CASE STUDIES OF STONE COLUMNS IMPROVEMENT IN SEISMIC AREAS

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\textbf{Abstract:} Soil improvement with stone columns is often implemented in seismic areas in particular as a countermeasure against liquefaction. The efficiency of stone columns can be explained by the compaction effect during their execution, by the reduction of the seismic stress on the soil, and by their draining effect as well. A historical evolution of the first realisations is presented, followed by an inventory of the observed performance of such systems in seismic conditions. Then, recent applications of stone columns combined with pile foundations are presented.

\textbf{Key-Words:} seismic area, stone columns, liquefaction, case study, pile foundation.

1. Introduction

Seismic loadings on structures have not only for consequence direct effects due to the soil vibration, but also induced effects like soil liquefaction with a loss of soil rigidity and possible lateral flow of light tilted areas, which can spread far over the limits of the structure. The liquefaction phenomenon corresponds to a loss of soil resistance due to an increase of the pore pressure leading to a fall or to a short term total loss of effective stresses in the soil, the soil behaving thus as a liquid. This happens mainly in fine relatively homogeneous loose sands with a moderate permeability, but in some cohesive soils as well \cite{17}. Therefore the main countermeasures to reduce the liquefaction risk are a soil compaction, a soil drainage and a reduction of the seismic stresses in the soil. Different techniques of soil improvement can be used in seismic areas: ground mass improvement, for example vibrocompaction of dynamic compaction, soil reinforcement with granular columns like stone columns or rigid inclusions, including injection and deep soil mixing techniques. The advantage of stone columns bears on their simultaneous action on soil densification, on the stress reduction in the soil and on the drainage of the soil to be improved. Besides, they can be implemented in all soil types (Fig. 1), and show the specificity to maintain their integrity, with no risk of internal failure of the column because of their granular constitution, on the contrary to rigid columns.

\begin{figure}[h]
\centering
\includegraphics[width=0.5\textwidth]{figure1.png}
\caption{Application range of the deep vibratory techniques \cite{18}}
\end{figure}

In order to guarantee the soil resistance to liquefaction, the EN 1998-5 imposes a safety factor of 1.25 between the cyclic resistance ratio (CRR, shear resistance divided by initial effective vertical stress) and the cyclic stress ratio generated by the earthquake (CSR, cyclic shear stress divided by initial effective vertical stress). Domains with and without liquefaction risk can be defined depending on the compaction state, represented in general either by the number of blows in the Standard Penetration Test (SPT) or by the cone resistance in the Cone Penetration Test (CPT) (Fig. 2).
Figure 2. Liquefiability depending on normalized cone resistance and cyclic stress ratio (magnitude 7.5) [2]

Stone columns act, on the one hand, on the cyclic shear resistance. The increase of the soil compaction level due to the vibrations, with a ratio of 1 to 3, allows to move to the non-liquefiable domain for a given CSR (Fig. 2), more or less importantly depending on the soil type, on the power of the tool used and on the mesh area. The gravel installation with soil displacement leads to an increase of the horizontal stress in the soil. Inserting a non-liquefiable material with high shear resistance creates an increase of the CRR as well. On the other hand, similarly to the permanent loads, the stone column concentrates the seismic loads, reducing thus the stress on the soil itself, which corresponds to a CSR reduction. Finally, the stone column plays a draining role thanks to the high permeability of the gravel combined with an augmentation of the hydraulic gradient from the dilatancy effect appearing on the columns under a dynamic loading [13]. Calculation methods exist to take this drain effect into account [20]. In general, the risk of contact erosion by the surrounding fine soil is negligible [22]. The proportion of these different favourable effects of the stone column varies depending on the soil type. For example, in sandy lenses, the compaction will play the major role, whereas in a silty soil, the reduction of the liquefaction risk bears mainly on the stress diminution and on the draining effect. A realistic estimation of the performance of a stone column system implies a calculation method which couples the different effects [14, 2]. For seismic endangered zones, constructive measures have to be taken for the possible effects on the surrounding zones [2], with in particular an extension of the improvement around the structure of at least one half of the thickness of the liquefiable layer and at least one row. According to EN 1998-5, liquefiable soils have to be considered up to a depth of 15 m under the structure, even if the underlying soil in depth is likely to liquefy as well [6].

2. Historical evolution

The techniques aiming a reduction of the liquefaction risk have notably developed since the 1960s, in particular after the devastating earthquakes of 1964 in Alaska and in Niigata. The soil reinforcement technique with stone columns meshes developed at that time, parallel to an understanding of the mechanisms at stake in the liquefaction phenomenon, and seemed appropriate due to the simultaneous compacting and draining effects. Stone columns made it possible to extend the vibrocompacting effect to fine and silty sands, corresponding to the liquefiable soil types.

The first historical application of stone columns for an important project in highly seismic area, and one of the first uses of stone columns in the United States at all, was the realisation of the wastewater treatment plant of Santa Barbara in California in 1976 [15]. The structure had to stand on recent and loose liquefiable alluvial soils, made of interlayered clayey, silty and sandy lenses. The site shows a high seismicity linked to active faults located at a close distance, able to cause earthquakes of magnitude 7 with maximum accelerations at the site of 0.25 g. The alternative solution was a pile foundation, which was quickly excluded because the piles can lose their skin friction resistance in case of soil liquefaction. The solution of a stone columns square grid has been chosen in order to limit the settlement under service loads and to eliminate the liquefaction risk. Because no case studies on the performance of stone columns under seismic loading were available at that time, a large-scale test program has been carried out in the scope of the Santa Barbara project [8].

Dobson (1987) compiled major sites in which stone columns have been implemented against liquefaction in the following years, between 1976 and 1982. The compacity increase necessary to justify the system has been
proven by means of SPT or CPT tests. In these first applications, only the densification effect has been taken into account in the design [1].

3. Observed seismic performance

The seismic performance of stone columns has been observed in particular during the earthquakes presented in Tab. 1, completed with the earthquake of Nisqually in 2001 with a magnitude of 6.8.

Table 1. Main earthquakes with observation of stone column performance [11]

<table>
<thead>
<tr>
<th>Year</th>
<th>Earthquake</th>
<th>No. Sites</th>
<th>Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>1999</td>
<td>921 Ji-Ji, Taiwan</td>
<td>TBD</td>
<td>7.6 MW</td>
</tr>
<tr>
<td>1999</td>
<td>Kocaeli, Turkey</td>
<td>5</td>
<td>7.4 MW</td>
</tr>
<tr>
<td>1997</td>
<td>Kagoshimaken Hokkaido, Japan</td>
<td>49</td>
<td>6.9 MW</td>
</tr>
<tr>
<td>1995</td>
<td>Hyogoken Nambu (Kobe), Japan</td>
<td>4</td>
<td>6.9 MW</td>
</tr>
<tr>
<td>1994</td>
<td>Sendai Hanaka Oki, Japan</td>
<td>3</td>
<td>7.5 JMA</td>
</tr>
<tr>
<td>1994</td>
<td>Hokkaido Toho Oki, Japan</td>
<td>4</td>
<td>8.1 JMA</td>
</tr>
<tr>
<td>1994</td>
<td>Northridge, California</td>
<td>5</td>
<td>6.7 MW</td>
</tr>
<tr>
<td>1993</td>
<td>Hokkaido Nansen Oki, Japan</td>
<td>4</td>
<td>7.8 JMA</td>
</tr>
<tr>
<td>1993</td>
<td>Kushiro Oki, Japan</td>
<td>5</td>
<td>7.8 JMA</td>
</tr>
<tr>
<td>1989</td>
<td>Loma Prieta, California</td>
<td>12</td>
<td>6.9 MW</td>
</tr>
</tbody>
</table>

The behaviour of stone columns during the Loma Prieta earthquake has been described by Mitchell and Wentz in [16]. Three structures founded on hydraulic fills of loose to medium dense sands located at approx. 60 km of the epicentre have been studied: a dental clinic and a pier approach area at Treasure Island, and an extension esplanade at Richmond. These structures had been designed for seismic accelerations of 0.35 and have been subjected to 0.11 to 0.16 during the earthquake. No liquefaction occurred over the depth improved by stone columns, whereas at greater depths and in the direct surroundings of the structures soil cracks and sand boils have been noticed, reflecting a liquefaction of these zones. Even if the magnitude and the duration of the event remained quite moderate in that case, it is obvious that for stronger earthquakes, the reinforcement with stone columns would considerably reduce the possible damages.

Hayden and Baez [12] cite a project reinforced with stone columns which underwent the Northridge earthquake in 1991. It consisted in a set of elevated railroad tracks at 50 km distance of the epicentre, where no damages have been noticed. Hausler and Koelling [10] studied the performance of stone columns during the Nisqually earthquake in 2001. The sites on stone columns did not suffer any damages, on the contrary to adjacent unimproved areas showing sink holes, cracks and sand ejection (Fig. 3).

Figure 3. Sand boils and cracks in non-improved zones during the Nisqually earthquake in 2001 [10]

Hausler and Sitar [2001] studied 90 sites improved by different methods whose efficiency could have been assessed during 14 different earthquakes. The performance of stone columns compared to other improvement methods proves to be good, with a higher increase of compacity measured by SPT tests for methods combining vibration and compaction than for methods using compaction only like preloading (Tab. 2). The rare cases of damages can be explained by soils tending to lateral flowing, by an insufficient improvement depth or by a too small improvement extent around the structure [11]. In those cases, the surrounding soil imposes additional stresses (pore pressure, loss of lateral confinement) to the improved zone. It seems more
reasonable to treat the whole liquefiable depth in many cases. The experience proves that an improvement extent around the structure between half and the whole liquefiable thickness is necessary.

Table 2. Seismic performance of different soil improvement methods [11]

<table>
<thead>
<tr>
<th>Method</th>
<th>Performance (Acceptable/ Unacceptable)</th>
<th>Average Increase in $N_{eq}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Densification through vibration and compaction</td>
<td>26 / 5</td>
<td>11</td>
</tr>
<tr>
<td>Sand compaction piles</td>
<td>15 / 0</td>
<td>5</td>
</tr>
<tr>
<td>Deep dynamic compaction</td>
<td>11 / 0</td>
<td>13</td>
</tr>
<tr>
<td>Vibrood/vibroflotation</td>
<td>7 / 1</td>
<td>8</td>
</tr>
<tr>
<td>Stone columns</td>
<td>5 / 0</td>
<td>5</td>
</tr>
<tr>
<td>Preloading</td>
<td>1 / 1</td>
<td>n/a</td>
</tr>
<tr>
<td>Compaction grouting</td>
<td>1 / 0</td>
<td>n/a</td>
</tr>
<tr>
<td>Timber displacement piles</td>
<td>1 / 0</td>
<td>n/a</td>
</tr>
</tbody>
</table>

Disipation of excess pore water pressure
- Gravel drains: 5 / 0, 7
- Sand drains: 5 / 0, 9
- Wick or paper drains: 2 / 0, n/a

Restraining effect through inclusions
- Deep soil mixing: 4 / 1, n/a
- Diaphragm walls: 0 / 1, n/a

Stiffening through chemical or cement addition
- Jet grouting: 5 / 0, n/a
- Chemical grouting: 1 / 0, n/a

4. Combined use with pile foundations

For some structures requiring a pile foundation to take the permanent loads, the liquefaction problem must be solved as well. The liquefaction of the soil would namely lead to a loss of soil resistance, reducing considerably the mobilizable pile skin friction and tip resistance. In addition to this loss of bearing capacity, the possible lateral spreading creates supplementary horizontal stresses on the piles. The different possible failure mechanisms are presented in Fig. 4.

Figure 4. Failure mechanisms of pile foundations by ground liquefaction [4]

Hausler and Sitar [11] present a structure on piles having showed damages during the Kobe earthquake. The concerned Mikage Hama gas tank itself, founded on piles up to 12 m depth under the liquefiable soil, did not settle. However, the surrounding soil settled of 35 to 60 cm with horizontal deformations of up to 2 m, leading to a dislodging of pipe lines from the tank with major leakage.

Those different liquefaction problems around piles can among others be solved by inserting stone columns between the piles or by improving the soil around the piles. Different application examples are presented in this section, but without observed seismic efficiency at the moment. The first presented example is a tank of liquefied natural gas in Vancouver [5]. Due to increased requirements of safety against ground liquefaction, the foundation had to be designed again for an increased base rock acceleration. The proposed solution consisted in improving the soil up to width of 25 m around the tank in order to ensure a satisfying horizontal support of the piles and to avoid a lateral spreading of the whole foundation.
In order to understand better the interaction between piles and stone columns in a liquefied soil, the large-scale testing project TILT (« Treasure Island Liquefaction Test ») has been carried out in 1999 [3], in a soil mainly made of silty sands. A group of four steel pipe piles and a single pipe concrete (CISS) pile have been loaded horizontally in a cyclic way and then subjected to a dynamic loading by controlled blasting in the soil, before and after the stone column installation. The tests showed a decrease of the excess pore pressures due to the explosion with columns compared to the case without columns, and a sharp dissipation of these pressures in the first seconds after blasting. In the unimproved case, water and sand boils have been observed at the surface, whereas no sign of liquefaction have been noticed in the improved case.

This pile-soil improvement combination against liquefaction has been implemented for example in the large-scale project of the national stadium Khalifa Bin Zayed in Abu Dhabi in the United Arab Emirates (Fig. 5).

![Figure 5. Position of stone columns between the piles](image)

The soil of the thermal power plant in Anpara, made of fly ash and silty clays imposed a soil improvement to increase the vertical and horizontal bearing capacity of a pile foundation. Because of the seismic risk in the area and the liquefiable nature of the soil, an improvement with stone columns has been chosen in order to solve at the same time the problem of bearing capacity under the permanent loads and the liquefaction risk (Fig. 6).

![Figure 6. Foundation layout with stone columns](image)

5. **Conclusions**

The case studies of use of stone columns in seismic areas between the beginnings in the 1970s until today show a very good efficiency of such systems against liquefaction. In the earthquakes investigated, the stone columns made it possible to avoid a loss of soil resistance as well as a lateral spreading of the surrounding zones, while maintaining their integrity, on the contrary to rigid columns for which a structural capacity check has to be done. The efficiency of stone columns can be explained by their capacity to reduce the load on the soil, by the compaction effect during their execution and by the draining role they play. A calculation considering all the favorable effects may make an optimized design possible in the future, knowing in particular that their relative importance vary depending on the soil type. More recently, combined systems of pile foundations with stone columns in between have been executed in order to ensure a good pile confinement in case of an earthquake.

6. **References**


