Ground improvement to reduce the liquefaction potential around pile foundations

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Summary
Experience shows that buildings on conventionally designed pile foundations can be substantially damaged by earthquakes. Reason for this is often a liquefaction of the soil around the piles. With modern methods of ground improvement in the surrounding area, like deep vibration or deep soil mixing methods, this liquefaction risk is crucially decreased. Some international examples are presented and recommendations for practice are given.

1 Introduction
Ground liquefaction can occur due to strong vibrations, which are caused by earthquakes. It arises particularly in loose to medium dense, homogeneous fine sands with low permeability. By the vibrations of the earthquake an excess pore water pressure is created in saturated soils, the ground loses its shear strength and the granular structure breaks down. Apart from the loss of vertical bearing capacity of the piles within the liquefied soil, enormous horizontal forces can be exerted on the pile foundations by the liquefaction. If piles are situated in a loose liquefaction-endangered layer and embedded into a dense bearing layer, they can be over-stressed or buckle due to large bending moments. Other possible failure mechanisms are presented in figure 1.

The liquefaction risk around the pile foundations can be reduced crucially by modern ground improvement methods. In the context of this publication, after a short review of design processes on projects with liquefaction risk, international projects are presented where the liquefaction risk around the pile foundations was reduced by ground improvement, like vibro replacement, vibro compaction and deep soil mixing.
2 A short view of the planning process of projects in areas with liquefaction risk

2.1 Determination of the liquefaction potential

In a first step for the estimation of the liquefaction potential, the cyclic (seismic) stress ratio $SSR = \tau_h / \sigma_{v\theta}$ is determined dependent on the depth (with an assumed earthquake acceleration).

The most common estimation can be made with the following formula (Seed, Idriss, Arango, 1983):

$$\frac{\tau_h}{\sigma_{v\theta}} = 0.65 \cdot \frac{a_{\text{max}}}{g} \cdot \frac{\sigma_{v\theta}}{\sigma_{v0}} \cdot r_d$$  \hspace{1cm} (1)

with

\begin{align*}
\tau_h & = \text{seismic shear stress [kN/m}^2]\] \\
\sigma_{v\theta} & = \text{total ground pressure [kN/m}^2]\] \\
\sigma_{v0} & = \text{effective ground pressure [kN/m}^2]\] \\
a_{\text{max}} & = \text{maximum earthquake acceleration [m/sec}^2]\]
\end{align*}
0.65 = average value of acceleration in relation to the maximum $a_{\text{max}}$

$g$ = gravity acceleration [m/sec²]

$r_d$ = depth dependent reduction value [~ 1 – 0.012 $z$]

$z$ = depth [m]

### 2.2 Determination of the cyclic resistance ratio

In a second step the cyclic stress ratio determined above is compared to the cyclic resistance ratio. The cyclic resistance ratio is the stress ratio, at which liquefaction occurs. This is known from the statistic evaluation of soundings in seismic zones.

The first evaluations in such way were based on standard penetration tests, similar to the chart in figure 2.

![Figure 2](Correlation between liquefaction resistance and SPT (Earthquake magnitude M = 7.5))

Figure 2: Relationship between liquefaction resistance and number of blows N of the standard penetration test in sandy soil with varying fines content (Seed, De Alba, 1986).

Recent diagrams prefer correlating the tip resistance $q_c$ of the cone penetration test (CPT) with the cyclic stress ratio, see figure 3.

Safety against ground liquefaction is given if the cyclic resistance ratio is larger than the cyclic stress ratio.
2.3 Detailed review of pile foundations

For pile foundations further considerations can be made during planning, like

- the interaction between building and subsoil
- the load bearing capacity and the settlement behavior of the foundation (e.g. consideration of the loss of the skin friction by liquefaction or negative skin friction by ground settlements)
- the displacements of the soil surrounding the foundations and the resulting additional forces and moments

Therefore in particular analytic and numeric methods are used (like e.g. LPILE, GROUP, SHAKE, FLAC, etc.), which are not described in detail here.

In case the proof of safety against liquefaction around the pile foundations cannot be provided in the design, ground improvement methods are specifically suitable, in order to increase the shear strength of the subsoil and to minimize the risk of the ground liquefaction.
2.4 Consideration of ground improvement design by means of an example with vibro replacement

By means of the vibro replacement method (Priebe 1998) it is shown how the risk of liquefaction potential can be minimized.

Fig. 4 yields a ground improvement factor for vibro replacement with simplifying assumptions. This factor depends both on the ratio of the grid area $A$ to the area of the column $A_c$, and on the friction angle $\phi_c$ of the column material.

![Figure 4: Design chart by Priebe, 1998 to calculate the soil improvement factor for settlements](image)

The reciprocal value of this improvement factor corresponds to the relationship of the remaining stress $p_s$, which acts on the ground between the vibro replacement columns, and the total stress $p$ without vibro replacement. Thus the presentation of the so-called reduction factor $p_s/p = \alpha$ results in figure 5.
With the plausible assumption that the area of the gravel columns and also the loads taken by it do not contribute to liquefaction, this reduction factor $\alpha$ can equally be used to reduce the seismic stress ratio created during the earthquake. Thus the density requirements of the subsoil exposed to liquefaction are reduced accordingly. Figure 6 shows, how the required cone penetration resistance decreases due to the reduced seismic stress ratio $\alpha \cdot \text{CSR}$.

![Diagram showing reduction factor for the seismic stress ratio using vibro replacement (Priebe 1998)](image1)

Figure 5: Reduction factor for the seismic stress ratio using vibro replacement (Priebe 1998)

![Diagram showing correlation between liquefaction resistance and CPT (Earthquake magnitude $M = 7.5$)](image2)

Figure 6: An example of reduction of the required cone penetration resistance $q_c$ due to ground improvement (vibro replacement)
3 Ground improvement methods to reduce the liquefaction potential around pile foundations

As already mentioned, ground liquefaction arises due to earthquakes in particular, however not exclusively, in loose to medium dense homogeneous fine sands with low permeability. The range of these soils, which are effected mainly by liquefaction, is represented in figure 7 (hatched area). Further the application ranges of vibro compaction and vibro replacement methods are included. While in clean sands both ground improvement methods are applicable, vibro replacement method is more effective with increasing fines content.

Based on recent information in clayey and silty sands the risk of ground liquefaction in such soils must be examined as well. In accordance with Eurocode EC 8 part 5 (DIN EN 1998-5) the risk of liquefaction should not be neglected in sands with a clay content of up to 20% (plasticity index PI>10) or with a silt content of up to 35% (SPT-ratio of $N_1$ (60) > 20).

Even cohesive soils are endangered of liquefaction. As an example the so-called „Chinese criteria“ is shown (see fig. 8), which makes a principle classification into potentially liquefiable and non-liquefiable soils depending on the water content and the liquid limit.
Figure 8: „Chinese Criteria“ (modified by ASTM definitions) for ground liquefaction by Perlea, 2000

Beside the vibro compaction and vibro replacement methods mentioned above a further ground improvement measure is the deep soil stabilization (Deep Mixing Method, briefly DMM), which is applicable in all soil types.

For special applications also earthquake drains can be used for the reduction of the liquefaction potential in soils.

4 Reference Projects for Vibro Replacement

4.1 Full-scale test: Reduction of the ground liquefaction by means of vibro replacement around a pile foundation (Scott et al., 2002)

As co-operation between the University OF California and the Brigham Young University a full-scale test, the so-called Treasure Island Liquefaction test (TILT), was executed in 1998/1999, in order to investigate the behavior of horizontal loaded piles in a liquefiable ground. The tests were executed first without ground improvement before and after liquefaction and afterwards with ground improvement (vibro replacement) before and after ground liquefaction. The ground liquefaction was produced by controlled explosions in the sand layer. The subsoil consisted of predominantly loose homogeneous silty sand up to a depth of 6m, underneath was soft fat clay with a thickness of approximately 10m. The ground-water level was 1.5m under ground surface. The horizontal forces on the 12 to 14m piles were applied by jacking a 4 pipe pile group (diameter 324mm) and a cast in place concrete pile (cast-in-steel-shell CISS, diameter 600mm) horizontally against each other by means of hydraulic presses.
The results can be summarized as follows:

- Due to the installation of stone columns the top soil layers have been improved (increase of the cone resistance from 4 MPa on average up to over 20 MPa). The conclusion was that the increase of the cone resistance goes along with a corresponding reduction of the liquefaction potential of these top sand layers.

- After installation of the stone columns the dissipation rate of the excess pore water pressure released by explosion (with ground liquefaction) was clearly increased.

- A certain time after the explosion the horizontal stiffness of the foundation system improved with stone columns was higher (factor 2.5 to 3.5) than without ground improvement.

4.2 Risk of the ground liquefaction in the Fraser Delta / British Columbia: Protection of a liquid gas tank by means of vibro replacement (Chamboss, 1983)

The Fraser River delta located in the south of Vancouver in British Columbia is one of the most strongly endangered seismic zones in Canada. Generally there are several meters of clay, silt or peat on top of up to 45m thick sand layers. Beneath silt, clay and glacial deposits follows. The in-situ rock is located in 200m depth or below it.

B.C. Hydro built a liquid gas tank with a capacity of 17 million cbm gas on 15m long wooden piles (grid spacing 0.9m) in the Fraser River Delta. The piles driven into medium dense sand were used primarily as compaction piles, i.e. the pile heads are not embedded into the foundation. A 50cm crushed rock layer and a 2m thick sand and gravel backfill separate the circular foundation of the tank from the upper edge of the piles (see fig. 9).

Due to increased requirements of safety against earthquakes and ground liquefaction the client and planner required a subsequent densification of the sand package surrounding the pile foundation by means of vibro replacement. Thus the pile foundation of the tank should be supported horizontally, in order to enhance the safety in case of an earthquake.

Thus on a width of 25m and a depth from 16.5 to 23.5m stone columns had to be installed subsequently around the tank. 38,000 tons of gravel material were installed in 1100 stone columns in a grid of 2.5m. Standard penetration tests (SPT) and cone penetration tests (CPT) confirmed the densification success.
From special interest at that time was the realization that also in silty sands (with an average fines content of 17%), an improvement of the SPT-blowes around the factor 3 was reached by vibro replacement. Thus it became obvious that also these grounds (silty fine sands) can be improved economically with vibro replacement.

4.3 Khalifa Bin Zayed National Stadium, Abu Dhabi: Vibro stone columns around bored piles (2009)

The Khalifa Bin Zayed National Stadium in Abu Dhabi was built on cast in-situ concrete piles with diameters 900mm and 1200mm and a pile spacing of 2.7m. At the time of the construction work the ground consists of a 2.4m thick fill (sand and gravel) placed as working platform, followed by 6-7m loose to medium dense silty, partly clayey sands, beneath gypsum-, sand- and claystone. In accordance with the official regulations the stadium was to be designed for an earthquake of magnitude 6.8 with a peak earthquake acceleration (PGA) of 0.22g. In order to avoid ground liquefaction of the top soil
layers in such case, ground improvement became necessary around the piles (extent of the ground improvement, see fig. 10, typical grid of the vibro replacement columns around the piles, see fig. 11).

![Stadium layout with pink colored range of the ground improvement](image1)

Figure 10: Stadium layout with pink colored range of the ground improvement

Since the subsoil predominantly consists of silty/clayey sands (the fines content well over 10%), the vibro replacement method (vibro stone columns) was used. Apart from the increase of shear resistance, increased load-bearing capacity and increase in soil density, a high permeability of the stone columns was recognized as large advantage as well. In case of an earthquake with accompanying excess pore water pressures it could be assumed that a significantly faster reduction of the water pressure and thus avoidance of ground liquefaction takes place.

![Typical arrangement of the vibro stone columns (crosses) around the bored piles (circles), with marked test field for load test](image2)

Figure 11: Typical arrangement of the vibro stone columns (crosses) around the bored piles (circles), with marked test field for load test
The assessment of the ground liquefaction risk and the design of vibro replacement were based on 74 cone penetration tests (see example calculation, fig. 12).

![Calculation of vibro replacement by Priebe (1998)](image)

Figure 12: Calculation of vibro replacement by Priebe (1998)

In fig. 13 the cone penetration resistance of the unimproved soil \(q_{c,\text{actual}}\) and the calculated penetration resistance required for the safety against ground liquefaction are presented both with \(q_{c,2}\) and also without stone columns \(q_{c,0}\). As shown, the risk of ground liquefaction in the depth between 1.0m and 2.7m can be avoided by the installation of stone columns.
Based on the site investigations the diameter of the vibro stone columns was set to 0.80m, the spacing to 1.35m and the depth to 6-7m.

The work sequence performed by Keller / UAE was as follows:

- Vibro stone columns had to be installed after the pile installation
- Level measurement of the working platform for the installation unit (vibrocat)
- Execution of pre-treatment CPT’s (1 per 1000m²)
- Setting out installation points of the vibro stone columns in a square grid with spacing of 1.35m
- Due to the partially very dense top layer the upper 2m of the installation points had to be predrilled and further 2m of soil had to be loosened
- Installation of vibro stone columns by means of vibrocat, including electronic recording of depth, time, energy consumption, etc.
- Execution of post-treatment CPTs, plate load tests and large-scale load tests in order to confirm the design requirements

In only 2.5 months 50,000 m³ gravel were installed in 13,000 nos. vibro stone columns.
4.4 Anpara Thermal Power Plant, India: Improvement of fly ashes by vibro stone columns (V. R. Raju, 2011)

The unavailability of suitable construction sites for extension of existing power plants makes it necessary to consider deposits of fly ashes as foundation subsoil. The Anpara Thermal Power Plant in Uttar Pradesh is an example where the extension of the plant had to be founded on those fly ash deposits. The thickness of the deposits varied from 3m to 13m, the in-situ density was loose to medium dense. Beneath the fly ash dense sandy silt or hard clayey silt was found. A typical sectional profile is shown in figure 14.

During the design phase a potential risk of liquefaction was identified and comprehensive investigations with respect to the effectiveness of ground improvement techniques were done. As a result the installation of vibro stone columns by means of the bottom-feed method was planned, see figure 15.

Figure 14: Typical subsoil profile at the Coal Handling Plant location / Anpara

Figure 15: Schematic of stone column installation (Dry bottom feed method)
The foundation design of bored piles required a design lateral load capacity of 7 tons (ultimate load 21 tons). After the installation of bored piles and vibro stone columns with two different spacings and diameters within a test field, lateral load tests were conducted on these two grid patterns. The layout of both test fields is presented in figure 16.

![Figure 16: Layout of both test fields](image1.png)

Figure 17 presents the observed load displacement curves of the test piles of both test fields, and the execution of the load test.

![Figure 17: Load displacement curves and execution of the load test](image2.png)

At both test fields the horizontal displacements were in the required range (<5mm at a load of 7 tons). A typical foundation layout is shown in figure 18. Figure 19 displays the construction site during installation of the bored piles and vibro stone columns.
In summary the vibro stone columns combine the following advantages:

- The in-situ density of the fly ash at the construction site was varying and the allowable soil pressure for shallow foundations was insufficient (< 100 kN/m²). By means of vibro stone columns with embedment into the stiff subsoil beneath the fly ash, the bearing capacity of the subsoil was improved considerably.

- In the case of an earthquake there was the potential risk of liquefaction as the present SPT – blow counts from 3 to 8 were below 20, which is the corresponding requirement according to Indian Standard. In comprehensive field tests the most adequate subsoil improvement technique and the optimum grid were determined in order to fulfil the design requirements.

- The vibro stone columns were arranged within the pile foundation in a manner, that the in-situ density of the fly ash was improved and thus the lateral capacity of the pile foundation as well.
5 Reference Projects for Vibro Compaction

5.1 Soil improvement by means of Vibro Compaction: Fort Calhoun Nuclear Station
(Fischer et al., 1972)

The construction of the Nuclear Station Fort Calhoun adjacent to the Missouri River in Nebraska started in 1968. It was founded on open steel pipe piles with a diameter of 50cm. The subsoil consisted mainly of fine sands with varying silt content in the upper part and with depth increasing in-situ density (from loose to medium dense). In the depth of about 20m rock is encountered. When pile installation works were under way, more rigorous requirements by the regulatory agencies made a revision of the seismic design criteria necessary. Consequently the sand between the piles had to be densified up to 20m depth in order to mitigate the risk of liquefaction. As by pile driving alone (open pipe piles) no considerable densification was observed, the subsoil had to be densified by other means. Within a test program the following methods were investigated: Sand piles and wooden piles (compaction piles), densification by blasting and vibro compaction. Whereas the compaction piles could not be installed up to the required depth (due to increasing in-situ density of the fine sands) the densification by blasting resulted to be unreliable and uneconomic. Finally vibro compaction was chosen as the most economic and effective soil improvement method. During a series of further tests a) the effectiveness and consistency and b) the optimum spacing (considering the given pile spacing) were determined for the densification target.

As result of the investigations it was found that by means of the vibro compaction method both clean and silty sands could be densified successfully up to a depth of 20m. For the successful densification of the silty sands a tighter grid, or the addition of imported clean sand, resulted beneficial.

5.2 Golden Ears Bridge in British Columbia, Canada (Naesgaard et al, 2008)

In the south-western British Columbia / Canada a six-lane cable-stayed bridge, the Golden Ears Bridge was built. It crosses the Fraser River near Vancouver (see figure 20). The main bridge (river crossing) is 968m long and the total length, including approaches is 3.6 km. The bridge structure is situated in an area of seismic activity as at the west coast of Canada and the north-western USA the eastwards drifting Juan de Fuca plate subducts the westwards drifting continental North American plate.
In the area of the four river piers fluvial sands of up to 20m to 35m thickness with high liquefaction potential were deposited. These sands are underlain by more than 100m thick soft to stiff silts and clays with occasional sand interlayers. A simplified subsoil profile is presented in figure 21.

The bridge design required vibro compaction in the area of the main bridge piers, which are founded on bored piles (diameter 2.5m, length 75 to 85m), in order to enhance the stiffness of the foundation system in case of an earthquake with accompanying ground liquefaction. A typical cross section through the main bridge piers outlining the extent of the planned and executed vibro compaction is shown in figure 22.
Due to the presence of a sand-silt mixture at the southern river bank vibro stone columns were installed adjacent to the foundation piles of the southern main pier. In direct vicinity to a settlement- and vibration-sensitive water main crossing the construction site, vibro stone columns were replaced by earthquake drains manufactured by NILEX.

5.3 The Guggenheim Abu Dhabi/ Saadiyat Island: Vibro compaction prior to pile installation (2010)

The Guggenheim Foundation will construct their biggest museum of modern and contemporary art in Abu Dhabi. The over 30,000 square-foot large museum will be built on the Saadiyat Island according to the design by the American architect Frank Gehry. The subsoil at the construction site consists of loose to medium dense sands to a depth of about 4m with embedded thin silt and clay layers (0.10-0.30 m thick) beginning at 1.50m depth. Below the depth of 4m the in-situ density of the sands increases from dense to very dense. The exploration depth of cone penetration tests was approximately 5 to 7m.
In order to eliminate the risk of ground liquefaction during an earthquake event of magnitude 6 with a peak earthquake acceleration of 0.20g, the reclaimed sand masses had to be densified by means of vibro compaction up to a depth of 5m. According to the recommendations by the Geotechnical Consultant the subsoil improvement had to be executed prior to the installation of the pile foundation. Figure 23 shows the layout of the building (left) and a detail of the densification grid with the planned pile locations (right).

In order to determine the required spacing between the densification points field trials with 3 different spacings (triangular grid), 3.5m, 4m & 4.5m, were performed. The trials included as well the execution of 33 “Pre-compaction” cone penetration tests (CPT’s) and 57 “Post-compaction” CPT’s.

The interpretation of the field trials, including liquefaction analyses, led to the following conclusions:

- With a grid spacing of 3.5m and 4.0m the required cone resistance could be achieved up to a depth of 5m in all compactable layers
- Although the required cone resistance could not be achieved in the non-compactable layers (layers with a high fines content), even there the risk of liquefaction could be reduced successfully

Based on the above mentioned conclusions the densification was performed with a grid spacing of 4m. The densification success was proofed by means of pre-treatment and post-treatment CPT’s (see comparison in figure 24, the required cone tip resistance is drawn as vertical line).
The results of two exemplary liquefaction analyses (before and after execution of the densification) are compared in figure 25. It can be seen that the liquefaction risk in the depth from 2.5m to 3.6m (horizontally hatched) was eliminated successfully.

Figure 25: Liquefaction analysis (left before densification, right after densification)
6 Reference Projects for deep soil mixing (Deep Mixing Method, DMM)

The effectiveness of the deep mixing method (DMM) in respect to the elimination or reduction of liquefaction was confirmed during the earthquake in Kobe in the year 1995 (Topolnicki, 2004). A hotel building under construction founded on bored piles on a reclaimed sand area stayed intact because the foundation piles were protected by means of DMM-elements against large-scale ground liquefaction and corresponding flow pressures and additional forces. On the other hand neighboured structures without DMM suffered large displacements towards the waterfront.

Especially the cell- or honeycomb-like arrangement of DMM-elements around piles is regarded as particularly effective, see figure 26. The single cells isolate and enclose the soil being liquefied during an earthquake event and prevent lateral spreading. In addition the cell-like arrangement reduces settlements as well and increases the safety against ground failure (Topolnicki, 2004).

![Figure 26: Example DMM – Application for the reduction of soil liquefaction, (Topolnicki, 2004)](image)

Babasaki und Suzuki (1996) reported about further examples of executed DMM in the harbour area of Tokyo in order to reduce the liquefaction risk.
7 Recommendations

- By means of the presented ground improvement methods the risk of ground liquefaction around pile foundations can be reduced. Thus a more economical design of pile foundations is possible.

- With the help of the diagrams of Priebe (1998), the planner has an instrument at hand to consider ground improvement quantitatively, particularly to reduce the seismic resistance ratio due to the application of vibro replacement and vibro compaction around pile foundations.

- While vibro compaction is suitable in coarse-grained soils, vibro replacement is applicable in both fine-grained and mixed soils.

- By means of the deep mixing method (DMM) honey-comb like DMM elements can be installed virtually in all soil types around piles in order to isolate individual soil blocks, and thus minimizes the effects of soil liquefaction.

- Earthquake Drains can be installed in the immediate vicinity of settlement- and vibration-sensitive structures or supply lines.

- In some cases the presented ground improvement methods can be performed even subsequently (seismic retrofit).
8 Literature

Boulanger, R. W., Kutter, B. L., Brandenberg, S. J., Singh P., Chang D., [2003]. „Pile foundations in liquified and lateral spreading ground during earthquakes: centrifuge experiments & analyses“., Bild 1-1, Seite 1-4, Univeristy of California


Raju, V. R., [2001]. “Ground Improvement Using Vibro Techniques in Fly Ash Deposits”, in National Conference on Recent Advances in Ground Improvement Techniques, CBRI Roorkee, India
