State of Practice in Soil Liquefaction Mitigation and Engineering Countermeasures

Emilio M. Morales, MSCE[1]
Mark K. Morales, M.Sc.[2]

Summary

The threat of Soil Liquefaction is all too real and the damage wrought to Dagupan City and other areas due to liquefaction during the 1990 Luzon Earthquake indicate the need to provide engineered responses to mitigate or eliminate the threat.

With the anticipated build up in private infrastructure and development, the availability of cheap land is becoming scarcer and scarcer and therefore, focus is being directed into the development of marginal lands which invariably would involve some risks due to potential for liquefaction and other geotechnical concerns.

Particularly in the Philippines which is in a very active Seismic Zone, with very long coastlines with marine sedimentary deposits and inland alluvial valley deposits, potentially liquefiable loose to very loose granular soil deposits are prevalent.

This paper discusses the phenomenon of soil liquefaction, the causative mechanisms and the *"State of the Art"* approaches to determining liquefaction susceptibility of a specific soil deposit and the factor of safety.

This is followed by current *"State of Practice"* discussion addressing anti-liquefaction counter measures and mitigation methodologies available to engineers and developers.

[1] MSCE major in Geotechnics and Structures, Carnegie Mellon University, Pittsburgh, PA., Chairman, PICE Geotechnical Specialty Division., Principal, EM2A Partners & Co.

[2] M.Sc. Master of Science major in Earthquake Geotechnical Engineering, University of California – Berkeley, CA. Managing Director , Philippine GEOANALYTICS Inc.

contact : www.pgatech.com.ph

1.0 INTRODUCTION

1.1 General

Soil liquefaction is a sudden loss in strength in loose to very loose saturated granular soils due to ground shaking followed by a rapid increase in pore pressure. The ground shaking, which is normally due to earthquakes or significant horizontal shearing and excitation of the loose to very loose soils, momentarily causes dislodgement of the precarious grain to grain contact of the individual soil grains.

A different phenomenon on soft to very soft cohesive soils, which has been wrongly attributed as soil liquefaction in the past, is another mechanism caused by repeated cyclic shearing of the soils. Particularly in very sensitive soils, the cyclic disturbance causes a significant loss in shear strength which could result in instability or bearing capacity failures. This second phenomenon is not addressed in this paper as it is a totally different failure mechanism with the same causative or triggering events.

Rapid increases in porewater pressure normally accompany this ground shaking. Due to the dislodgement, the superimposed weight on the ground is momentarily transferred to the porewater because the soil loses its strength due to loss of grain to grain contact. This momentary transfer further increases the porewater pressure in the saturated zone further buoying up the already dislodged soil grains. Buoyancy causes the total collapse of the soil structure resulting in a "*liquefied mass*" which does not possess any shear strength or load carrying capacity.

Thus, the loads (structures) imposed on the soil before the liquefaction which originally was deemed "*stable*" momentarily loses the soil support leading to partial collapse or tilting to total collapse.

The following effects of Liquefaction can occur in a vulnerable site when liquefaction is induced by significant ground shaking:

- Lateral Spreading from Liquefaction. Lateral deformation induced by earthquakes is discussed below.

- Lateral Deformation. The occurrence of liquefaction and its associated loss of soil strength can cause large horizontal deformations. These deformations may cause failure of buildings, sever pipelines, buckle bridges, and topple retaining walls.

Three types of ground failure are possible. Flow failures may occur on steep slopes.

Lateral spread may occur on gentle slopes.

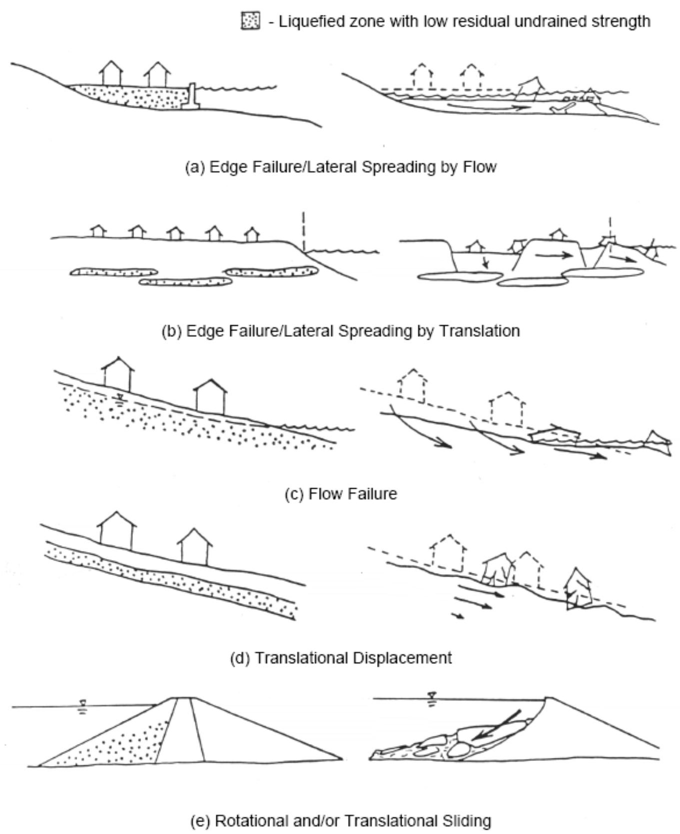


Fig. 44: Schematic Examples of Liquefaction-Induced Global Site Instability and/or "Large" Displacement Lateral Spreading

Figure 1.1 Examples of Lateral spreading due to Liquefaction. [3]

The third type of failure involves ground oscillation on flat ground with liquefaction at depth decoupling surface layers. This decoupling allows rather large transient ground oscillation or ground waves.[3]

In the past, as in the present, empirical and semi-empirical methods have been used in order to assess the liquefaction susceptibility of a site. This ranged from the use of comparison charts of characteristic grain size envelopes of sites worldwide that have liquefied in the past *(see Figure 3.1.1.)* on which the characteristic grain size of a specific site is superimposed, the use of rule of thumb checks, to the development of the Cyclic Resistance ratio (CRR). Even in the latter procedure, which is now universally accepted, recent developments in the understanding of liquefaction has resulted in significant changes in our understanding of this phenomenon in soils, its assessment as well as the feasible countermeasures to mitigate or reduce the effects on civil engineering structures.

It is the purpose of this paper to look into possible liquefaction mitigation technologies and discuss their effectiveness.

2.0 BACKGROUND ON LIQUEFACTION

Liquefaction is sudden loss of soil strength due to flotation of the individual soil grains from excess pore pressure and ground shaking during an earthquake.

However, before Liquefaction can occur the following conditions need to be satisfied which according to *Seed* [4] are:

- **Soil-type** - Soils with 50% or more of their grain size in the range of 0.02mm to 0.2mm are potentially liquefiable when saturated.

- **Intensity of Ground Pressure** - To initiate liquefaction local ground acceleration greater than 0.10g is required.

---

[3] US DOD NAVFAC DM 7.4 "*Soil Dynamics and Special Design Aspects*"

[4] Seed & Idriss:" Simplified Procedure for Evaluating Soil Liquefaction Potential" ***Journal of ASCE SM9 September 1971.***

- **Initial Confining Pressure** - The stress required to initiate liquefaction increases with confining pressure.

- **Duration of shaking** - It is necessary for the shaking to continue for some time *(a characteristic of large earthquakes).*

Liquefaction associated failure may be of the following types:

- Tilting due to instability
- Direct settlement due to loss of bearing capacity
- Uplift due to buoyancy effects
- Translation of structure

3.0 LIQUEFACTION ASSESSMENT

Most of the discussions in this Section were lifted from a "*State of the Art"* paper by *Seed et al* [5]

3.1 Analysis of Liquefaction

3.1.1 Empirical Correlations

Empirical correlations were based essentially on comparison of Grain size distribution of the site to the grain size envelope of sites that have liquefied in the past worldwide. This follows the work of *Nishida, Fitton and others* as well as recorded liquefaction at *Turnagain Heights* in Alaska.

If the grain sizes of the target site fall within the envelope of grain sizes that have liquefied in the past, then most likely the site will also experience liquefaction given an earthquake large enough to cause shearing and dislodgement of the loose to very loose sands.

A sample of this procedure is shown below and is used still to gage susceptibility to liquefaction in conjunction with other methods.

---

[5] R. B. Seed ***"Recent Advances in Soil Liquefaction Engineering-a Unified and consistent Framework"*** 26th Annual ASCE Los Angeles Geotechnical Spring Seminar.

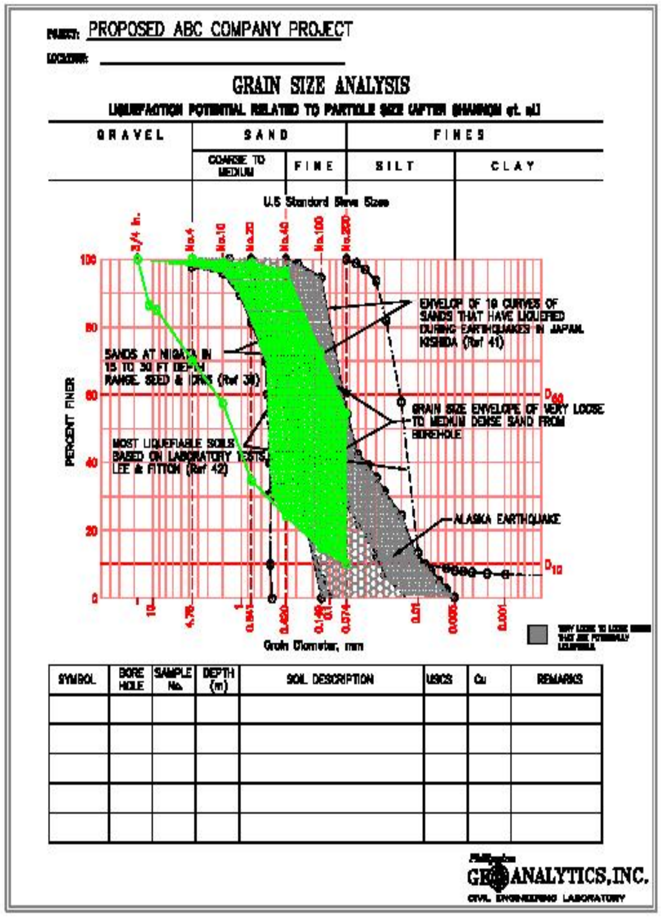


Fig. 3.1.1 - Comparison of Grain Size with Envelope of Grain Size that Liquefied in the past.

*3.1.2 The "***Simplified Procedure***" by Seed and Idriss*

Analytical Evaluation of liquefaction potential of a site is based originally on the pioneering work by *H Bolton Seed* and *Idriss* (1971) The "simplified procedure" originally developed involves the calculation of the Factor of Safety obtained by determining the Cyclic Resistance Ratio and Cyclic Stress Ratio of the site soils. The method has been modified and improved by several researchers. The current "simplified procedure" calculates the factor of safety, *FS*, against liquefaction in terms of the cyclic stress ratio, *CSR* (the demand), and the cyclic resistance ratio, *CRR* (the capacity), according to the formula:

$$FS = \left(\frac{CRR_{7.5}}{CSR}\right) MSF \cdot K_{\sigma} \cdot K_{\alpha}$$

where:

$CRR_{7.5}$ is the cyclic resistance ratio for magnitude 7.5 earthquakes, *MSF* is the Magnitude scaling factor, $K\sigma$ is the overburden correction factor, and $K\alpha$ is the correction factor for sloping ground.

CSR is estimated using the Seed and Idriss (1971) equation multiplied by 0.65:

$$CSR = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right)r_d$$

where:

> *amax* is the peak horizontal acceleration at the ground surface generated by the Earthquake,
>
> *g* is acceleration due to gravity,
>
> σ_{vo} and σ'_{vo} are the total and effective overburden stresses, respectively, and
>
> r_d is the stress reduction coefficient.

Other than the purely empirical grain size comparisons, the three commonly used methods to evaluate the liquefaction resistance, *CRR*, *Gutierrez Ref* [6] are:

1) Using the Standard Penetration Test (SPT),
2) Using the Cone Penetration Test (CPT), and 3) Using Seismic Shear wave velocity

Associated uncertainties in the development of probabilistic methods for liquefaction risk analysis based on the "*simplified"* method are:

1) the uncertainty in demand particularly the maximum acceleration amax and the earthquake magnitude Mw , required to estimate the magnitude scaling Factor MSF and

---

[6] M. Gutierrez, J. M. Duncan, C. Woods and M. Eddy " ***Development of a Simplified Reliability-Based Method*** **for Liquefaction Evaluation***" Civil and Environmental Engineering Virginia Polytechnic Institute & State University*

2) the uncertainty in the capacity CRR . For CRR, the uncertainties are due to natural variability of the soil and geotechnical properties, in-situ testing procedures, and most importantly the *simplified* method. *Gutierrez* [4]

Recent researches into this field have resulted in further refinements in the procedure particularly in both the *"deterministic"* and "*Probabilistic"* determination of liquefaction potential.

In a recent groundbreaking publication by *Raymond Seed et al* known as *the* Queen Mary Paper," ref[3] refinements in the procedure over that of the "*simplified" Seed (Senior)* procedure have been proposed.

New models presented and described in this specific research paper deal explicitly with the issues of:

(1) Fines content (FC),
(2) magnitude-correlated duration weighting factors (DWF_M), and
(3) Effective overburden stress (K_σ effects),

and they provide both

(1) An unbiased basis for evaluation of liquefaction initiation hazard, and
(2) Significantly reduced overall model uncertainty.

3.1.3 Influence of Fines Content and Plasticity

The fines content (% *passing No 200 sieve*), more specifically Plasticity of these fractions greatly influences the susceptibility to liquefaction.
The chart below is the recommendation from the paper by *Seed et al* [3] regarding the influence of the fines content, more specifically the effects of its Liquid Limit LL and Plasticity Index PI on the *liquefiability of soils.*

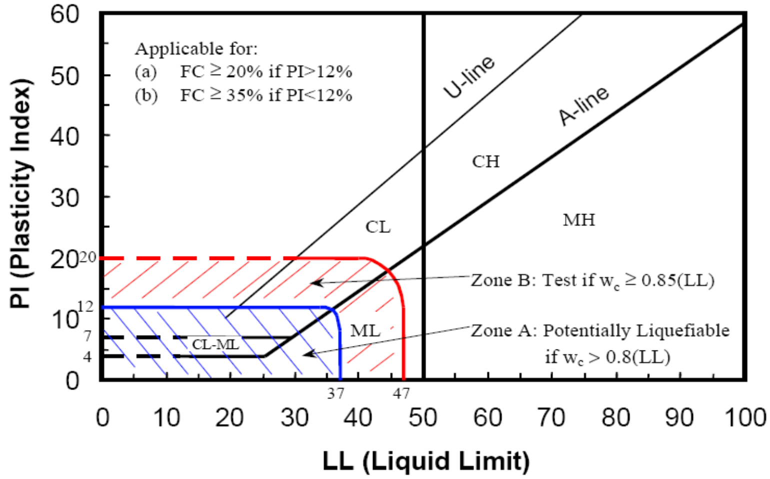


Fig 4: Recommendations Regarding Assessment of "Liquefiable" Soil Types

For soils with sufficient fines content that the Fines separate the coarser particles and control overall behavior:

(1) Soils within Zone A are considered potentially susceptible to "classic" Cyclically induced liquefaction,

(2) Soils within Zone B may be
Liquefiable, and

(3) Soils in Zone C (not within Zones A or B) are not generally susceptible to "classic" cyclic liquefaction, but should be checked for potential sensitivity (loss of strength with remolding or monotonic accumulation of shear deformation).

It has been found out that for soils with sufficient fines content FC, the characteristics of the fine fractions greatly influences susceptibility to cyclically induced Liquefaction.

3.1.4 Magnitude Correlated Duration Weighting DWFm

Both the probabilistic and "deterministic" (based on PL=20%) new correlations presented in Figures 10 and 11 are based on the correction of "equivalent uniform cyclic stress ratio" (CSReq) for duration (or number of equivalent cycles) to CSRN, representing the equivalent CSR for a duration typical of an "average" event of MW = 7.5. This was done by means of a magnitude-correlated duration weighting factor (DWF_M) as:

$$CSRN = CSR_{eq,M=7.5} = CSR_{eq,\,M=M} / DWF_m \text{ -> (a)}$$

This duration weighting factor has been somewhat controversial, and has been developed by a variety of different approaches (using cyclic laboratory testing and/or field case history data) by a number of investigators.

The Chart below shows the Duration Weighting Factor DWFm with a value of 1.0 for an earthquake Magnitude of 7.5. Thus for M > 7.5 the DWFm is less than 1.0 resulting in a higher CSRN in equation (a) above.

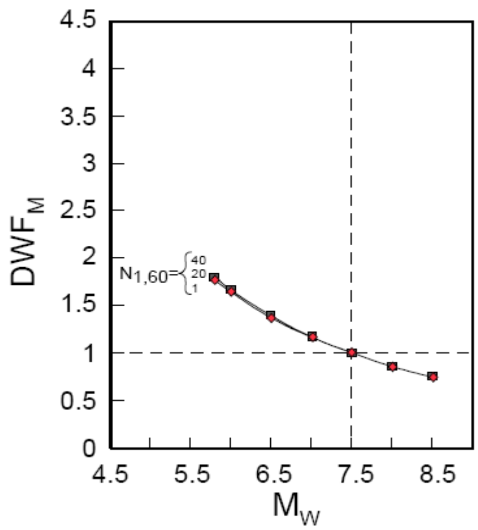


Fig. 13: Recommended Magnitude-Correlated Duration Weighting Factor as a Function of $N_{1,60}$

3.1.5 SPT Based Triggering Correlations

One of the more important contributions of the *Queen Mary Paper* is in the improvement of the SPT based correlations which has reduced the uncertainty in the use of such charts in the past. For one, Key elements in the development of this new correlation were:

(1) Accumulation of a significantly expanded database of field performance case histories,
(2) Use of improved knowledge and understanding of factors affecting interpretation of SPT data,
(3) Incorporation of improved understanding of factors affecting site-specific ground motions (including directivity effects, site-specific response, etc.),
(4) Use of improved methods for assessment of in-situ cyclic shear stress ratio (CSR),
(5) Screening of field data case histories on a quality/uncertainty basis, and

(6) Use of higher-order probabilistic tools (Bayesian Updating).

The charts below are the recommended procedure for determining Probabilistic and Deterministic Cyclic Stress Ratio (CSR) from the corrected and normalized SPT Nvalues. The probabilistic chart shows a family of curves based on Probability values (P_L), while for the Deterministic Chart; the family of lines represents different Fines content (FC) values. The solid data points represent correlated "liquefied" zones while the unshaded data points represent sites that have not *liquefied.*

However, before these could be applied, corrections on the SPT values need to be made as follows:

- Correction for Hammer Energy
- Correction for Rod Length
- Correction for Overburden stress
- Procedural corrections

The discussion of the foregoing effects is outside the scope of this paper. However, the reader is referred to the paper by *Seed et al* [5].

The downloadable *.pdf* version can be downloaded at : ***www.pgatech.com.ph***

4.0 LIQUEFACTION COUNTERMEASURES AND MITIGATION OF EFFECTS

4.1 General

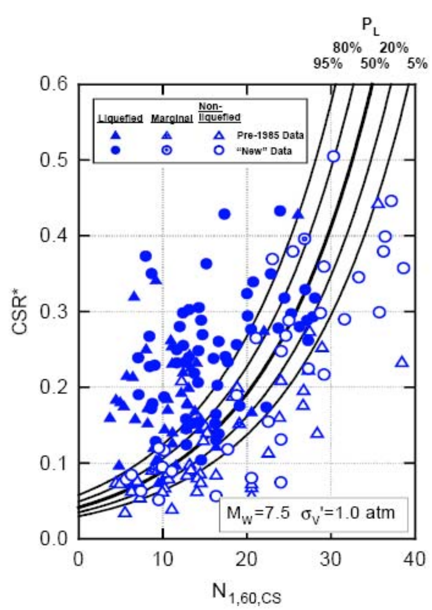


Fig. 16: Recommended "Probabilistic" SPT-Based Liquefaction Triggering Correlation (For M_W=7.5 and σ_v'=1.0 atm)

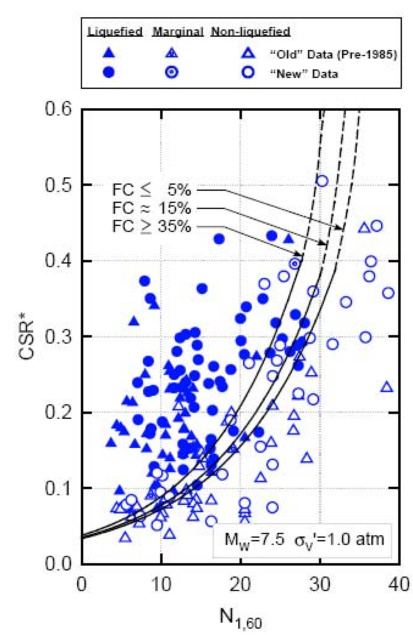


Fig. 17: Recommended "Deterministic" SPT-Based Liquefaction Triggering Correlation (For M_W=7.5 and σ_v'=1.0 atm) with Adjustments for Fines Content Shown

There are several procedures to mitigate or eliminate the harmful effects of liquefaction ranging from hard responses to simple avoidance.
These are:

- **Site Selection** - Potentially liquefiable areas can be identified and avoided. However, as premium lands become scarcer, marginal lands become attractive and thus this solution may not be commercially acceptable to developers.
- **Use of piling** to bypass the potentially liquefiable zones. This is the brute force solution. Piling would need to be designed for the unsupported length equivalent to the liquefied depth and for potential negative skin friction from clay layers overlying liquefiable zones. Detailing would also be under Seismic Zone 4 Conditions.
- **Chemical or Cement Injection grouting** to solidify the liquefiable soils . The permeability of the target soils should be determined to assure proper grout dispersion. Injection points may be numerous as grouting pressure can not be boosted or hydro fracturing can result.
    - *Joosten* 2 Part Process
    - Portland cement Injection.

However, the *State of Practice* has evolved through the years, to make available cost effective liquefaction mitigation technologies
that could be effectively used in countering liquefaction or in preventing the build-up of the critical conditions before set up of liquefaction.

Among these are:

- **Ground Densification** - The liquefiable loose to very loose grounds can be densified to the desired density to eliminate its susceptibility to liquefaction.

    - Compaction Piling / Resonant Column
    - Dynamic Compaction
    - Vibratory Methods
    - Stone Columns / Vibroflotation
    - Rammed Aggregate Piers

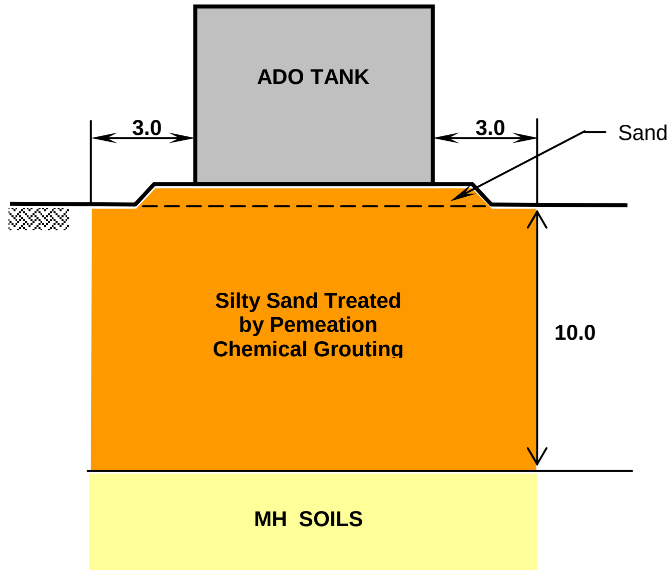


Figure 4.1 - Showing Ground Improvement Through Permeation or Chemical Grouting

- **Pore water Relief-** pore pressure buildup during the initiation of liquefaction can be prevented by rapid drainage.

  - Prefabricated Vertical Drains / Sausage Drains
  - Granular Piles

- **Compaction Grouting -** is defined as the staged injection of low slump (less than 3 inches) mortar-type grout into soils at high pressures (500 to 600 pounds per square inch), is used to densify loose granular soils. At each grout location a casing is drilled to the bottom of a

  previously specified soil target zone. Compaction grout is then pumped into the casing at increments of one lineal foot. When previously determined criteria are met such as volume, pressure, and heave, pumping will be terminated and the casing will be withdrawn. The casing will be continuously withdrawn by one foot when it meets previously determined criteria until the hole is filled. [3]

4.2 Ground Improvement Procedures

4.2.1 Strength Increase of Loose Sand Deposits with Vibratory Densification Procedures

Ground Densification procedures have the beneficial effect of improving the ground through densification and also through reinforcement of very poor clayey soils and to some extent in the acceleration of the consolidation process through *radial drainage* into the permeable granular columns. In addition, the densified granular columns serve to carry the major part of the load because of its relatively very high stiffness compared to the surrounding matrix soil.

The strength gain of the soil with time is also one of the beneficial effects.

In addition, performance of granular piles in the *Hyogen-Nambu (Kobe) Earthquake* has shown that areas improved with Granular piles did not fail during the liquefaction event, whereas surrounding areas that were not improved showed significant damage and collapse of structures. In this case, the granular piles serve as *Chimney drains* to relieve the pore pressures.

But of more critical importance is the almost immediate strength gain through reinforcement of the weak subsoils by densified columnar elements. The failure plane or the slip circle has to pass through and cut through the relatively very dense granular materials before it can propagate any further. In effect, the factor of safety is enhanced by the reinforcing effect of the Granular columns with large Angle of internal friction.

4.2.2 Generic and Proprietary Ground Densification using Vibratory Method

4.2.2.1 Compaction Piling/Resonant Column

Compaction piling, using steel rigid steel retractable mandrels, are driven at regular intervals or spacing in triangular or square array, within the liquefiable soils. The mandrel is then withdrawn and the hole is backfilled with sand. The mandrel is then redriven and retracted until full treatment depth has been completed.

4.2.2.2 Resonant Column Apparatus

This is a proprietary German technology which drives a steel probe attached to a vibratory equipment which induces vibration through the probe. The vibrating frequency is chosen at or near the *Resonant Frequency* of the soil to be densified. Thus, resonance builds up and the loose soils vibrate in themselves causing a denser packing to be achieved. The target density or N_{value} is controlled through the spacing of the insertion of the probe.

4.2.2.3 Dynamic Compaction

Dynamic compaction, developed by the late *Louis Menard* and originally marketed as Dynamic consolidation became popular in the seventies. The procedure involves the dropping of a heavy tamping weight over a free fall height of greater than 10 meters in order to cause shock waves The effective depth of treatment depends on the energy of the falling weight as it impacts on the ground. Empirically, the effective depth **De** of treatment is given as:

$$\mathbf{De = f \sqrt{W*H}}$$

Where:

D=Effective depth of Treatment ft
f= a constant depending on the
soil (normally ½)
W=Impact weight kips
H=freefall height

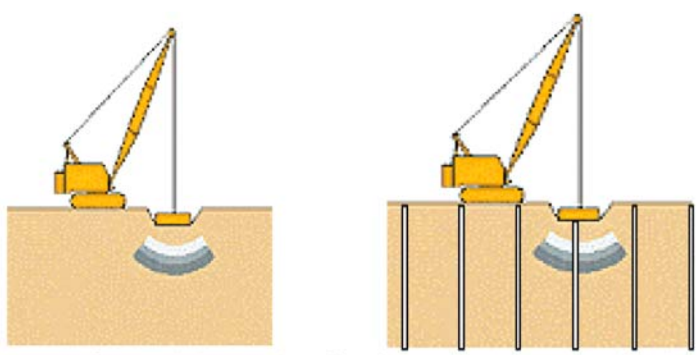
In order to avoid impact energy losses, a single line crane must be used to lift the weight. Thus, relatively heavy lift capacity cranes are needed to lift the weight due to lack of mechanical advantage derived from multiple pulley systems.

The depth of treatment based on the largest system ever built, is approximately 8.0 meters. Beyond this depth, energy from the surface is dissipated resulting in significantly reduced compaction.

Figure 4.2.2.1 Mammoth DC Equipment

Several drops are needed to finish one location with the interval between drops governed by the pore pressure dissipation.

Treatment procedures [7] would be different for various types of soils. PVD's would need to be installed in clays and impermeable soils to aid in drainage and rapid pore pressure decay.



a) DC - sand sites b) DC - silty sand sites (with wick drains)

[7] Nashed, R. " *A Design Procedure for Liquefaction Mitigation of Silty Soils using Dynamic Compaction*"

4.2.2.4 GEOPIER Impact Piers®

The Impact Pier® is a Proprietary Technology developed by *Geopier Corporation* [8] similar to stone columns in some respects but achieving higher aggregate column stiffness due to the patented beveled tamper foot. The Impact System uses vertical displacement Rammed Aggregate Piers (RAPs) to reinforce good to poor soils, including loose sands, silts, mixed soil layers including clays, uncontrolled fill and soils below the ground water table.

The installation process displaces soil during installation and utilizes vertical impact ramming energy to construct vertical displacement RAPs, which exhibit high strength and stiffness. The RAP procedure is designed to provide total and differential settlement control and increase bearing support.

The cavity is created to full depth by pushing a specially designed mandrel and tamper foot using a relatively large static force augmented by dynamic impact energy. This method eliminates spoils as all penetrated spoils are displaced laterally. A sacrificial cap prevents soil from entering the tamper foot and mandrel.

After driving to design depth, the hollow mandrel serves as a *funnel* for the placement of aggregate. The aggregate is placed inside the mandrel and the mandrel is lifted, leaving the sacrificial cap at the bottom of the pier. The tamper foot is lifted approximately three feet and then driven back down two feet, forming a one-foot thick compacted lift. Compaction is achieved through static force and dynamic impact energy from the hammer.

The Impact hammer blows densify aggregate vertically and the patented 45° beveled tamper foot forces aggregate laterally into cavity sidewalls. This results in effective coupling with surrounding soils and Settlement control and strength and stiffness. The lateral compactive energy results in *prestressing* and *Prestraining* the matrix soil.

---

[8] www.geopiers.com

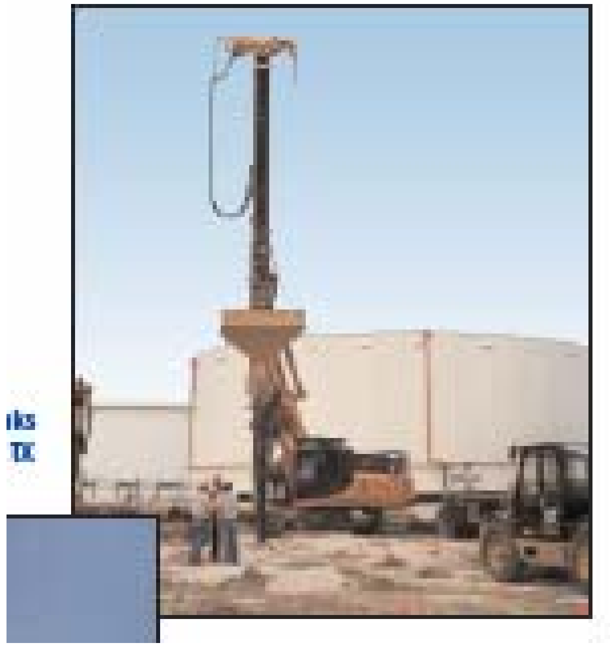


Fig 4.1 – Impact Pier Equipment

The RAP can develop Friction Angles of 42 to 48 depending on the aggregate and confining matrix soil.

Fig 4.2 - Conventional GEOPIER Installation Equipment

4.2.2.5 Vibrodensification or Vibroflotation®

Vibrodensification using proprietary technologies consists of introducing a rigid vibratory steel mandrel into the granular soil to be densified by vibratory excitation. This method is used primarily for densifying clean granular Cohesionless soils. The action of the vibrator usually used in conjunction with water jetting momentarily reduces the intergranular friction of the loose sand grains causing these to assume a denser state due to vibratory excitation. Granular material *(sands and gravels)* is dropped down the annular cavity between the mandrel and soil.

However, care must be used in order to prevent uplift or heaving of previously installed columns.

The granular materials are dropped and vibrated by a vibratory hammer. The resulting vibration results in densification of the coarse grained materials and the surrounding loose sand layers. Although the surrounding very soft clays would not generally densify, the densified sand columns will act as vertical reinforcement to increase the *composite* shear strength of the sub seabed clay soils. In addition, the sand columns will serve as vertical drains to accelerate the consolidation process by reducing drainage paths and providing for increased pore pressures momentarily during installation as to accelerate the drainage process further.

The accelerated consolidation will also allow for the accelerated development of increased shear strengths. In thick very poor cohesive soils, Vibroflotation may not be as effective as bulging of the vibroflot pile may result. This should be avoided as significant load reduction due to bulging could occur.

Vibroflotation operates effectively within a certain Grain Size Envelope as shown below:

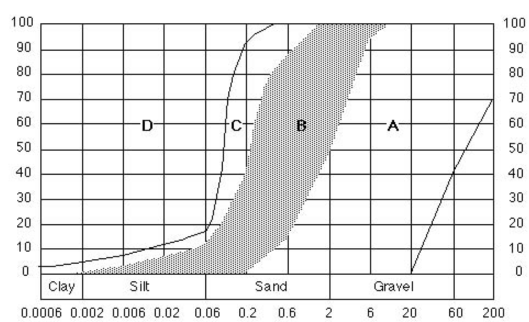


Figure 4.2.2 Grain Size Envelope Showing Applicability of Vibroflotation (from Vibroflotation Website)[9]

[9] www.vibroflotation.com

Figure Picture Showing Vibroflotation Equipment

4.2.2.6 Vibro Replacement or Stone Columns

MIL-HDBK-1007/3

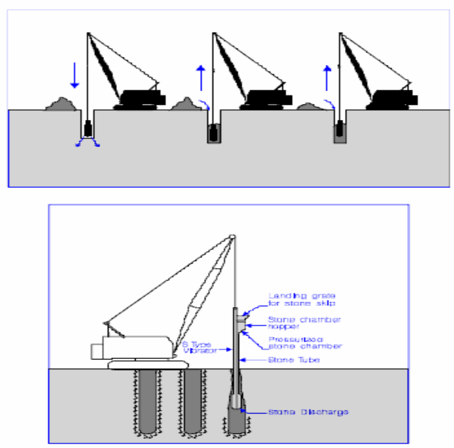


Figure 35
Installation of Stone Columns
120

Vibro replacement or Stone columns, uses columns of dense crushed rocks which are dropped into the cavity and incrementally densified by two horizontal counter rotating eccentric weights, that imparts vibratory excitation to the surrounding granular materials causing increased densification and thus reducing liquefaction susceptibility. The effectiveness of treatment depends on the c-to-c spacing of the densified granular columns. Usually this is done in triangular pattern.

Stone columns extend the use of deep vibratory process to even cohesive soils such as clays and silts. The stiff stone column reinforces the soil matrix and also increases the composite strength of the soil.

The stone columns reduce foundation settlement, enhance bearing capacity and more importantly reduce *Liquefaction Potential.*

Care must be used in order to prevent uplift or heaving of previously installed columns particularly in very poor cohesive soils.

4.2.3 Pore water Relief

Of very great importance is the recognition that porewater pressure relief through the use of vertical drainage materials will assist in preventing the set up of pore pressures to cause liquefaction.

Documented evidence during the *Hyogen-Nambu (Kobe)* Earthquake showed that Port works where granular piles were installed did not liquefy in stark contrast to adjacent failed structures where no granular piles were installed.

The granular piles acted as chimney drains in which the pore pressure was dissipated through rapid drainage into the highly permeable granular columns.

Thus, in this specific case, vertical drainage in the form of Granular piles, Geopiers, prefabricated vertical drains, or sausage drains can be deployed to prevent liquefaction initiation.

Of the foregoing, the use of granular piles and prefabricated vertical drains (PVD) hold prominence as a liquefaction countermeasure.

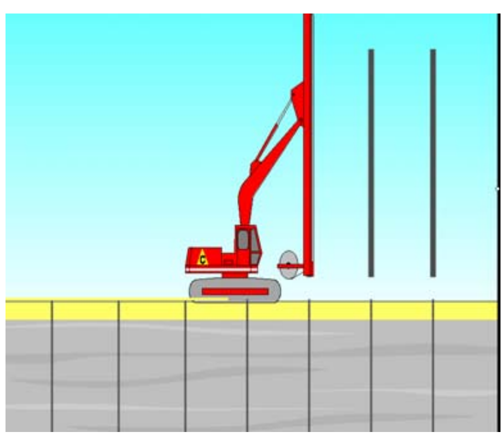


Figure 4.2.3.1 - Showing PVD Installation

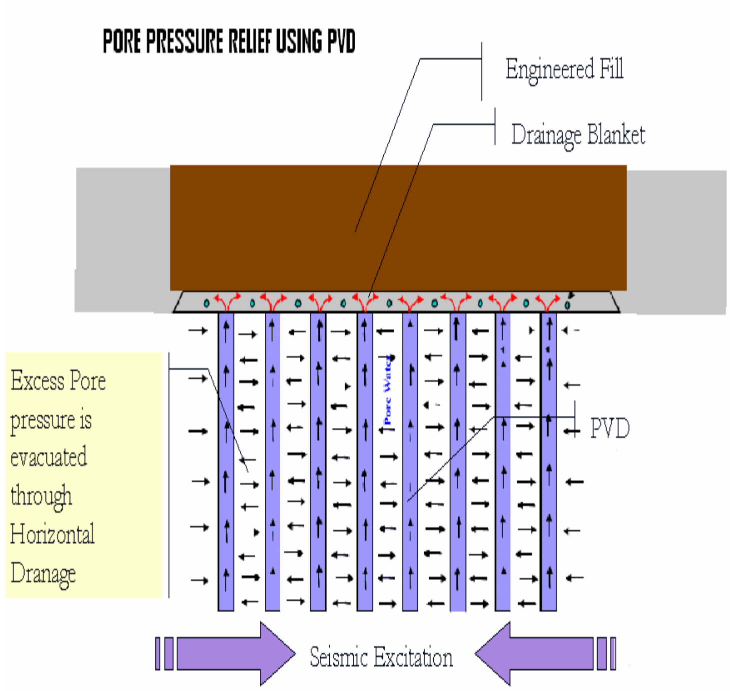


Figure 4.2.3.2 - Showing Pore pressure relief in Liquefiable soil using PVD

4.2.4 Compaction Grouting

Compaction grouting using cement injected at high pressures incrementally can assist in densifying the poor granular soils. The cement slurry is injected under high pressure to form spherical bulbs of slurry
expanding into the soil. This expansion would cause lateral stressing of the soil. However, the influence of the expanding grout is limited requiring numerous elements to be installed.

Similar to compaction grouting but different in installation procedure and effect is achieved using Jet Grouting.

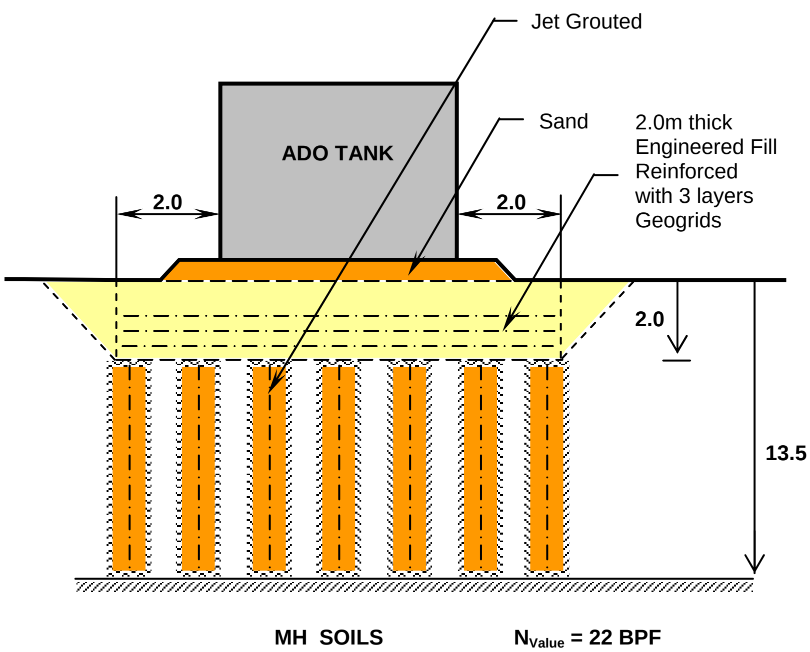


Figure 4.2.4 - Showing Jet Grouted Installation to bypass Liquefiable Soils

Jet grouting involves the creation of large diameter columnar soil cement piles known as "*Soilcrete*" to bypass the liquefiable or poor soils and also induced lateral compaction of the surrounding ground to a limited extent. The main advantage is significantly larger loads can be carried and transferred to more competent ground thus bypassing the potentially liquefiable soils.

4.2.5 Use of Explosives "Camouflet"

Explosives have been used in the American Civil War in order to either collapse enemy tunnels or introduce shock waves and gases into the tunnel.

In Civil Engineering, *camouflets* were used specifically by the Russians to densify Loessial soils and loose to very loose sands. The spherical shock wave after the explosion creates an instantaneous cavity after expansion which then collapses in itself thus densifying the ground.

It would be necessary to drill a hole sufficiently deep enough for the overburden thickness not to be lifted bodily by the explosion.

Once initiated, the area subjected to the explosion is densified by a series of well placed explosives. The explosion is most effective under saturated soil conditions.

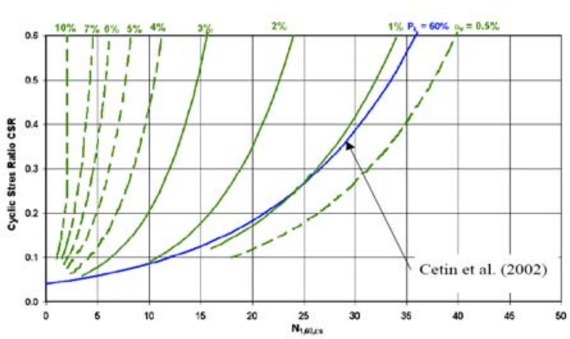
5.0 RESEARCHES ON LIQUEFACTION MITIGATION

A lot more need to be understood regarding the liquefaction phenomenon. Research in this field is continuing particularly in the areas of post liquefaction residual strength prediction as well as prediction of settlements induced by liquefaction.

5.1 Post Liquefaction Prediction of Volumetric Reconsolidation

The prediction of settlements or volumetric changes after the triggering of liquefaction, needs to be quantified or estimated.

The work by *Cetin et al* 2002 is a step towards this direction.



(a) Volumetric Reconsolidation Strains

The horizontal axis of the Figure above represents fines-adjusted, and normalized SPT penetration resistance, using the same fines corrections that were employed previously in the new "triggering" relationships. The vertical axis represents equivalent uniform cyclic stress ratio adjusted for: (1) magnitude-correlated duration weighting (DWF_M), and (2) effective overburden Stress (Kσ). In using this figure, the earthquake-induced CSR_{eq} must be scaled by both DWF_M and K_σ.

To estimate expected site settlements due to volumetric reconsolidation, the recommended procedure is to simply divide the subsurface soils into a series of sub-layers, and then to characterize each sub-layer using SPT data. Volumetric contraction (vertical strain in "at-rest" or K0 conditions) for each sub-layer is then simply summed to result in total site settlements.

5.2 Post Liquefaction Prediction of Residual Strength $S_{u\,r}$

Corollary with the need to predict Post liquefaction volumetric changes is the need to predict residual strengths after a liquefaction event has occurred..

Seed and Harder 1990 have come out with recommendations relating SPT N_{value} $N_{1,\,60}$ with the mobilized undrained critical Strength $S_{u\,r}$.

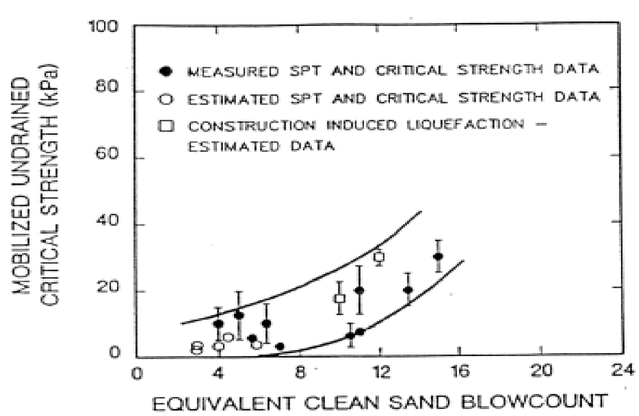


Fig. 41: Recommended Relationship Between $S_{u,r}$ and $N_{1,60,CS}$ (Seed and Harder, 1990)

Prediction of the residual strength is important in determining whether the insitu residual strength could result in a catastrophic event due to the significant weakening of the liquefied ground.

6.0 CONCLUSIONS

The foregoing has presented current "*state of practice*" in Liquefaction assessment and has presented the available anti-liquefaction measures that could be mobilized by the engineering professions.

Most of the information has been culled from literature reviews by the authors and also from combined experiences.

The state of the art in the understanding of the Liquefaction Phenomenon is still evolving and can be considered a "*work in Progress"* by various researchers worldwide.

More work needs to be done in order to fully understand Liquefaction and how to mitigate or eliminate its effects.

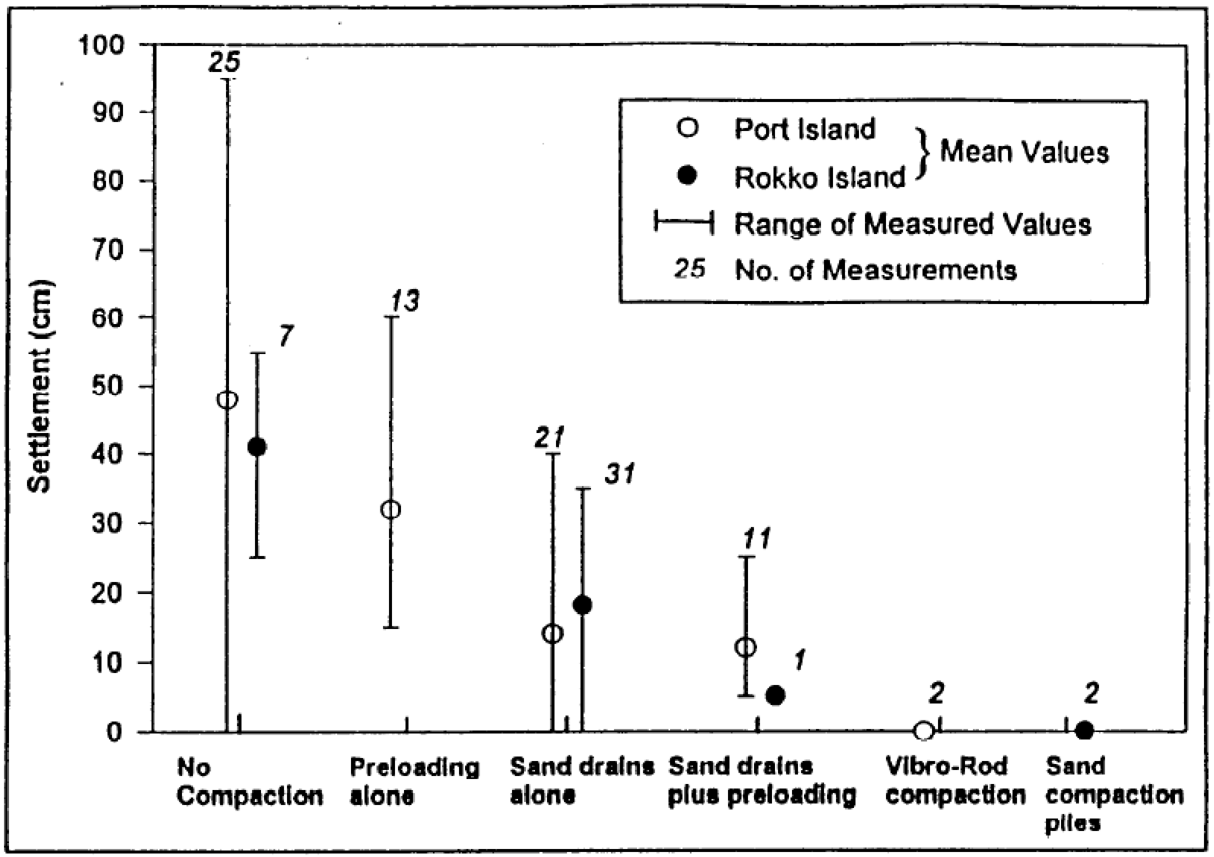


Figure 45. Measured settlements at improved sites due to the 1995 Hyogo-ken Nambu (Kobe) earthquake (after Yasuda et al., 1996).

Table 1.0 Summary of Anti-Liquefaction Measures and their Effects

	Liquefaction Countermeasure	Main Action Against Liquefaction	Densification	Lateral Compaction	Drainage and Pore Pressure Relief	Remarks
1)	Chemical Grouting	Cementation	No	No	No	Shallow Depths
2)	Conventional Piling	Bypass liquefiable layer	Yes for driven Piles	No	No	Driven piles can induce localized Densification
3)	Stone Columns	Transfer load to competent soil	Yes	Yes (Medium)	Yes	Proven performance in liquefaction Zones
4)	Geopier	Transfer load to Surrounding Improved Ground	Yes	Yes	Yes	Proven performance in liquefaction Zones
5)	Compaction Piling	Densification	Yes	No	No	Effective for shallow depths but laborious Installation
6)	Resonant Column	Densification	Yes	No	No	Densification is achieved for Shallow depths
7)	Dynamic Compaction	Densification	Yes	Slight	No	Shallow dept effectiveness < 8.0 meters
8)	Vertical Drains	Porewater relief	No	No	Yes	Effective for Rapid Pore Pressure relief
9)	Compaction Grouting	Densification	Yes	Yes	No	Shallow soils
10)	Jet Grouting	Transfer load to competent soil	Yes (Slight)	Yes (Slight)	No	Esentially used to bypass liquefiable soils

START

Determine Liquefaction Susceptibility of Soil

Soil Type, % Fines
LL, PI

Determine Triggering Potential

SPT N_{Values} Cyclic Triaxial Test

SPT Based ➞ Use Correlation Chart

Corrections:
- *N_{Value}*
- *Rod Length*
- *Overburden*
- *Samples*
- *Hammer Efficiency*
- *Borehole Φ*
- *Fines Content*

Determine Cyclic Stress Ratio (CSR) using $CSR_{eq} = (0.65)(CSR_{peak})$

$$CSR_{peak} = \frac{a_{\max}}{g} \cdot \left(\frac{\sigma_v}{\sigma_v}\right) rd$$

Correct for Magnitude using DWF

$$CSRN = CSR_{eq\ \mathrm{M}=7.5} / DWF$$

DWF – Magnitude Weighing Factor

Correct for Overburden Stress

$$CSR^* = CSR_{eq=7.5} / K\sigma$$

Use Fig. 16 for Probability Assessment
Use Fig. 17 for Deterministic Assessment
with Fines Content *(see page 8 of paper)*

Evaluate using

$$CRR\left(N_{1,60}, CSR, M_w, \sigma'_v, FC, P_L\right) =$$

$$\exp\left[\frac{N_{1,60} \cdot (1 + 0.004 \cdot FC) - 29.53 \cdot 1n(M_w) - 3.70 \cdot 1n(\sigma'_v) + 0.05 \cdot FC + 44.97 + 2.70 \cdot \Phi^{-1}(P_L)}{13.32}\right]$$

Calculate Factor of Safety against Liquefaction

$$FS_{liq} = \frac{CRR}{CSR} > 1.0$$

END

REFERENCE:
R. B. Seed ***"Recent Advances in Soil Liquefaction Engineering-a Unified and consistent Framework"*** 26th Annual ASCE Los Angeles Geotechnical Spring Seminar

Appendix 1: Flow Chart for Liquefaction Susceptibility Assessment