International Journal of Innovative Research and Scientific Studies, 6(2) 2023, pages: 310-321

IJIRSS International Journal of Innovative Research & Scientific Studies

ISSN: 2617-6548

URL: www.ijirss.com

Evaluating the efficiency of stone columns in mitigating liquefaction risks and enhancing soil bearing capacity: A case study

Mohamed BZIAZ[1*], Lahcen BAHI[2], Latifa OUADIF[3], Anas BAHI[4], Hamou MANSOURI[5]

[1,2,3,4]GIE Laboratory, Mohammadia Engineering School, Mohammed V University in Rabat-Morocco.
[5]LPEE Public Laboratory for Tests and Studies, Morocco.

Corresponding author: Mohamed BZIAZ (*Email: bziazm@gmail.com*)

Abstract

Several processes allow the improvement of soils such as vibrocompaction, stony columns, static horizontal compaction etc. These processes reduce the risk of liquefaction potential and making it possible to build on this type of soil when the space restriction require it. Stone columns are a recognized method of soil improvement, which consists of creating large diameter columns using special vibrators with granular filling materials introduced into the ground. The objective of this work is to evaluate the effectiveness of stone columns, made in a seismic zone containing liquefiable materials, with regards to the reduction of the risk of liquefaction and the improvement of the bearing capacity of the soil. The approach followed is the exploitation of geotechnical investigation tests (CPT Cone Penetration Test), (SPT: Cone Penetration Test, (pressuremeter tests), carried out before and after soil treatment. This study showed that the network of gravelled columns produces an enhanced soil improvement effect by improving the modulus of elasticity of the soil and reducing settlement and the risk of liquefaction after treatment. The process allows for the improvement of the geotechnical characteristics of its soils, making it possible to build infrastructure and development projects in the area.

Keywords: CPT, Liquefaction, Potential, SPT, Stone columns, Vibrocompaction.

DOI: 10.53894/ijirss.v6i2.1406
Funding: This study received no specific financial support.
History: Received: 30 August 2022/**Revised:** 13 February 2023/**Accepted:** 28 February 2023/**Published:** 15 March 2023

Authors' Contributions: All authors contributed equally to the conception and design of the study.
Competing Interests: The authors declare that they have no competing interests.
Transparency: The authors confirm that the manuscript is an honest, accurate, and transparent account of the study; that no vital features of the study have been omitted; and that any discrepancies from the study as planned have been explained.
Ethical Statement: This study followed all ethical practices during writing.
Publisher: Innovative Research Publishing

1. Introduction

Limiting the risk of liquefaction is a permanent concern for the designers, who very quickly seek to improve or to reinforce soil with the concern of retaining a process that is both effective and the least expensive in a cost/benefit type analysis. The technique of stone columns is an extension of vibro-flotation to land with silty or clayey layers whose elements cannot be rearranged by vibration. Stone columns make it possible to treat these soils by incorporating granular materials, commonly known as ballast, compacted by ascending passes. They also serve as drains and accelerate the natural process of consolidation. The extent to which the capacity of granular soil is improved by vibrocompaction depends on geotechnical parameters that are difficult to assess, as well as the equipment and method used. The difficulty of making a reliable prognosis is offset by the ease of in situ measurement of the degree of compactness achieved, with simple soundings. This relationship is reversed to a certain extent with stone columns. The reinforcement effect of stone columns can only be measured in situ by very expensive loading tests. On the other hand, it is possible to establish a reliable prognosis on the improvement factor obtained in a given soil, which can be determined without considering any modification of its initial characteristics, only from the existence of the network of stone columns. This is possible because the soil improvement parameters, which in fact only depend on the geometry of the treatment and the characteristics of the filler material, can be determined with satisfactory accuracy. In such a calculation, the difficult-to-determine geotechnical parameters of the soil, the equipment and the method used, play only an indirect role, essentially only in determining the diameter of the columns. The purpose of this article is to describe the effect of the placement of stone columns on the mass of the globally and locally reinforced soil (inter-column soil).

2. Study of a Case (Souani Zone in the City of Al-Hoceima)

Our case study is located in northern Morocco in the city of Al Hoceima, precisely, the area of Souani, Figure1 illustrates the Geological Map of the study area.

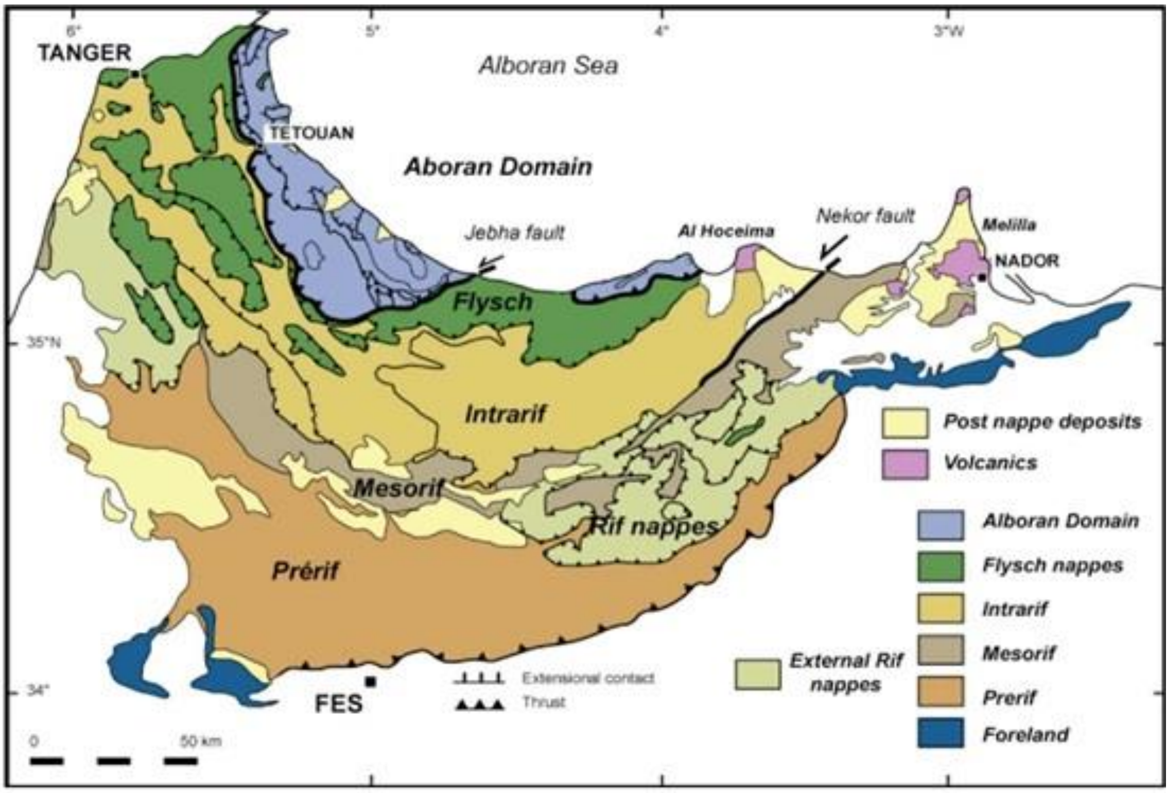

Figure1.
Geological map of the study area.

2.1. Geological Setting

The lands of the municipality of Al Hoceima, are predominated by outcrops of carbonate rocks of the Ghomarides and internal limestone ridge, which are in the form of massive and limestone dolomites with outcrops of shale and marl-sandstone formations. This context is delimited via abnormal contacts (overthrust and stacking) to the East, South and West by a Liassic unit of limestone with flint and microbreccias containing levels of limestone, dolomites and radiolarites, to the North -West by a block of massive Triassic dolomite and to the north by the Paleozoic unit presented by limestones from the Tirhanimine and the southern tip of Al Hoceima. These units are covered in places by lands of recent Quaternary cover.

2.2. The seismicity of the Region

The province of Al Hoceima is part of the Rif chains, and this region is the seat of a very active instrumental seismic activity, which has been verified and validated by a set of studies which have highlighted the existence of numerous structures active within the Riffian chains. This seismic activity is also notable through the historical seismicity as evidenced by the earthquake of May 26, 1994. The region of Al-Hoceima was hit by an earthquake of magnitude Mw = 5.8-6, destroying many habitats in the city of Al -Hoceima and in the surrounding countryside. Ten years later, on February 24, 2004, a deadly earthquake of magnitude Mw=6.3-6.5 also occurred in the same province, which led to the death of more than 600 people and the destruction of 2,500 homes. This earthquake was felt more than 300 km from the epicenter. In addition, the region of Al Hoceima is part of zone No. 3 according to the RPS 2000 revised in 2011, and therefore a maximum acceleration of 0.14 g is considered.

2.3. In Situ Tests

Before the soil treatment ten (10) Standard penetration test (SPT): tests were carried out. Given the number of these tests, a detailed example was carried out for a single SPT test. For the other polls, we simply used the final result. After the reinforcement of the soil, sixteen (16) Cone Penetration Test (CPT) tests, four (4) pressuremeter surveys and four (4) loading tests were carried out.

3. Methodology

3.1. Assessment of Liquefaction Risk

In the presence of groundwater, the resistance to liquefaction of the soil is evaluated by applying the NCEER (National Center for Earthquake Engineering Research) method, developed by Youd and Idriss [1], Equation 1 presents the safety factor of soil liquefaction:

$$LSF = CRR/CSR \quad (1)$$

With:

CSR: cyclic stress ratio.

CRR: cyclic resistance ratio.

According to EN 1998-5 [2] and RPS [3], liquefaction occurs if the ratio is:

$$CRR/CSR < 1.25 \ (2)$$

3.1.1. Cyclic Stress Ratio (CSR) Assessment

Seed and Idriss [4] formulated the following Equation 3 for the calculation of the cyclic stress ratio CSR.

$$CSR = \frac{\tau_{av}}{\sigma'_{v0}} = 0.65\left(\frac{\alpha_{max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) r_d \quad (3)$$

Where τ_{av} is the average cyclic shear stress; $\alpha_{max}$is the maximum horizontal acceleration at the ground surface; g = 9.81 m² / s is the acceleration due to gravity; σ_{v0} is the initial vertical total stress; σ'_{v0} is theinitial effective vertical stress; r_d is the stress reduction factor.

The stress reduction coefficient is expressed as a function of depth by the following Equations 4 and 5 [5]:

$$r_d = 1 - 0.00765z \qquad z \leq 9{,}15m (4)$$

$$r_d = 1.74 - 0.0267z \qquad 9{,}15m < z \leq 23m \ (5)$$

3.1.2. Evaluation of Cyclic Resistance Ratio (CRR) [6-8]

The determination of cyclic soil resistance can be carried out using data obtained from in-situ tests (Standard penetration test, Cone penetration test, shear wave velocity measurement). To incorporate the effect of earthquake magnitude (duration of earthquake or number of cycles), an MSF magnitude correction factor that adjusts the CRR value to an earthquake magnitude of 7.5 is added in the following Equation 6 which becomes as follows:

$$CRR = CRR_{7.5} * CM \qquad (6)$$

With CM: correction factor, determined according to EN 1998-5 (2004) (English) [2] based on surface wave amplitude M.

Table 1 presents the Correction factor (CM), determined according to Annex B of EN 1998-5 standard [2], based on the magnitude of surface waves (M).

Table 1.
Correction factor (CM), determined according to Annex B of EN 1998-5 standard [2], based on the magnitude of surface waves (M).

M	CM
5.50	2.86
6.00	2.20
6.50	1.69
7.00	1.30
8.00	0.67

Note: CM: Correction factor, M: Magnitude of surface waves.

For a magnitude of 7.5 an approximation of CRR is given by the following formula :Idriss and Boulanger [8]

$$CRR_{7.5} = \mathrm{EXP}\left[\frac{(N1)_{60cs}}{14.1} + \left(\frac{(N1)_{60cs}}{126}\right)^2 - \left(\frac{(N1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N1)_{60cs}}{25.4}\right)^4 - 2.8\right]$$

$$(N1)_{60cs} = (N1)_{60} + \mathrm{EXP}\left(1.63 + \frac{9.7}{FC+0.1} - \left(\frac{15.7}{FC+0.1}\right)^2\right) \quad (7)$$

10 SPT tests were carried out in the study area, the result of the calculation of the safety factor LSF based on these spt tests is presented in Table 2.

Table 2.
Evaluation of the risk of liquefaction.

Sondage	Depth	NSPT	FC%	CSR	CRR(M=7.5)	CRR(M=7)	*LSF*
SPT1	1	13	4	0.14	0.43	0.56	3.96
	2	26	4	0.14	6.04	7.85	55.75
	7	4	49	0.21	0.12	0.15	0.72
	8	6	22	0.21	0.12	0.16	0.75
	9	7	22	0.22	0.13	0.17	0.77
	10	14	22	0.22	0.18	0.23	1.06
	11	22	39	0.21	0.29	0.36	1.69
	12	58	5	0.21			> 10
SPT2	1	16		0.14	1.31	1.70	12.00
	2	15		0.14	0.26	0.34	2.42
	3	18	6	0.17	0.30	0.39	2.34
	4	13	12	0.18	0.19	0.25	1.36
	5	24	5	0.20	0.41	0.54	2.74
	6	13	3	0.22	0.15	0.20	0.93
	7	12	9	0.22	0.14	0.19	0.86
	8	9	16	0.22	0.14	0.18	0.81
	9	10	13	0.22	0.13	0.17	0.78
	10	11	35	0.22	0.16	0.21	0.95
	11	11	30	0.21	0.16	0.20	0.93
SPT3	1	6	9	0.14	0.15	0.19	1.37
	2	14	8	0.14	0.24	0.31	2.23
	3	13	5	0.17	0.19	0.24	1.46
	4	20	4	0.18	0.31	0.40	2.19
	5	12	34	0.20	0.20	0.26	1.35
	6	15	26	0.22	0.23	0.31	1.42
	7	22	22	0.22	0.38	0.50	2.30
	8	51	2	0.22			> 10
	9	18	20	0.22	0.23	0.29	1.34
	10	12	4	0.22	0.12	0.16	0.74
	11	12	11	0.21	0.13	0.17	0.80
	12	12	11	0.21	0.13	0.16	0.79
	13	14	27	0.20	0.17	0.21	1.04
SPT4	3	37	3	0.17			> 10
	5	26	6	0.20	0.56	0.73	3.73
	8	7	13	0.22	0.11	0.15	0.68
	9	2	23	0.22	0.10	0.13	0.57
	10	6	23	0.22	0.12	0.15	0.72
	11	10	31	0.21	0.15	0.19	0.89
	12	8	24	0.21	0.13	0.17	0.79
	13	7	25	0.20	0.12	0.16	0.76
	14	14	22	0.20	0.16	0.20	1.01
SPT5	3	21	5	0.17	0.47	0.61	3.62
	4	26	3	0.18	0.84	1.09	5.90
	6	8	10	0.22	0.12	0.16	0.73
	7	12	10	0.22	0.15	0.19	0.88
	8	12	36	0.22	0.18	0.23	1.06
	9	8	45	0.22	0.14	0.18	0.84
	10	8	13	0.22	0.12	0.15	0.69
	11	12	9	0.21	0.13	0.16	0.76
	12	8	20	0.21	0.13	0.16	0.77
	13	12	21	0.20	0.15	0.19	0.93
SPT6	4	27	5	0.18	1.08	1.40	7.57
	5	24	4	0.20	0.41	0.54	2.74
	6	8	33	0.22	0.15	0.20	0.92
	7	14	5	0.22	0.15	0.20	0.92

Sondage	Depth	NSPT	FC%	CSR	CRR(M=7.5)	CRR(M=7)	*LSF*
	8	31	14	0.22	0.90	1.17	5.32
	9	17	16	0.22	0.20	0.26	1.20
	10	18	9	0.22	0.18	0.22	1.03
	11	13	6	0.21	0.13	0.16	0.76
	12	17	10	0.21	0.16	0.20	0.96
	13	8	20	0.20	0.13	0.16	0.77
	14	12	7	0.20	0.12	0.14	0.72
	6	25	9	0.22	0.41	0.56	2.58
	7	36	5	0.22	2.24	3.01	13.71
	8	34	5	0.22	0.91	1.18	5.38
SPT7	9	30	5	0.22	0.41	0.52	2.40
	10	18	5	0.22	0.17	0.21	0.99
	11	22	5	0.21	0.20	0.25	1.16
	12	18	5	0.21	0.16	0.20	0.94
	4	12	15	0.18	0.19	0.25	1.35
	5	26	7	0.20	0.57	0.75	3.79
	6	19	14	0.22	0.27	0.36	1.68
	7	11	8	0.22	0.13	0.17	0.79
SPT8	8	12	10	0.22	0.14	0.18	0.84
	9	6	17	0.22	0.12	0.15	0.68
	10	10	22	0.22	0.15	0.19	0.87
	11	10	21	0.21	0.14	0.18	0.85
	12	16	26	0.21	0.19	0.24	1.14
	3	19	97	0.17	0.68	0.89	5.29
	4	16	15	0.18	0.27	0.35	1.88
	5	9	6	0.20	0.12	0.16	0.82
	6	14	2	0.22	0.16	0.22	0.99
	7	11	2	0.22	0.13	0.17	0.77
SPT9	8	13	4	0.22	0.14	0.18	0.83
	10	7	8	0.22	0.10	0.12	0.57
	11	5	10	0.21	0.09	0.11	0.54
	12	14	56	0.21	0.18	0.22	1.07
	13	16	35	0.20	0.19	0.24	1.15
	4	8	86	0.18	0.16	0.21	1.15
	5	7	50	0.20	0.15	0.19	0.98
	6	12	2	0.22	0.14	0.19	0.88
	7	16	7	0.22	0.17	0.23	1.04
	8	20	10	0.22	0.22	0.29	1.31
SPT10	9	18	10	0.22	0.19	0.24	1.10
	10	10	26	0.22	0.15	0.19	0.89
	11	11	7	0.21	0.12	0.15	0.69
	12	12	13	0.21	0.14	0.17	0.83
	13	9	33	0.20	0.14	0.17	0.85

It is noted that at the level of the lithological section, the sand and the sandy marl between the depths of 6 and 12 m where the water table is reached, presents a risk of liquefaction, a soil treatment is essential. It should be noted that the water table is encountered at a depth of 3m.

3.2. Use of Pressure Tests

To assess the bearing capacity of the soil, four pressuremeter tests were carried out indicating the lithological sections of the terrain. Similarly, the synoptic diagrams showing the vertical variations of the various recorded parameters appear in the sections of the destructive soundings. The results obtained by pressuremeter tests at ground level between 0 and 15m deep show the following pressuremeter characteristics 0.5Mpa ≤ PL ≤ 3.00 Mpa.

1 Mpa≤ EM ≤ 50 Mpa.

Based on these findings, it is evident that the sand, silt marl and gravel formations encountered between 1 and 15m deep have low to moderate mechanical strength and exhibit characteristics of an under consolidated to normally consolidated material.

3.3. Soil Reinforcement

Several soil reinforcement techniques can be considered, the choice depends mainly on the grain size of the soil to be treated [9], these techniques can be presented as follows:

3.3.1. Improvement of Fine Soils

Fine soils are characterized by:

- They undergo significant deformation when subjected to loads.
- These deformations are not instantaneous and can last for months or even years.
- Their bearing capacity is often low which can result in excessive settlement, differential settlement and long-term deformations, leading to instability or even rupture of structures.

These strengthening techniques include:

* Pre-loading: This is a technique which aims to consolidate the compressible layers and obtain settlement in order to stabilize the surface of the ground before setting up the structure.

*Acceleration of consolidation: In general, the rate of consolidation of fine soil is too low. This is due to the long path that the water must travel to get out of the soil, which implies a compaction which lasts over time. In order to reduce these "unacceptable" times, a drainage system (vertical and horizontal drains) is instilled to shorten the distance traveled by the water.

*Reinforcement by stone columns: These columns consist of granular material and are placed in the ground either by repression (dry way) or by replacing part of the soil (wet way) and are then compacted by successive passages.

3.3.2. Improvement of Granular Soils

Unlike fine soils, the permeability of cohesionless soils prevents an increase in pore pressure (unless liquefaction occurs). The problems that this type of soil can encounter are the extent of settlement and the resistance to liquefaction.

Methods for strengthening these soils are:

*Preloading: The principle is the same as for the fine soil, except in this case, the consolidation time is shorter. *Vibrocompaction: This involves the application of vibrations to the ground, using a vibrating needle, which leads to subsidence of the soil around the needle, reflecting a densification of the soil.

*Static compaction: It is about the introduction of mortar in force in the ground, this technique allows the possibility to densify the ground and to increase its load-bearing capacity.

*Dynamic compaction: This method involves repeatedly placing heavy steel rammers (10 to 20 tons) on the soil surface from a height of 15 to 20m.

3.4. Soil Treatment

a- Choice of Treatment

According to paragraph 3.1, there is a risk of liquefaction at depths 6 to 12 m. The particle size analysis shows that the fines content is greater than 5% for this zone. So, vibro-compaction alone cannot constitute a treatment. In addition, we opted for a treatment using stone columns because the columns intervene at three levels [10]:

i. Reduction of the shear stress applied to the soil.
ii. Evacuation of pore pressures.
iii. Increase of soil compaction between the columns.

b-Dimensioning of Stone Columns

The dimensioning of stone columns is based on the document “Recommendations for the design, calculation, construction and quality control of stone columns under buildings and sensitive structures (CFMS)” [11]. Based on pressuremeter test data - the dimensioning was carried out for the footings of (2*2)m2, whose surface Ss=4m2, supported on 2 columns of 0.8m in diameter with surface Scol=0.5m2, bearing surface load qELS= 2, 5 bar

- First, we checked the condition:

$$\{n.Scol.qa + [(Ss - n, Scol).q'u / 3]\} > qELS.SS(8)$$

Where:
n: number of columns.
qc: Maximum allowable stress in the column.

$$qc = min(0.8;\ 2Pl *)(9)$$

Where Pl* is the equivalent limit pressure, deduced from the pressuremeter tests, Pl*=0.6 Mpa. qc =0.8 Mpa
q’u: Breaking stress of the ground before improvement under a centered load.
According to Fascicule 62 [1], the breaking stress of the soil is equal to:

$$qu' = kP\ .PL(10)$$

Where kp is the lift coefficient of sandy formations (kp=1).
q’u = 0.6 Mpa
{n. Scol. qc + [(Ss - n, Scol). q'u / 3]} / Ss = 2.50 bars >qELSthe lift condition is verified.

- Then, we calculated the settlement ws without treatment for qELS = 2.5bars, either on the basis of the Ménardpressuremeter method.

or using the FOXTA software, we found ws = 1.00 cm. Thus, Ks could be determined:

$$Ks = qELS / ws (11)$$

Where Ks is the stiffness of the soil.
Ks = 2.5 bar/cm

- The settlement equation for a column wcol with head stress qcol is:

$$wcol = \beta'.qcol.H / Ecol (12)$$

Where H is the level at which settlement is calculated;
β' is a ratio which shows that there is a distribution of the stresses of the column on the ground;
Ecol is Young's column modulus (Ecol = 60 MPa). In practice, H = min (2.5B; Lc) = min (5; 12) = 5m, is used since more than 85% of soil settlement occurs between 0 and 2.5B, and we retain: β' = 1 (no delivery). Thus, the stresses transmitted to the ground and to the columns, respectively qs and qcol, are determined on the basis of the following two equations:

- Equality of soil settlement and column settlement:

$$qs / Ks = qcol / Kcol \ (13)$$

With: $Kcol = Ecol / (\beta' * H)$ (14)

- Distribution of stresses between floor and columns:

$$\{n.Scol.qcol + [(Ss - n.Scol).qs]\} = qELS.Ss \ (15)$$

We obtained: qcol = 6.16 bars and qs = 1.28 bars
• at this stage, the stiffness of the structure "sole+column" assembly at the height in question:

$$K = \{[ks.(Ss - n.Scol)] + (n.kcol\ Scol)\} / (B\ L) \ (16)$$

K = 4.87 bars / cm
we could then deduce:

- Settlement after treatment at height H:

$$wsH = qELS / K \ (17)$$

lH = 0.52 cm

- -Final settlement after treatment:

$$wsf = wsH / 0.85 \ (18)$$

W=0.61cm
Finally, we checked that the values were still included in the pseudo-elastic domain of validity:
qs = 1.28 bars <kp .Ple/2 +q'0 = 3.00 bars
qcol = 6.16 bars <qa = 8.00 bars
wsf = 0.61 cm Admissible

The installation of 2 stone columns 0.8 m in diameter and 12 m in length under an insulated footing of (2 * 2) m2, has improved bearing capacity from 2.00 to 2.50 bars.

c-Evaluation of The Attenuation of Liquefaction by Stone Columns

Stone columns have a significant capacity to dissipate pore pressures. Unlike a drain, the stone column is made of a highly permeable compacted gravel material. Its high capacity to evacuate pore pressures results from its high permeability, and also the appearance during the earthquake of a strong hydraulic gradient, linked to the phenomenon of dilatancy of the gravel of the columns [12]. To assess the risk reduction of the liquefaction potential, we used the method of Seed and Booker [13] which is based on the dissipation of excess pore pressure [14]. According to the RPS 2000 version 2011 [3], to have a safety factor greater than 1.33, it is necessary to find a ratio rumax = U / v0'< 0.6 (U: Interstitial pressure, v0': Initial effective stress). This ratio (rumax) is deduced from Figure 2.
We have
Neq: Number of equivalent seismic cycles, defined in Table 3 (Neq = 8 for a zone of average seismicity).
Nl: Number of cycles leading to liquefaction, deduced from the nomogram in Figure 3 [2] (Nl=8).
Therefore Neq/ Nl = 1
We have also
a: Radius of the stone column, (a = 0.4 m).
b: Radius of influence of the column, for the square grid, b= 1.13 * I / 2, where I is the spacing, (I = 1.7m and b = 0.960 m).
Therefore a/ b = 0.42
For Tab which is a dimensionless parameter defined by the following formula:
Tab = $(Ks.td)/(mv.a^2.\gamma w)$
Where:
Ks: horizontal soil permeability (Ks=10-5 m/s).
td: Duration of the earthquake (td = 14s for an area of average seismicity).
γw: Water density.
mv: Soil compressibility mv = 1 / EoedBy correlationwith the results of pressuremeter tests
Table4 represents the results of the pressuremeter tests.
Eoed = EM/α where:
Eoed is the oedometric modulus.
EM is the pressuremeter module. α is the rheological coefficient of the soil

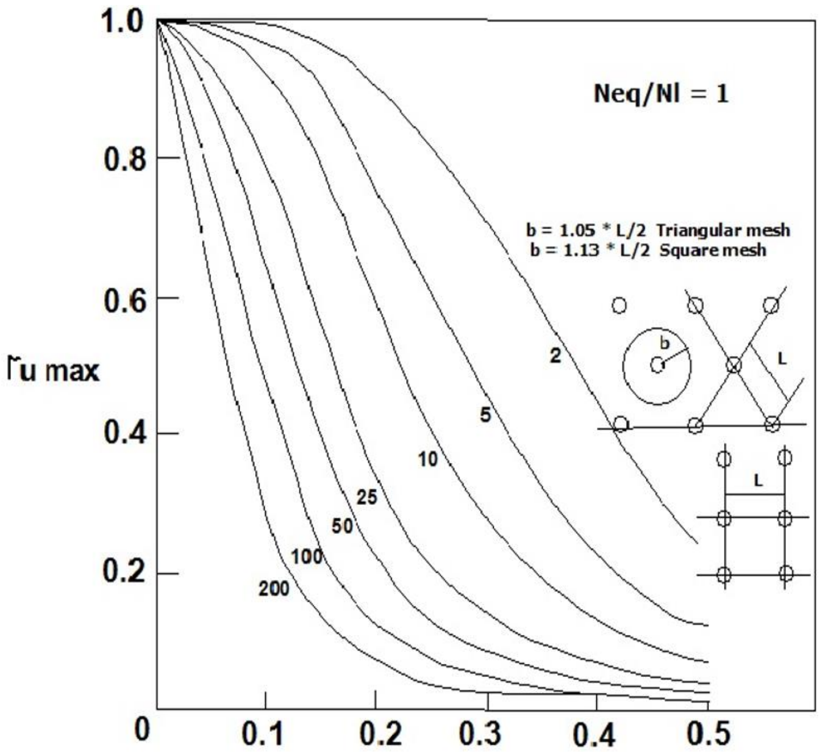

Figure 2.
Determination of a/b ratio (a = Drain radius and b = Half spacing).
Note: "*" Means multiplication.
Source: Booker, et al. [15].

Table3.
Number of equivalent cycles and duration of the earthquake according to the seismic zone.

Seismicity zone	Conventional magnitude	Number of equivalent earthquake cycles Neq	Duration of the earthquake td(s)
3 (Moderate)	5.5	4	8
4 (Average)	6.0	8	14
5 (Strong)	7.5	20	40

Source: Lambert [14].

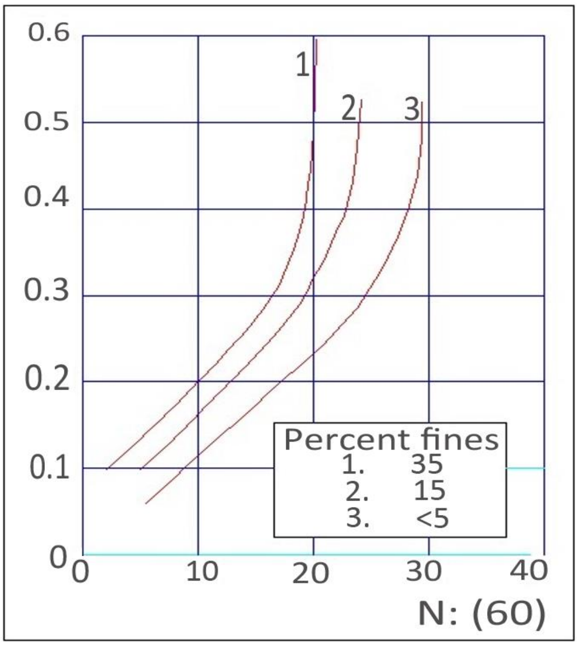

Figure 3.
Number of cycles leading the soil to liquefaction according to the percentage of fine.
Source: EN 1998-5 [2].

Table 4.
The pressuremeter module according to the depth in Mpa.

Depth (m)	Lithology	PL(Mpa)	EM(Mpa)
0.6	Alluvium and gravelly sand not very compact		
1	Coarse to medium sand becoming gravelly from 3.20 m	2.02	26
2.6		1.55	20.5
4		0.68	6.8
4.5			
5.4	Gravelly alluvium with sandy matrix	0.62	3
6			
7	fine sand	0.27	1.1
7.5			
8	Coarse gravelly sand		
8.3	Marl with fine sand of low consistency	0.76	9.6
10		0.76	7.3
11.6		1.27	8.5
12.5			
13	Pebbles and blocks with a sandy matrix	3.01	25.5
14.6		2.96	20.3

This method does not take into account the expansion of the column which further reduces the pore pressure .

Table 5.
Evaluation of liquefaction mitigation by stone columns for 10 SPT test

Borehole	Depth	FS	EM (Mpa)	ἀ	mv (Mpa-1)	Tab	ru max
SPT1	7	0.72	1.10	0.33	0.3000	0.29	< 0.6
	8	0.75	5.35	0.33	0.0617	1.42	< 0.6
	9	0.77	9.60	0.33	0.0344	2.55	< 0.6
	10	1.06	7.30	0.33	0.0452	1.94	< 0.6
SPT2	6	0.93	2.30	0.33	0.14	0.61	< 0.6
	7	0.86	1.10	0.33	0.30	0.29	< 0.6
	8	0.81	5.35	0.33	0.06	1.42	< 0.6
	9	0.78	9.6	0.33	0.03	2.55	< 0.6
	10	0.95	7.3	0.33	0.05	1.94	< 0.6
	11	0.93	7.92	0.33	0.04	2.10	< 0.6
SPT3	10	0.74	7.3	0.33	0.05	1.94	< 0.6
	11	0.80	7.92	0.33	0.04	2.10	< 0.6
	12	0.79	12.5	0.33	0.03	3.31	< 0.6
	13	1.04	25.5	0.33	0.01	6.76	< 0.6
SPT4	8	0.68	5.35	0.33	0.06	1.42	< 0.6
	9	0.57	9.60	0.33	0.03	2.55	< 0.6
	10	0.72	7.30	0.33	0.05	1.94	< 0.6
	11	0.89	7.92	0.33	0.04	2.10	< 0.6
	12	0.79	12.50	0.33	0.03	3.31	< 0.6
	13	0.76	25.50	0.33	0.01	6.76	< 0.6
	14	1.01	22.25	0.33	0.01	5.90	< 0.6
SPT5	6	0.73	2.3	0.33	0.14	0.61	< 0.6
	7	0.88	1.1	0.33	0.30	0.29	< 0.6
	8	1.06	5.35	0.33	0.06	1.42	< 0.6
	9	0.84	9.60	0.33	0.03	2.55	< 0.6
	10	0.69	7.30	0.33	0.05	1.94	< 0.6
	11	0.76	7.92	0.33	0.04	2.10	< 0.6
	12	0.77	12.50	0.33	0.03	3.31	< 0.6
	13	0.93	25.50	0.33	0.01	6.76	< 0.6
SPT6	6	0.92	2.3	0.33	0.14	0.61	< 0.6
	7	0.92	1.1	0.33	0.30	0.29	< 0.6
	9	1.20	9.60	0.33	0.03	2.55	< 0.6
	10	1.03	7.30	0.33	0.05	1.94	< 0.6
	11	0.76	7.92	0.33	0.04	2.10	< 0.6
	12	0.96	12.50	0.33	0.03	3.31	< 0.6

Borehole	Depth	FS	EM (Mpa)	ά	mv (Mpa-1)	Tab	ru max
	13	0.77	25.50	0.33	0.01	6.76	< 0.6
	14	0.72	22.25	0.33	0.01	5.90	< 0.6
SPT7	10	0.99	7.30	0.33	0.05	1.94	< 0.6
	11	1.16	7.92	0.33	0.04	2.10	< 0.6
	12	0.94	12.50	0.33	0.03	3.31	< 0.6
SPT8	7	0.79	1.1	0.33	0.30	0.29	< 0.6
	8	0.84	5.35	0.33	0.06	1.42	< 0.6
	9	0.68	9.60	0.33	0.03	2.55	< 0.6
	10	0.87	7.30	0.33	0.05	1.94	< 0.6
	11	0.85	7.92	0.33	0.04	2.10	< 0.6
	12	1.14	12.50	0.33	0.03	3.31	< 0.6
SPT9	5	0.82	5.6	0.33	0.06	1.48	< 0.6
	6	0.99	2.3	0.33	0.14	0.61	< 0.6
	7	0.77	1.1	0.33	0.30	0.29	< 0.6
	8	0.83	5.35	0.33	0.06	1.42	< 0.6
	10	0.57	7.30	0.33	0.05	1.94	< 0.6
	11	0.54	7.92	0.33	0.04	2.10	< 0.6
	12	1.07	12.50	0.33	0.03	3.31	< 0.6
	13	1.15	25.50	0.33	0.01	6.76	< 0.6
SPT1	4	1.15	6.8	0.33	0.05	1.80	< 0.6
	5	0.98	5.6	0.33	0.06	1.48	< 0.6
	6	0.88	2.3	0.33	0.14	0.61	< 0.6
	7	1.04	1.1	0.33	0.30	0.29	< 0.6
	9	1.10	9.60	0.33	0.03	2.55	< 0.6
	10	0.89	7.30	0.33	0.05	1.94	< 0.6
	11	0.69	7.92	0.33	0.04	2.10	< 0.6
	12	0.83	12.50	0.33	0.03	3.31	< 0.6
	13	0.85	25.50	0.33	0.01	6.76	< 0.6

According to the Table 5 above ru max is much less than 0.6. Therefore, the risk of liquefaction of the soils of the liquefiable depths is solved, and hence the effectiveness of the treatment. Consequently, a square mesh of stone columns spaced 2.5 m apart can eliminate the risk of liquefaction in this zone.

d- Verification of the Improvement of the Mechanical Characteristics of the Treated Soils

To check the state of compactness of the stone columns and the improvement of the mechanical characteristics of the treated soils, which is a function of the radius and the mesh of the columns, intra-column, inter-column and off-site CPT surveys were carried out in addition to the pressuremeter surveys.

d.1 Stone Columns

The CPT tests carried out intra-column show that the peak resistances measured are well above 10 MPa, the minimum values required according to D.T.U 13.2. At the level of some boreholes, the tip left the tested columns to seek an easier passage, this is due to the fact that the column is very compacted. The pressuremeter tests carried out on the columns tested also show that the limit pressures are well above the required value of 1.5 MPa. The objective of the loading test is to observe the full-scale settlement of a stone column to justify its behavior with respect to deformations under a vertical load. Four plate loading tests were carried out, the results show that under the test load which was 20 tonnes, the recorded settlements did not exceed 1 cm and the critical load deduced from the creep curves remained above 20 tonnes, indicating the performance of the stone columns. Consequently, the results obtained at the end of the loading tests are conclusive.

d-2 Inter-Column Ground

The correlative graphs between the inter-column and control cpt show that the mean of the improvement rates of the resistances at the QC tip (Figure 4) vary between 1.4 and 2.4%. The presence of gravelly horizons (pebbles) whose QC improvement rates remain below 1%. At the level of sandy horizons, the improvement rates were generally good and exceeded 1.5%.

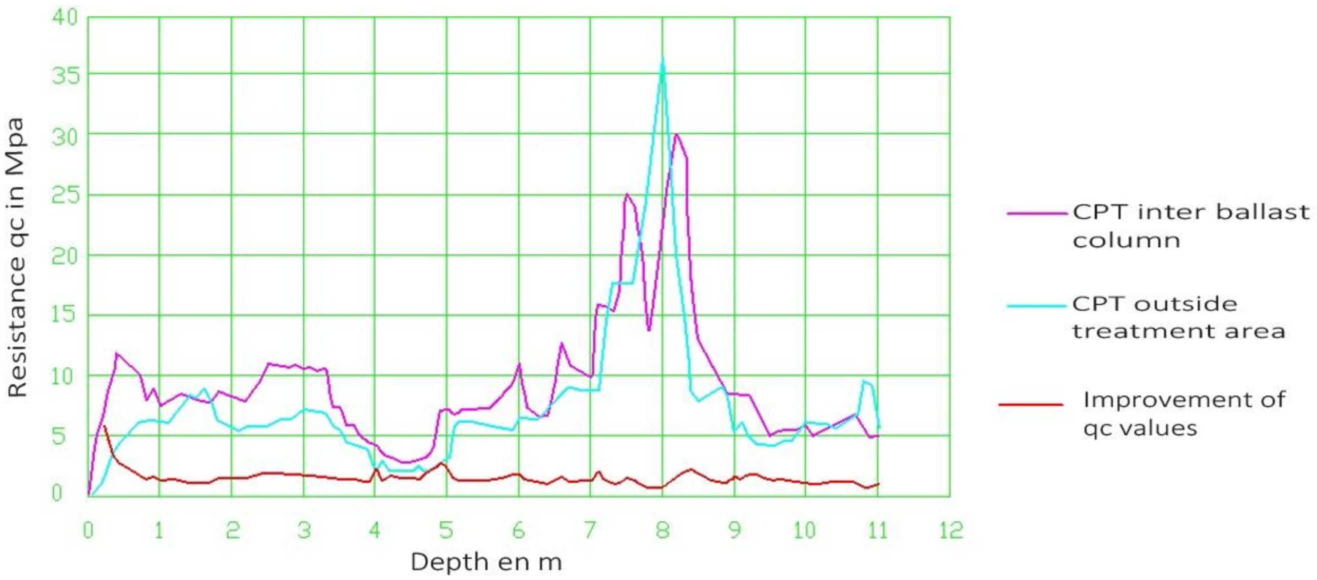

Figure 4.
Improvement rates of the resistances at the QC tip inter-column ground.

4. Analysis and Discussions

The evaluation of liquefaction potential indicates that sand and sandy marl between depths of 6 and 12 m, where the water table is reached, pose a risk of liquefaction for which soil treatment is required. The particle size analysis shows that vibro-compaction alone cannot be a treatment option. And hence, we opted for a treatment by stone columns, and to reduce the shear stress applied to the soil, to evacuate the pore pressures and to increase the compactness of the soil between the columns. The dimensioning allows us to conclude that the installation of 2 stone columns of 0.8 m in diameter and 12 m in length under an insulated footing of (2 * 2) m2, has improved the bearing capacity from 2.00 to 2.50 bars , and the settlement reduces from 1.00 cm to 0.61 cm, which has been verified by the loading tests after execution, and it also avoids the risk of soil liquefaction. The CPT tests carried out intra-column show that the peak resistances measured are well above 10 MPa, the minimum values required according to D.T.U 13.2. The pressuremeter tests carried out on the columns tested show that the limit pressures are well above the required value of 1.5 MPa. The correlative graphs between the inter-column and control cpts show that the resistance to the QC tip varies between 1.4 and 2.4%. In the presence of gravelly horizons (pebbles) the QC improvement rates remained below 1%. At the level of sandy horizons, the improvement rates are good and generally exceeded 1.5%.

5. Conclusion

The Souani area in the city of Al hoceima is characterized by the presence of sandy deposits. This study provides guidelines to address issues related to these soils, particularly the risk of liquefaction. Based on in situ SPT tests, it is confirmed that this area presents a risk of soil liquefaction at depths of up to 18 m. Due to the high proportions of fines in the soils, which is more than 14%, vibro-compaction alone cannot be used to treat this soil. Therefore, installation of stone columns 0.8m apart and spaced 2.5m up to the depth of 18m is mandatory. This treatment could ward off the liquefaction of the soil. This treatment improves the bearing capacity of the soil from 2.00 bar to 2.5 bar and reduces the settlement of the footings of constructions from 1.00m to 0.61 cm. Post-treatment tests (CPT, pressuremeter) show that the compactness and bearing characteristics of the stone columns meet the requirement. The improvement factor of the soil treated by stone columns depends on the network of columns installed (column geometry) and the characteristics of the filler materials. These columns have an overall effect on the reinforced soil. The effect of the mesh of the stone columns is superimposed on that of the vibrocompaction, during the construction of the stone columns, there is also an effect of vibrocompaction in the soil between the columns, which generates an improvement in Qc soil between-column by 1.5%. Knowing that this dimensioning method does not take this improvement into account (only relates to the improvement effect generated by the columns in the treated soil). Therefore, the characteristics of the treated soil and stone columns are taken separately, in other words the specific characteristics of the reinforced soil remained unchanged after the treatment.

References

[1] T. L. Youd and I. M. Idriss, "Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils," *Journal of Geotechnical and Geoenvironmental Engineering,* vol. 127, no. 4, pp. 297-313, 2001. https://doi.org/10.1061/(asce)1090-0241(2001)127:4(297)

[2] EN 1998-5, "(English): Eurocode 8: Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]. Retrieved from: https://www.phd.eng.br/wp-content/uploads/2014/11/en.1998.5.2014.pdf," 2004.

[3] RPS, "The RPS 2000 seismic building regulations, 2011 version, Department of quality and technical affairs, Ministry of Housing and Urban Policy in Morocco," Retrieved: www.academia.edu/33259147. 2000.

[4] H. B. Seed and I. M. Idriss, "Simplified procedure for evaluating soil liquefaction potential," *Journal of the Soil Mechanics and Foundations division*, vol. 97, no. 9, pp. 1249-1273, 1971. https://doi.org/10.1061/jsfeaq.0001662

[5] S. S. C. Liao and R. V. Whitman, "A catalog of liquefaction and non- liquefaction occurrences during earthquakes, department of civil engineering ". Cambridge, MA: Massachusetts Institute of Technology, 1986, p. 117.

[6] I. M. Idriss and R. W. Boulanger, "Soil liquefaction during earthquakes." Oakland, CA: Earthquake Engineering Research Institute, 2008, p. 261.

[7] I. M. Idriss and R. W. Boulanger, *SPT - based liquefaction triggering procedure, Centre for geotechnical modeling, department of civil and environmental engineering*. Davis, California: Universitu of California, 2010.

[8] I. Idriss and R. Boulanger, "Semi-empirical procedures for evaluating liquefaction potential during earthquakes," *Soil Dynamics and Earthquake Engineering*, vol. 26, no. 2-4, pp. 115-130, 2006. https://doi.org/10.1016/j.soildyn.2004.11.023

[9] B. Laurent, L. Philippe, P. Claude, and S. Bruno, *Soil improvement and reinforcement*. Le Moniteur, 2018.

[10] A. Dhouib and F. Blondeau, *Colonnes Ballastées - Implementation techniques, fields of application, behavior, justifications, lines of research and development*. Presses of the National School of Bridges and Roads, 2005.

[11] P. Aguado *et al.*, "Recommendations on the design, calculation, execution and control of stone columns under buildings and under structures sensitive to settlement," *French Review of Geotechnics*, vol. 136, pp. 71-86, 2011.

[12] M. R. Madhav and J. N. Arlekar, "Dilation of granular piles in mitigating liquefaction of sand deposits," presented at the in 12th World Conference Earthquake Engineering, 2000.

[13] H. B. Seed and J. R. Booker, "Stabilization of potentially liquefiable sand deposits using gravel drains," *Journal of the Geotechnical Engineering Division*, vol. 103, no. 7, pp. 757-768, 1977. https://doi.org/10.1061/ajgeb6.0000453

[14] S. K. Lambert, "Fondations spéciales, evaluation of the risk reduction of liquefaction by stone columns," in *Proceeding of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris*, 2013.

[15] J. R. Booker, M. S. Rahman, and H. B. Seed, "GADFLEA—A computer program for the analysis of pore pressure generation and dissipation during cyclic or earthquake loading," Rep. No. EERC 76–24, Univ. of California at Berkeley, Berkeley, Calif, 1976.