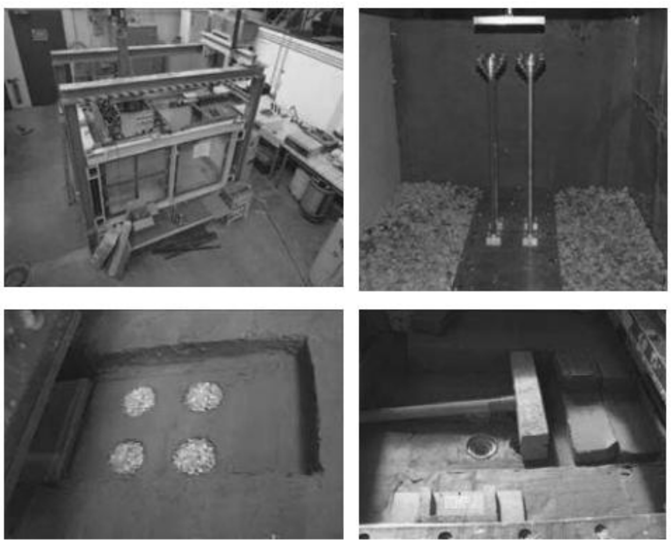


Fig. 2 Experimental study in Laboratoire 3S-R (Lambert et al. 2013)

To ensure optimal efficiency in reducing solicitations at the rigid part, the length of the flexible part of the CMM must exceed 1 m.

2.2 Finn model

Finn Model was used for the liquefiable materials when the pressure of the fluid increases and the effective stress acting on the grain matrix decreases. Martin et al. (1975) approach note that the relation between irrecoverable volume-strain and cyclic shear-strain amplitude is independent of confining stress. The principle in this method is the capture skeleton behavior under cyclic loading that imposes a volumetric constraint to consider EPWP.

This is a coupled equation between the incremental shear and the increment of volumetric strain under simple shear load (Martin et al. 1975; Byrne 1991; Itasca Consulting Group, Inc. 2005)

$$\Delta\varepsilon_{vd} = C_1.(\gamma - C_2.\varepsilon_{vd}) + \frac{C_3.\varepsilon_{vd}^2}{\gamma + C_4.\varepsilon_{vd}} \tag{1}$$

C_1, C_2, C_3 and C_4 are constants depending on the relative density of the sand and are related as follows C_1. C_2. C_4 = C_3 when $\Delta\varepsilon_{vd}$ = 0 (Itasca Consulting Group, Inc. 2005)

Byrne (1991) proposed a modified and simpler volume change model with two constants (C_1 and C_2).

$$\frac{\varepsilon_{vd}}{\gamma} = C_1.\exp\left(-C_2\left(\frac{\varepsilon_{vd}}{\gamma}\right)\right) \tag{2}$$

There is also the index form which allows the computing of the incremental volumetric strain induced in freely draining soil by each half cycle in a shear strain time history by the Eq. (3) (Carter et al. 2013):

$$(\varepsilon_{vd})_i = 0.5.(|\gamma_i| - \gamma_{thresh}).C_1.\exp\left(-C_2\frac{(\varepsilon_v)_i}{(|\gamma_i| - \gamma_{thresh})}\right) \tag{3}$$

Where $(\Delta\varepsilon_{vd})_i$ is the incremental volumetric strain induced by the i^{th} half cycle in the shear strain time history; γ_i is the amplitude of the i^{th} half cycle in the shear strain time history; γ_{thresh} is the threshold shear strain below which no volumetric strain will occur ; $(\varepsilon_v)i$ is the cumulative volumetric strain before the i^{th} half cycle in the shear strain time history is applied.

The parameter C_1 controls the amount of volume change (volumetric strain increment), and C_2 controls the shape of variation for the accumulated volume by the number of cycles (volumetric strain curve). Byrne (1991) recommended a correlation equation to obtain the model constant C_1 in terms of sand density D_r as:

$$C_1 = 7600(D_r)^{-2.5} \tag{4}$$

As the form is the same for all densities, the C_2 parameter is a constant fraction of C_1 for all relative densities and can be prescribed as follows:

$$C_2 = \frac{0.4}{C_1} \tag{5}$$

The coefficient C_1 is assigned from the SPT value (N_{60}) as below (Byrne 1991; Itasca Consulting Group, Inc. 2005):

$$C_1 = 8.7.(N_{60})^{-1.25} \tag{6}$$

3. NUMERICAL SIMULATION

3.1 Model geometry, soil properties and constitutive soil model

A finite difference software, (*FLAC 3D* - **F**ast **L**agrangian **A**nalysis of **C**ontinua), is used during this study for the analysis of the liquefaction of a foundation on a saturated loose sand deposit without and with reinforced by CMM, coupled with a dynamic-groundwater flow (effective stress), the model is represented by 4280 zones and 5908 grid-points. Calculations' code (FLAC3D*)* for this study uses the Finn model to carry out pore-pressure build-up (Itasca Consulting Group, Inc. 2005).

The shallow foundation is plan rectangular of 8.05 m in length, 4.0 m in width and 1.0 m in thickness resting on loose sand layer without and with eight CMMs, the CMM 's diameter is 0.8 m, the matress with 0.55 m in thickness supporting a shallow foundation and a load of 150 KPa (Fig. 3).

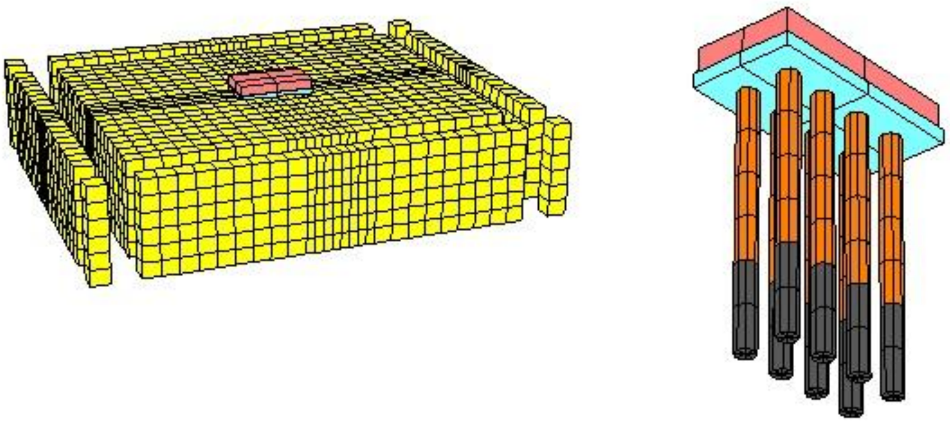


Fig. 3 The grid of the numerical model of the CMM

The limits of the free field impose in which lateral limits of the main grid of the model are coupled into the grid of the free field by viscous dashpots to simulate a quiet boundary. With this model, plane waves propagating upward suffer no distortion at the boundary because the field-free grid supplies conditions that are identical to those in an infinite model (Itasca Consulting Group, Inc. 2005).

The frequency content of the input wave and the wavespeed characteristics of the system will affect the numerical accuracy of wave transmission, and they can result in a numerical distortion of the propagating wave. To solve this problem, the spatial element size, *Δl*, must be smaller than approximately one-tenth (1/10) to one-eighth (1/8) of the wave length associated with the highest frequency component of the input wave ($\Delta l \leq \frac{\lambda}{10}$) (Lysmer and Kuhlemeyer 1969).

The loose sand representing a liquefiable site which is in the region of Boudouaou-Algiers, the geotechnical characteristics of the sandy material are (Bouafia 2014):

- Friction angle $\phi = 29^o$;
- Shear velocity V_s = 185 m/s;
- Poisson's ratio v = 0.4;
- Saturated unit weight $\Upsilon_{sat} = 17kN/m^3$.

The ballasted column has the following properties (Dhouib and Blondeau 2005):

- Unit weight of ballast $\Upsilon = 20kN/m^3$;
- Cohesion C = 0 kPa, $\phi = 38^0$;
- Module of deformation $E_c/E_s = 10$, E_s is the dynamic deformation module of the soil calculated in function of V_s.

A Rayleigh damping of 5% is considered in this model. The mattress has the same properties of stone column.

The behavior of the soil is studied by the Finn model for modeling the liquefaction phenomenon using a built-in pore pressure generation, the classic Mohr-Coulomb criterion for flexible part and Elastic behavior for rigid part.

3.2 The seismic load

On May 21, 2003 at 19:44:19 local time, Zemmouri area in the Boumerdes region about 70 km east of the capital Algiers has suffered an earthquake of magnitude in the order of 6.8. The location of the epicenter is (36,90 North, 3.71 Est) determined by the U.S.G.S (united states geological survey). The focal depth of the earthquake was about 10 km. The recorded amplitude is given by the station of Dar El Beida (0.52 g) source (C.G.S. Center of earthquake engineering, Algeria) (Japanese Reconnaissance Team. Boumerdes earthquake, May 21, 2003).

The earthquake lasted 27.675 seconds, and the strong accelerations were between 6 and 10 seconds. The maximum acceleration is of 556.79 cm/s^2 (0.57 g) at 7.70 seconds after the *Baseline Correction* made by *SeismoSignal* software. In the response spectrum, the strong horizontal seismic acceleration corresponds nearly to 3 Hz and 6 to 8 Hz (Fig. 4).

The earthquake is modelized by a sinusoidal acceleration applied at the base of the model in the horizontal direction after static equilibrium was achieved. For the long calculation time when the accelerogram is applied, the accelerogram is converted to the variable-amplitude harmonic ground motion record illustrated in Fig. 5 and expressed by the following equation.

$$a(t)=\phi.ampl.e^{(-t/tf)}.\sqrt{\beta.e^{-\alpha t}}\,t^{\xi}\sin(2\pi ft) \qquad (7)$$

Where ϕ = 0.73 and *tf* = 6 (time of ground motion).

This equation is a contribution from the expression of Bathurst and Hatami (1998):

$$a(t)=\frac{ampl}{2}.\sqrt{\beta.e^{-\alpha t}}\,t^{\xi}\sin(2\pi ft) \qquad (8)$$

The wave has a frequency (*f*) of 3.3 Hz with an amplitude (*ampl*) equal to the maximum acceleration of the accelerogram of Dar El Beida EW = 0.57g. Where: *α* = 5.5, *β* = 55, and *ζ* = 12 are constant coefficients; *f* = frequency; and *t* = time = 6 seconds.

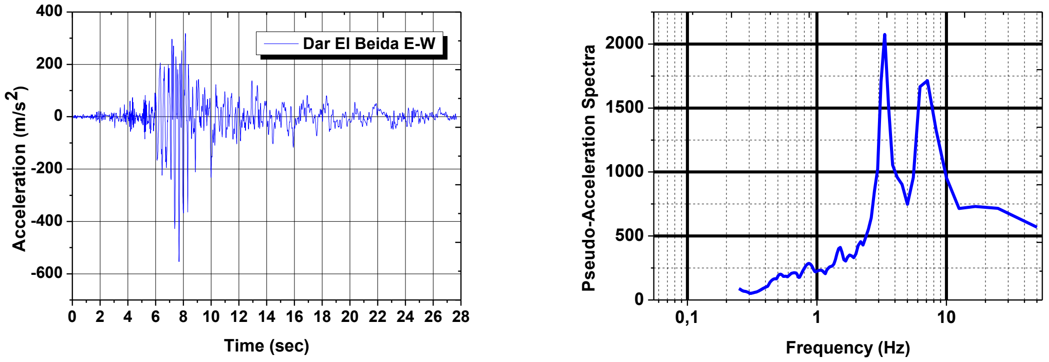


Fig. 4 Accelerogram Signal Est-West at the Dar El Beida station and corresponding response spectrum.

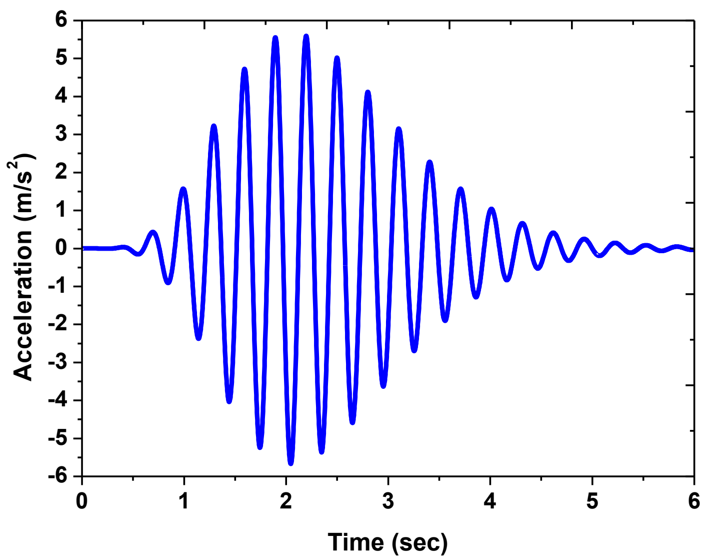


Fig. 5 Variable-Amplitude harmonic ground motion

4. RESULTS AND DISCUSSION

4.1 Comparison between CMMs and stone columns

After a numerical analysis of the variation of the EPWP at 2.485 m in depth (zone close to the surface) with improved by CMMs with different length of rigid part (3 m and 5m) and stone columns under the centerline of foundation.

Fig. 6 displays the recorded EPWP and velocities during the motion under the centerline of foundation.

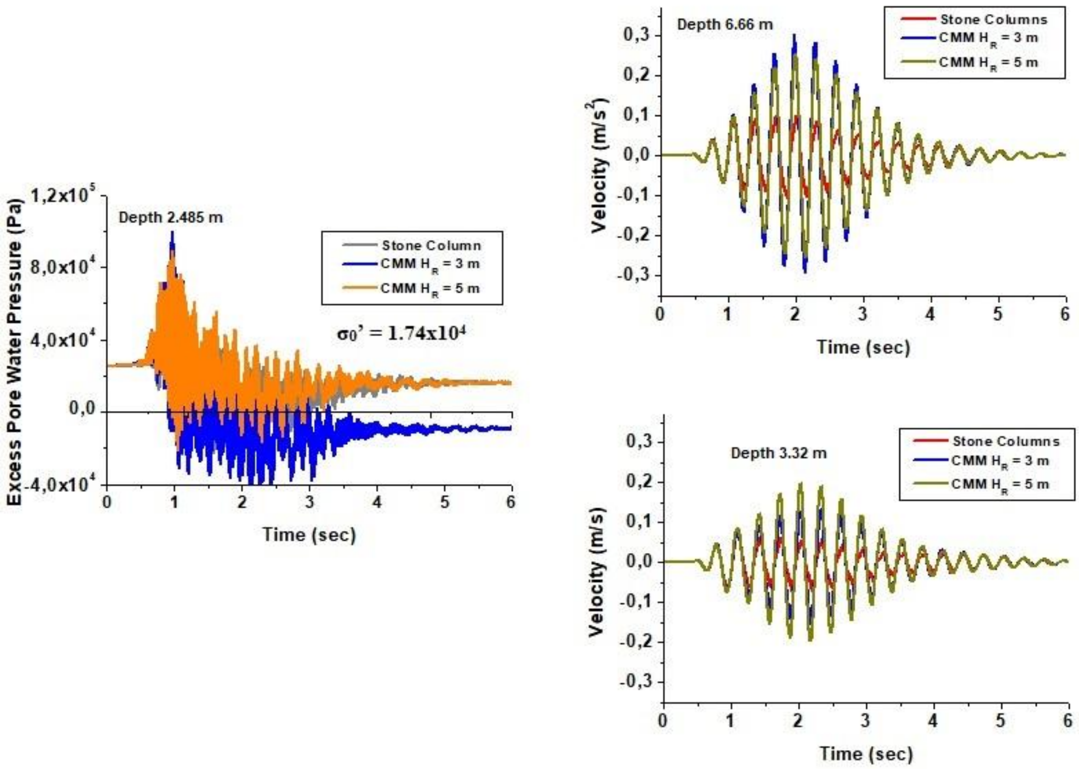


Fig. 6 EPWP and velocities measured under the centerline of foundation

As seen in Fig. 6, a negative EPWP build-up tendency (the increase in the magnitude and the spatial extent of horizontal normal strains in the foundation with stronger base input excitation (Adalier et al. 2003), in other words dilatation of soil under the centerline of foundation) with improved by CMMs where the length of rigid part = 3 m. However, with CMMs where the rigid part = 5 m and stone columns, a significant positive EPWP was attained and the liquefaction eliminates.

The velocity with CMMs is much stronger (2 to 3 times) than with stone columns.

4.2 Effect of the length of the rigid part of CMMs in settlement

In this paper, to examine the influence of the length of the rigid part of CMMs in settlement. A series of modeling, each one is modeled with a value of length of the rigid part (3 m and 5 m). The other model considered the improvement by stone columns.

Fig. 7 presents change of the magnitude of settlement for different models under the centerline of foundation.

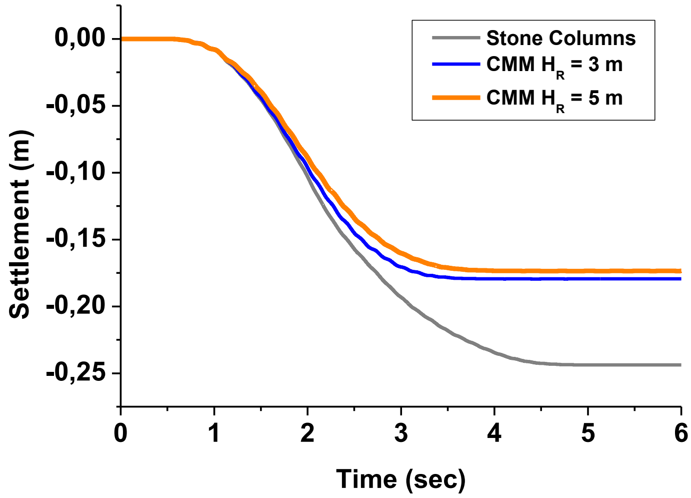


Fig. 7 Effect of the length of the rigid part of CMMs in settlement

In general, the settlement with the improvement with CMMs is lower than with stone columns [(settlement $_{(CMM)}$/Settlement $_{(Stone\ Column)}$)] = 0.74 (Reduction of settlement with CMMs = 26%), the settlement in the case of CMMs with the two values of H_R almost the same. The accumulation of settlement with CMMs is faster than improvement by stone columns.

5. CONCLUSIONS

A comparative dynamic analysis between of a compressible soil of foundation reinforced by mixed-module columns (CMM) and stone columns was made using numerical modeling.

EPWP and velocity results indicate a stiffer role of CMMs for the loose sand compared to stone columns. However, in this study the velocity in the soil improved by CMMs is greater than improved by stone columns and the negative EPWP build-up under the centerline of foundation was eliminated with stone columns and CMMs where the length of the rigid part equal 5 m in this study and the same build-up in EPWP but with the advantage of stronger velocity with CMMs, that's mean clearly that the CMMs largely
preserved foundation stiffness more effectively than stone columns.

The increase in the length of the rigid part of CMMs helps the upper flexible part to reduce te settlement, in this study the reduction has reached 26%.

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