Earthquake Induced Liquefaction Analysis and Ground Improvement as a Remedial Measure: A Review

Ubaid Hussain\textsuperscript{1*} and Amanpreet Tangri\textsuperscript{2}

\textsuperscript{1*M.E Scholar and 2Assistant Professor} Department of Civil Engineering, Chandigarh University, Punjab, India

Email: \textsuperscript{1*}ubeyabi1618@gmail.com and \textsuperscript{2}amanpreet.civil@cumail.in

Abstract: Liquefaction is the phenomenon in which partially or fully saturated, loose sandy soils behave like a liquid due to loss of strength and rigidity owing to sudden increase in the pore water pressure as a result of dynamic loading such as earthquake. Liquefaction induced by dynamic loading as a result of earthquake is the most destructive feature of earthquake that may results in settlements and collapse of structures .The severity of this phenomenon can be predetermined by the geological and hydro-geological setup of the soil in the study area. The aim of this study is to present a review of various aspects of earthquake induced liquefaction analysis, case evidences from field studies and some of the liquefaction hazards from past earthquakes. Remedial measures using ground improvement techniques to prevent liquefaction hazard is also studied in this paper. Further, investigating the performance of remedial methods against liquefaction is also presented in this paper.

Keywords: Liquefaction, Pore water pressure, dynamic loading, geological setup, ground improvement.

1. Introduction

Liquefaction hazard induced by earthquake is a well known phenomenon occurring in loose sandy, saturated soils. Analysis of liquefaction related features in seismically active regions can be done by cyclic stress ratios. Soil parameters such rigidity looseness, void ratio and soil stress conditions are affecting the shear /cyclic stress ratios (CSR or SSR). Cyclic loads induced in the soil by earthquake and soil liquefaction resistance are the parameters to estimate the liquefaction . Cyclic loads just sets out as initiating mechanism for liquefaction on the other hand, it is the sustained initial stress which is basically responsible for liquefaction triggered hazards . Method of liquefaction analysis depends upon the shear/cyclic resistance ratio (SRR or CRR), cyclic stress ratio, soil thickness [1–4]. Cyclic loads are being determined by shear wave velocity (\(V_s\)), soil and earthquake parameters. Cyclic stress ratios can be determined by using small strain shear waves, which is having an advantage of being applicable to all types of soils and with only one disadvantage of being incapable to differentiate soil types and thin layers. Small shear strain waves are suitable most in cemented soils and silty soils above ground water table. Earthquake and soil parameters for determination of cyclic loads for liquefaction analysis include: Magnitude scale factor (MSF), soil pressure correction factor, magnitude of earthquake, peak acceleration values. The most important parameter is small strain shear...
wave velocity, which can be used in place of SPT. The extent of liquefaction and its potential is dependent upon various soil and earthquake parameters such as subsurface water conditions, ground shaking strength and propagation of seismic wave energy [5,6].

Liquefaction poses a serious threat near embankments, sloping terrain and structural foundation because of having working initial shear stress which promotes liquefaction hazard. Sliding failure is observed in this case. Fine grained soils are vulnerable to strong earthquake induced liquefaction hazards. The tendency of these soils to earthquake induced liquefaction can be analysed by screening criteria and geotechnical analyses can be done to measure their resistance to earthquake induced liquefaction [7–9]. Liquefaction damages induced due to great earthquakes (Cristchurch I & II) had adverse effects on the major part (more than one third) of the Cristchurch urban area, New Zealand, owing to ground damages and instability hazards. Case evidences from field analysis are being carried out to support these liquefaction damages. In soil frozen regions, liquefaction occurs in medium fine sand and has been observed to occur in their lower portion, which comes under seismic deformation zones. Earthquake induced liquefaction and damage induced by it are major source of destruction during an event of earthquake. Liquefaction induced by earthquakes leads to various forms of ground failure such as sand boils, settlements, ground flow failures, lateral spreading etc resulting in damage to structures and life losses. Liquefaction hazard and its induced damages can be mitigated by various ground improvement techniques [10,11].

The present study is aimed to give review of earthquake induced liquefaction, analysis of liquefaction estimation and some case evidences from field data to support liquefaction hazard and analyse damages induced by it. Remedial measures for liquefaction prevention using ground improvement techniques such as stone columns, PVDs, CS grout, containment walls and sand tyre mix and performance evaluation using centrifuge modelling is also presented in this study [12,13].

The aim of this study is to help the field engineers to select the suitable method of ground improvement technique based on the soil and earthquake parameters, soil and stress conditions and liquefaction potential of the soil. To identify the soil susceptible to liquefaction hazard: by identifying the stratum which is prone to liquefaction at the ground surface and having lowest corrected shear wave velocity. Thus critical layer triggering the liquefaction can be identified and hence the depth to which ground improvement will be required so as to prevent liquefaction damages. To estimate the liquefaction potential of the soil strata, thus helping the engineers to avoid construction on the liquefiable soils and also improve the liquefaction resistance of soils to prevent possible hazard to the life and property. The liquefaction related settlements of existing structures experiencing earthquake shaking can be remediated by containment walls, provided the whole liquefiable layer is improved up to the point of liquefaction susceptibility [14,15].

2. Literature review

2.1 Liquefaction estimation and susceptibility analysis:

Determination of three parameters is required for the estimation of liquefaction using shear wave velocity. These parameters include: Shear stress ratio (which represents the cycling loading on the soil), shear resistance ratio (which reflects the soil liquefaction resistance) and lastly the factor of safety to differentiate liquefaction and non liquefaction. Shear stress ratio is being determined from corrected shear wave velocity, soil & earthquake parameters [16,17].
2.2 Study of parameters for liquefaction analysis:

P waves are not being considered in computing soil liquefaction, because stress induced by these waves is compressive and is transferred to the soil voids via water as a result, there is no change occurring in the effective stress. S waves inducing horizontal shear stress forms the major component for liquefaction analysis during an event of earthquake. The equation for computation of SSR at a particular depth in a level soil deposit is given by:

\[
SSR = \frac{\tau_{av}}{\sigma'_{vs}} = \frac{\sigma_{vs}}{\sigma'_{vs}} a_h g r_d
\]

Where, \( \sigma_{vs} \) is dynamic vertical stress, \( \sigma'_{vs} \) is dynamic effective vertical stress, \( \tau_{av} \) is average shear stress, \( a_h \) is horizontal ground surface acceleration and \( r_d \) is stress reduction factor.

CSR by an earthquake in water saturated soils is computed as:

\[
CSR = \frac{\tau_{av}}{\sigma'_{vs}} = 0.65 \frac{\sigma_v}{\sigma'_v} a_{max} g r_d
\]

Where, \( \sigma_v \) is vertical stress, \( \sigma'_v \) is effective vertical stress and \( a_{max} \) is maximum ground surface acceleration.

Volumetric reduction in the liquefaction phenomenon is related to cyclic deformation. Liquefaction estimation depends upon the deformation extent. The relationship between cyclic shear deformation and cyclic shear stress is given by:

\[
\gamma_{av} = \frac{\tau_{av}}{G \gamma_{av}}
\]

Where, \( \tau_{av} \) is average shear resistance, \( \gamma_{av} \) is average shear deformation peak and \( G \gamma_{av} \) is shear module.

The average shear deformation during an earthquake is given by:

\[
\gamma_{av} = \frac{a_{max}}{g} \sigma_{vs} r_d \frac{G_{max} 1}{\rho v_s^2 G_{max} } \gamma_{av}
\]

CRR OR SRR in terms of average shear deformation and shear wave velocity can be presented as:

\[
CRR = \frac{\tau_{av}}{\sigma_{av}} = f(\gamma_{av}) v_s^2
\]

\[
SRR = \frac{\tau_{av}}{\sigma'_{v}} = f(\gamma_{av}) v_s^2
\]

\[
F(\gamma_{av}) = \gamma_{av} \frac{\rho}{\sigma'_v} \frac{G_{max}}{\sigma_{max} v_s^2}
\]

It has been observed that when \( V_s \) is maximum (i.e. \( V_{max} \)), liquefaction is said to occur. Further, \( V_{max} \) decreases as the percentage of fines increases. Liquefaction analyses based on SPT shows that when \( N > 30 \), liquefaction is not possible. Further it is assumed large magnitude earthquakes are more predominant in giving rise to the liquefaction hazard than small earthquakes. Relationship between CSR or SSR with magnitude scale factor based on SPT values from lab tests and field data as well as from shear wave velocity have been presented.
\[
\text{CSR}_{7.5} = \frac{\text{CSR}}{\text{MSF}} \quad \text{Eq. 8}
\]

\[
\text{SSR}_{7.5} = \frac{\text{SSR}}{\text{MSF}} \quad \text{Eq. 9}
\]

Where MSF is magnitude scale factor.

The third parameter for estimation of earthquake induced landslides is factor of safety (FOS), which is given by:

\[
\text{FOS} = \frac{\text{CRR}}{\text{CSR}} \quad \text{Eq. 10}
\]

\[
\text{FOS} = \frac{\text{SRR}}{\text{SSR}} \quad \text{Eq. 11}
\]

It has been concluded that for the following cases of FOS given by:

\text{FOS} > 1.2, \text{ No liquefaction occurred.}

\text{FOS} = 1-1.2, \text{ Possibility of liquefaction.}

\text{FOS} < 1, \text{ Liquefaction will occur.}

For the analysis of liquefaction behaviour of sandy soils with various relative densities and fines content under the influence of initial shear stress, torsional simple shear undrained cyclic loading test was performed. Hollow cylindrical torsional shear test apparatus was used. Failure mechanism under initial stress condition: for better understanding of cyclic failure mechanism sands under initial stress conditions, monotonic loading tests were performed on the same sand sample and same apparatus as used for torsional test. It was observed that for fines content = 0%, stress path shows dilative response. For sands with fines 10% & 20%, behaviour was seen contractive, with strain softening after reaching peak stress. For sands with fines 10%, stress path goes down and then rises up slightly. From the observations, it was seen that the liquefaction induced flow failure occurs when initial state of sand lie on the contractive side of the steady state line. The integration of number of loading cycles, initial shear stress and cyclic stress amplitude should be enough for effective stress path to reach CSR line and also the shear stress of PT points (\( \tau_{\text{pt}} \)) should be smaller than shear stress on CSR line for strain softening flow to occur. Also in cyclic loading under initial stress condition, various types of failures were being observed namely cyclic failure (CF): where in periodic strain growth occurs, cyclic biased failure: wherein pore pressure rises up to 100% and shear stress develops swiftly and lastly cyclic biased gradual failure (CBGF): wherein pore pressure does not rise up to 100%, hence biased strain elevates gradually.

Liquefaction potential estimation was done using Piezocone (CPTU) and seismic dilatometer (SDMT) tests. The CRR at 7.5 magnitude was evaluated by normalised cone tip resistance for CPTU and horizontal stress index (\( K_0 \)) for DMT. On the other hand, CSR at same earthquake magnitude of 7.5 was evaluated. CPTU provides high liquefaction potential (10.5) in comparison to DMT which ranges between low and high liquefaction potential (3.8 and 6.8) in terms of liquefaction potential index (LPI), while shear wave velocity ranges between high and very high LPI (14 and 16.5).

Field analysis for liquefaction related features was being carried out on a drainage channel up to a depth of 1.8-2.0 m. The stratum is composed of brown reddish agricultural soil 60-100 cm thick underlain by lacustric succession. In situ tests were performed to observe the liquefaction susceptibility of the site under consideration. Piezocone test (CPTU) and seismic dilatometer test
(SDMT) has been used to obtain the soil parameters supporting liquefaction susceptibility. Consistency limits or % finer deposits guided as a tool for analysis of liquefaction susceptibility of fine grained soils. It was found that:

- Silty soils with clay content < 10% and L.L ≥ 32% are liquefiable.
- Loose soils with P.I < 12 % and L.L > 0.85 are also prone to liquefaction.

CPT provides high liquefaction potential (10.5) in comparison to the DMT which ranges between low & high liquefaction potential (3.8&6.8) in terms of liquefaction potential index (LPI), While as the shear wave velocity ranges between high & very high LP (14&16.5 respectively) [8].

Estimation of liquefaction potential due to earthquakes using Evolutionary Polynomial Regression (EPR), which is a data driven method, is based on the evolutionary computation using the expression given by:

$$y=\sum_{j=1}^{n} F(\mathbf{X}, f(\mathbf{X}), a_j) + a_0$$  \hspace{1cm} Eq. 12

Where, \(y\) = estimated output vector of the process, \(a\) = a constant, estimated by method of least squares, \(F\) = function developed by the process, \(f\) = user defined function, \(\mathbf{X}\) = input variables matrix and \(n\) = number of terms of target expression.

Earthquake induced liquefaction occurs when Pore pressure reaches confining pressure in the water saturated loose Sandy soils as a result of loss of strength of soil and rigidity. Onset of liquefaction hazard depends upon number of factors such as soil characteristics, ground water table depth, and ground shaking parameter and distance of earthquake from source [9]. Susceptibility to liquefaction can be done by identifying the stratum which is prone to induce liquefaction at the ground surface. This can be identified by checking that the stratum is liquefiable and lies below ground water table from lab tests (soil classification and soil boring tests), stratum has lowest corrected shear wave velocity \((v_s)\) from field test. Following this we can identify the critical layer triggering liquefaction.

The corrected shear wave velocity can be computed as:

$$V_{cs} = V_i C_v = V_i \left(\frac{p_a}{\sigma_v}\right)$$  \hspace{1cm} Eq. 13

Where, \(p_a\) is atmospheric pressure and \(n\) is exponent in Hardin equation [10].

Analysis of liquefaction potential of the study area was being executed by performing CPT (cone penetration test) and CPT\(U\) (Piezocone test) data by Robertson and Wride (1989) with the 6.2 magnitude of earthquake. This analysis observed that relationship between CSR due to earthquake loading and CRR of the soil in the study area. The ratio of CRR and CSR was designated as a factor of safety. Factor of safety equal to 1 was taken as the threshold. Field analysis for liquefaction related features was being carried out on a drainage channel oriented WNW -ESE up to the depth of 1.82 to m. Piezocone test CPT\(U\) and seismic dilatometer test SDMT has been used up to 15 m depth. Data from CPT\(U\) you and SDMT can be used to obtain the soil parameters supporting liquefaction susceptibility. Atterberg’s \(l\)imits for percentage finer deposits guided as a tool for analysis of liquefaction susceptibility of fine grained soils. It was concluded that silty soils with clay content less than 10% and liquid limit ≥32% are liquefiable, secondly loose soils with plasticity index <12% and W.C/L.L >0.85 are also prone to liquefaction.
3. Case evidences of earth induced liquefaction

Case evidences associated with the phenomenon of liquefaction and liquefaction damages induced by some past earthquakes such as 2015 Gorkha earthquake, 13 February 2016 Fairview earthquake, 25 November 2016 Akto earthquake of the 2016, March 2011 Tohoku earthquake, 27 February 2010 Chile earthquake.

3.1 Liquefaction evidences:

This earthquake resulted in severe damage to structures and loss of lives. Soil liquefaction was witnessed comprising of sand blows and lateral spreading at 12 locations. It was found to be the main cause of structural damages. Liquefaction sites we are mostly observed in the city outskirts of Kathmandu Valley. The liquefaction occurrence was localised in alluvial deposits of pleiocene to quaternary age. Liquefaction induced surface expressions were also observed in rice fields and fallow lands. Fissures triggered by liquefaction was characterized by turbulent water flow out of those fissures. Send boiling post liquefaction was also witnessed in the area.

27 February 2010 Chile earthquake: This earthquake occurred in South Central part of Chile with a magnitude 8.8 and focal depth 35 km. Soil liquefaction was observed at several sites in 100 kilometres range. After carrying fieldwork survey and other related reviews, liquefaction affected sites we are identified, which include 170 sides with clear signs of liquefaction. The liquefaction was widespread and constituted an area of length around 1000 km, which comes about to be twice the rupture zone length. Lateral spreading owing to liquefaction was observed along the south of rupture zone. South of the rupture zone experienced most of the liquefaction cases which is attributed to the presence of saturated sand soil deposit in the area.

Tohoku earthquake Japan in year 2011 occurred with the magnitude of 9.0 to 9.1 with epicentre 70 kilometres east of oshika and focal depth of 29 km. This earthquake resulted in extensive and widespread liquefaction in its areas of close vicinity. Liquefaction induced damage due to tohoku earthquake include house damages and infrastructure damages. 177 house were damaged most of them situated in the liquefaction areas. Damage to electric poles, roads was also recorded. Some cases of damage to underground sewer lines and water distribution pipes were also observed.

4. Ground improvement techniques as liquefaction remediation

4.1 Stone columns: Introduction of stone columns to mitigate liquefaction hazard was first studied by seed and broker. This technique of stone columns by Vibro-replacement (Vibroflotation and backfilling) reduces the liquefaction potential by increasing the relative density of the soil, providing drainage path to dissipate excess pore water pressure and increasing stress carrying capacity via stone columns and hence improving the liquefaction resistance of the surrounding soil. Stone columns tend to keep the pore water pressure ratio \( t_\sigma = \frac{\sigma_v}{u_c} \) (\( u_c \) is excess PWP, \( \sigma_v \) is initial effective vertical stress) low (< 0.5) by increasing pore water pressure dissipation rate and hence giving advantage of preserving large portion of soil strength, preventing large settlements and decreasing high hydraulic gradients which finally results in increase of soil liquefaction resistance.

4.1.1 Construction: Stone columns can be constructed by various methods, the most frequently used method of Vibro-replacement and auger casing system. The first method: Vibro-replacement utilizes combination of two techniques viz: Vibroflotation and backfilling. In Vibroflotation a probe is inserted up to a depth of interest by vibration and jetting action of water or air. Water reduces inter particle friction of surrounding soil during the process to cause densification of the soil. Vibroflotation
is done simultaneously followed by backfilling using gravel material either by top feed or bottom feed methods. Backfill material is compacted by vibration and alternate raising and lowering of probe and when desired density is achieved which depends upon the type of soil, fines content, relative density, area of stone column, spacing etc, the process is stopped. In the second method: auger casing system, using current Japanese practice involves internal gravel feeding and compaction rod system resulting in soil densification. It is important to ensure that the technique is operating efficiently and effectively during the construction so that the low densification of the soil is not discovered after the construction. This can be done by proper quality control and monitoring of the process. Also, in order to verify that desired results are obtained after the site improvement, in situ tests such as SPTs, CPTs are performed. Currently CPTs are most commonly used for verifying relative densities. Stone columns have advantage that no significant vibration or noise produced during the construction and hence suitable for urban sites adjacent to existing structures.

4.1.2 Case evidences: Stone column method to the immediate liquefaction was applied for the first time in Japan 1978 and until 1993 more than 200,000 stone columns constructed in Japan and during period of 1993 to 2000, 150,000 stone columns were installed in Japan.

4.1.3 Seismic performance from case evidences: 12 sites of ground improvement subjected to loma prieta earthquake 1991 were examined to evaluate their performance. It was found that these sites were subjected to little or no liquefaction damage with respect to ground and built up structures. Case histories of several sites in Japan also revealed that performance of ground modification with stone columns was also improved.

4.2 Colloidal silica (CS): This technique has been proposed by Gallagher 2002 to reduce liquefaction hazard. Colloidal silica is the cheapest advanced nanomaterial incorporated in the passive soil stabilisation. Colloidal silica involves passive soil stabilisation, it is therefore necessary to maintain smooth delivery of grouting materials under low gradient terrain. CS to be used requires low initial viscosity and noteworthy permeability features. It is because of this principle of low viscosity (2cp), it can flow through pores of liquefiabe soil and reaches across the soil stratum. After CS reaches the location of interest, extraction is terminated and then CS gets changed into solid form. Advantages CS is non-toxic and inert chemically and biologically, and also safe to the environment. CS has the ability to decrease axial strain and also reduces the tendency of pore water pressure development in the soil experiencing cyclic loading. Because of this property is useful material to be incorporated in the ground improvement of liquefiable soils strata.

4.2.1 Properties of CS stabilized soil: It has been found that strength of the soil increases by 2, 7.5 and 8 times more than clay lime mixture after the addition of 1%, 3% and 5% of CS respectively. There is increase in the unconfined compressive strength of CS improved soils than original soils. Shear strength of the soil stabilized with CS is also higher than pure soil samples owing to enhancement of shear strength parameters. The extent of strengthening is dependent on the curing time also unconfined compressive strength of soil increases with increase in the percentage of CS. As the extent of Cementation increases due to the properly grounded CS mixture, it results in the improvement of dynamic soil properties such as stronger cyclic resistance and liquefaction resistance. It has been observed that addition of 15% by weight of colloidal silica grout to the soil will increase the dynamic resistance of the soil against liquefaction.

4.2.2 CS stabilized soil performance: Field tests were performed by Gallagher et al 2007 to verify the performance of CS stabilized soil for liquefaction remediation. Side of study was within a Canadian liquefaction expose its side situated in the Richmond British Columbia. Soil strata at the site is
proposed of sand and silty sand at the surface, with layer of silt and sand silt underlying the surface layers, followed by 10 metre to liquefiable layer of sand to silty Sand at a depth of 5 to 15 metre. Site was improved by CS grouting. The site was tested for performance in liquefaction by installing two decks of Pentolite 50/50 explosives to create blast induced liquefaction. It was observed that pore water pressure reached 1 in the treated area, indicating grouting layer to be liquefied. Maximum settlement was 0.5 in untreated area, 0.3 in the treated area and it was believed that overall settlement can be prevented if entire layer is grouted. It was concluded that CS has ability in improving ground for liquefaction resistance during any earthquake, which is attributed to decrease in the settlement and Shear Strain during cyclic loading and reduces built up of excess pore water pressure by this improvement technique. Since some failures were observed in the field tests conducted by Gallagher et al., still settlements are reduced significantly by CS grouting to mitigate liquefaction. It also has advantages over other ground improvement techniques for liquefaction resistance because of less disturbances to the surrounding areas, low cost and environment friendly.

4.3 Containment walls: This technique is used as a liquefaction remediation to reduce settlements of existing structures by preventing the lateral moment of the foundation sand. These walls act as a cut off walls and prevent seepage into a mediated zone. These are also found to be effective in reducing settlements of the structure. Migration of poor water pressure is also prevented by use of contaminant walls and hence found useful method of remediation for existing structures.

4.4 Sand tire mix: This is one of the economic, cost effective and environmental friendly method for both new and existing structures using waste materials. Two types of materials were used which includes clean sand with fines content less than 6% taken from liquefied site in Christchurch and tire chips (TC). Tire chips were taken from used tires with metal and fibres removed and then assembled in smaller pieces. Tire chips used we are of two types depending upon the diameter viz TC_s: smaller particle size of dia 1.18 mm, TC_b: bigger particle size of dia 3.25 mm. The mixture was prepared by mixing air dried and oven dried chips with pure sand separately, with initial water content of 20%. The Specimen of sand tire chips mixture was then prepared in layers, each layer being compacted by moist tamping by dropping a rammer from stipulated height and using suitable number of blows to obtain a compaction energy of 26 KJ/m^3. In case of oven dried chips (50°C), the oil oozing from the tire chips covered surface of both the chips as well as the sand particles, making it hydrophobic. As a result of this, under the compaction effort, water in the voids can easily expulse out, thus a denser specimen with a relative density (62%)which is more than that of a pure sand (Dr=50%). It was found that the sand tire mix incorporating oven dried chips achieved a higher relative density and hence liquefaction resistance of this mixture is higher than that of pure sand. For the specimen using air dried chips, lower relative density (Dr=38%) was achieved and hence less liquefaction resistance than the pure sand (Dr=50%). The lower relative density of an oven dried sample was attributed to the hydrophilic nature, due to which the water in the voids is locked firmly and hence less denser packing of this mixture. All other details of the materials and mixture are given in the literature. Undrained monotonic and cyclic triaxial tests were performed on the samples of sand tire chips mixture to evaluate the best performance under practical conditions. The sample giving the most effective results in the in terms of higher undrained shear strength and liquefaction resistance was found to be sample with large sized chips and oven dried.

4.5 PVDs: Prefabricated vertical drains: Are corrugated plastic pipes with open slots and wrapped in fabric filter to prevent clogging. These are characterised by rapid dissipation of excess PWP and better liquefaction prevention than stone columns owing to larger discharge capacity and storage. The excess pore water pressure generated due to earthquake loading can be drained by PVDs and also reduce excess pore water pressure which can prevent the liquefaction hazard to occur. As the drainage
part is reduced by installing PVDs and due to high permeability of PVDs, the rate of dissipation of excess pore water pressure is increased. It also checks the accumulation of pore water in layers of low permeability by creating a pathway through them. PVDs are hollow perforated plastic pipes ranging from 75 to 200 mm in dia. These can be installed in triangular or square pattern using either a vibratory or auger casing system. As the water is drained out, densification of soil also occurs in addition to the main benefit of drainage path. At various sites across United States, PVDs have been installed. From field tests using controlled blasting it was concluded that PVDs are effective in dissipation of pore water pressure and reduce settlement significantly after their installation. However settlements may occur during the installation procedure.

5. Investigating performance of ground improvement techniques for liquefaction mitigation

5.1 Shaking table model testing:

Performance of gravel drains (stone columns) was studied by performing small-scale shake table tests. It was observed at the stone columns resisted liquefaction within a range of 0.5 m radius around the axis of the drain of total radius 0.4m. Also, high frequency earthquakes overpowered the stone columns due to quick build-up of pore water pressure. In another large shake table tests on stone column (0.6 m dia), it was observed that these drains perform very well when \( r_u \) lies below 0.5 and performance decreases as \( r_u \) increases above 0.5. The success of gravel drain performance during earthquake depends upon its ability to maintain \( r_u \) below 0.5.

5.2 Centrifuge modelling:

Single degree of freedom system (SDOF) at model scale was employed in the centrifugal testing. In order to investigate the efficiency and effectiveness of ground improvement techniques for liquefaction mitigation under earthquake, centrifuge modelling was done. It involves the small scale model testing in enhanced field of gravity of geotechnical centrifuge, Realistic simulation of practical conditions based on small scale physical models. In the centrifuge simulation, earthquake loading is modelled, which is accomplished by the actuator sufficiently powerful to induce strong shaking of small-scale models, which then undergo lateral shaking. Through various instrumentations, the behaviour of a model before, during and after failure can be predicted and monitored. Centrifuge tests on loose and dense, dry and saturated models of homogenous, horizontal sand layers was carried out.

5.2.1 Ground improvement using stone columns & PVDs:

This method uses drain to relieve excess PWP generated during an earthquake to mitigate Liquefaction hazard. Series of centrifuge tests were carried out to investigate the performance in terms of mechanism and effects by which these drains mitigate the liquefaction risk of liquefiable soil. Tests were performed on loose, saturated horizontal layer of sand. Sand used in the test was dry pulverized with the relative density 50% and comprised of LB fraction E sand with \( D_{50} = 0.14 \), \( e_{min} = 0.613 \) and \( e_{max} = 1.014 \). Drains were installed from LB fraction B sand with the \( D_{50} =0.9 \) mm by dry deposition in plastic moulds. Tamping was done to a relative density of 71%. Centrifugal models were saturated with silicone oil under vacuum. It was observed that these drains perform well after the liquefaction, during which they help the ground to regain strength more quickly. It was also found that \( r_u \) falls to about 0.4 after 2 seconds if impermeable layer is present outside, thus making drains more effective and efficient as compared to when no impermeable layer is present outside (\( r_u = 0.8 \)). These impermeable layers prevent the inflow of fluid and cause redistribution of excess pore water pressure, thus making functioning of drains better. Based on these observations, drains were classified as internal drains, sub- perimeter and perimeter drains. It was proposed that design of drains at the site
should be carried out as internal drains, with additional rows of drains provided on the Periphery to act as a perimeter and Sub perimeter drains, so that better performance would be expected to close to unit cell assumption.

Centrifuge model testing was performed by U C Davis on the ground treated with PVDs to investigate their behaviour for mitigating liquefaction hazard when compared to untreated (with and without non draining tubes) sloping ground with liquefiable sand underlain by crust of Low permeability. Total nine shaking phases were given to the model over a period of 5.5 hours with sufficient intervals in between to account for PWP dissipation. From the observations of centrifugal testing it was observed that \( r_u \) increases in untreated area during the shaking and remains higher throughout and even after shaking for several seconds and then reduces to below 0.5 (usually above 0.2). In the treated area the \( r_u \) level during shaking is smaller than in untreated area and then it decreases to a value of 0.2 (usually below 0.3) after few seconds of the end of shaking. Thus it is evident that these drains are effective in rapid dissipation of excess PWP both during and post shaking phase because of smaller \( r_u \) values, hence useful in mitigating liquefaction risk.

5.2.2 Ground improvement using grouting techniques:

Grouting technique (cement grout, CS grout) of ground improvement is another method to mitigate liquefaction hazard by Cementation and improving shear strength of the material. From the centrifuge tests conducted on the cement block below the raft foundation, it was observed that the settlements of the improved ground were smaller as compared to the benchmark centrifuge test with no ground improvement. In order to keep the ultimate settlement (which in this case was 0.275m) low, it is proposed to extend the improvement of ground up to the full depth of the liquefiable layer or to the point of liquefaction susceptibility. Also the excess pore water pressure is seen to be reduced in the improved ground location after earthquake loading due to monotonic dilation. However, the excess pore water pressure also seems to be larger in the improved ground then in the benchmark test with no ground improvement because of surcharge pressure exerted by the structure and the overburden stress exerted by the cement in block.

From the centrifuge models of colloidal silica (CS) treated samples and untreated ones, it was observed that the strains are significantly lower (~0.5-1%) for treated models as compared to untreated models having strains (3-6%) in the same test. Further, from the centrifuge models of loose sand, it was observed that shear strains recorded are 0.5% and 1% in the first and second shaking respectively, while settlements limited to 30 mm and 10 mm were recorded respectively in the two phases of shaking. The treated soil performed well and did not liquefy during both phases of shaking, hence giving evidence of incorporating this technique in liquefaction mitigation. Also it was observed that CS treated soil controls lateral spreading and settlement, thus found useful to be used as a ground improvement technique for remediation of liquefaction hazard.

Centrifuge modelling of cemented soil and untreated soil (deep, homogenous layer of loose saturated sand and shallow layer of loose saturated sand overlying dense sand) was carried out to assess the effectiveness of the cement grouting as a remediation method. From the testing, it was observed that the settlements in the benchmark centrifuge model with no treatment were large (0.2 to 0.4 m, prototype scale), while as in fully cemented zone, the settlements were negligible. Relatively small settlements were observed in cementing zone passing through loose sand and extending to dense sand layer. It was found that settlements can be negligible if the improvement is done up to a full depth of liquefiable layer or to a point where there is potential of liquefaction. It was observed that under relatively smaller earthquakes, partial depth Cementation is effective when liquefiable layer is deep.
and free field liquefaction occurs above the base of remediated zone. However, for larger anticipated earthquakes, the depth of improvements adopted should be increased to reduce liquefaction hazard. Also increasing the depth of improvement further far beyond the expected free field liquefaction depth, there is no significant reduction in the structural settlements.

The depth of treatment is dependent on the vertical extent of material that is susceptible to liquefaction risk and also the expected degree of settlement. According to US practice, the depth of remediation zone is determined by using SPT, CPT and measurements of shear wave velocities and simplified procedure as given by Seed and Idris (1971). The methods involving vibration and compaction exhibit greater increase in SPT-N value than the methods based on compaction only. Based on the area controlling the stability of the structure, the lateral extent of ground improvement is determined to prevent damage induced liquefaction hazard. Based on centrifugal model testing and field experience, treatment should be extended to a distance equal to a depth of layer densified reported.

5.2.3 Containment walls:

Centrifuge tests were performed on the benchmark (Huston S28 sand) with no ground improvement and compared with centrifugal test results of containment walls to assess the performance of containment walls a remediation for liquefaction. From the results, it was observed that under low shaking intensity for the containment walls extended up to part of liquefiable layer, the settlements of soil and structure are little as high excess PWP are not generated. Under moderate shaking intensity, liquefaction occurs to the base level of containment wall (doesn’t reach below the base), PWP is higher in liquefied region and structural settlement is small because structure is supported by unliquefied soil inside the containment walls. After sometime liquefaction reaches slightly below the base of containment wall and leads to some settlement of the structure and walls. Under strong shaking intensity, liquefaction reaches to a depth below the base of containment walls and then depth increases leading to the sinking of containment walls and structure in the softened soil. Partial depth containment walls are effective in small earthquakes, when liquefaction extends up to a depth less than depth of containment walls. Full depth containment walls are effective in reducing structural settlements even in larger earthquakes. Overall, it was concluded that rigid containment walls are very effective in remediating liquefaction risk by restricting lateral spreading of soil as well as reducing soil volume changes.

6. Conclusions

The analysis methods for liquefaction estimation and assessment depends upon soil and earthquake parameters which include depth of groundwater, fines content, soil thickness, density and shear wave velocity in first case and moment magnitude, dominant period of earthquake and maximum acceleration in the latter case.

Centrifuge modelling was carried out to investigate the performance of some ground improvement techniques as a liquefaction remediation measure. Following conclusions were made:

a) Containment walls are affective in remediating liquefaction risk of existing structures by restricting the lateral spreading of the soil as well as reducing settlements by reducing soil volume changes. Partial depth containment walls are effective in relatively smaller earthquakes, therefore, if large earthquakes are expected, full depth containment walls should be provided.

b) Ground improvement using sand tire chips was found to be effective in liquefaction resistance and having high undrained shear strength when incorporating large sized oven dried chips. Also, oven dried mix achieved a higher relative density (62%) than the pure sand sample ($D_r = 50\%$), hence more resistant to liquefaction risk than pure sand.
c) Using drains (Gravel drains, Sand drains etc) as a liquefaction remediation method, it was found that the drains should be installed as an internal drains, with extra rows of drains on the Periphery to act as a perimeter and sub perimeter drains, to achieve significant improvement in the performance.

d) Grouting techniques such as CS grout, Cementation etc are effective in reducing foundation settlements. For this, site improvement up to full depth of liquefaction susceptible layer should be done to achieve significant improvement in the reduction of settlements, if larger earthquakes are anticipated. Partial depth of improvement is found to reduce settlements under relatively smaller earthquakes.

e) The performance of PVDs during and after earthquake is found to be effective by dissipating excess PWP as well as reducing shear deformations. PVDs also tend to reduce settlements and cause soil densification as the water is drained out.

Analysis of liquefaction by CSR even though being applicable to all soil types, but it does not differentiate between distinct soil types and thin layers. Incorporation of small strain shear wave is not much effective for soils below GWT. Estimation of liquefaction potential using Piezocone tests (CPTU) is not effective in soils with low liquefaction potential and gives better results in soils with high liquefaction potential only. There is a significant improvement in the performance of gravel drains to enhance liquefaction resistance by installing them as internal drains with extra rows on the periphery as perimeter and sub perimeter drains. But in this study, the spacing between rows and drains in a row for this layout is not specified. The lateral extent of ground improvement to prevent liquefaction damage depends upon the area controlling the stability of the structure. There is no emphasis given on the determination of lateral extent of improvement in this study.

Since the ground improvement techniques presented in this study are more effective in small earthquakes and if large earthquakes are anticipated, their effectiveness reduces. The future scope lies in finding the various aspects in the design and construction of these GI techniques that would enhance and improve their effectiveness to prevent liquefaction related damages during large magnitude earthquakes also. To determine the lateral extent of the ground improvement needed based on the area controlling the stability of the structures. Determining the tests, which are effective for estimating the liquefaction of soils with relatively low liquefaction potential. To specify the spacing of Gravel drains to be installed as internal, perimeter and sub perimeter drains for further enhancement of their liquefaction resistance.

7. References

[1] Singh G, Pruncu C I, Gupta M K, Mia M, Khan A M, Jamil M, Pimenov D Y, Sen B and Sharma V S 2019 Investigations of machining characteristics in the upgraded MQL-assisted turning of pure titanium alloys using evolutionary algorithms Materials (Basel). 12

[2] Sarowa S, Singh H, Agrawal S and Sohi B S 2018 Design of a novel hybrid intercarrier interference mitigation technique through wavelet implication in an OFDM system Digit. Commun. Networks 4 258–63

[3] Khan A M, Jamil M, Mia M, Pimenov D Y, Gasiyarov V R, Gupta M K and He N 2018 Multi-objective optimization for grinding of AISI D2 steel with Al2O3 wheel under MQL Materials (Basel). 11

[4] Ramteke D D, Balakrishna A, Kumar V and Swart H C 2017 Luminescence dynamics and investigation of Judd-Ofelt intensity parameters of Sm3+ ion containing glasses Opt. Mater. (Amst). 64 171–8

[5] Sidhu B S, Sharda R and Singh S 2021 Spatio-temporal assessment of groundwater depletion in Punjab, India Groundw. Sustain. Dev. 12

[6] Chohan J S, Kumar R, Singh T B, Singh S, Sharma S, Singh J, Mia M, Pimenov D Y, Chattopadhyaya S, Dwivedi S P and Kaplonék W 2020 Taguchi S/N and TOPSIS Based Optimization of Fused Deposition Modelling and Vapor Finishing Process for Manufacturing
of ABS Plastic Parts Materials (Basel). 13 5176

[7] Velusamy K, Devanand J, Senthil Kumar P, Soundarajan K, Sivasubramanian V, Sindhu J and Vo D-V N 2021 A review on nano-catalysts and biochar-based catalysts for biofuel production Fuel 306

[8] Sharma K, Castello D, Haider M S, Pedersen T H and Rosendahl L A 2021 Continuous co-processing of HTL bio-oil with renewable feed for drop-in biofuels production for sustainable refinery processes Fuel 306

[9] Yao Y, Zhang M, Deng Y, Dong Y, Wu X and Kuang X 2021 Evaluation of environmental engineering geology issues caused by rising groundwater levels in Xi’an, China Eng. Geol. 294

[10] Shen R, Lu J, Yao Z, Zhao L and Wu Y 2021 The hydrochar activation and biocrude upgrading from hydrothermal treatment of lignocellulosic biomass Bioresour. Technol. 342

[11] Nagula S S, Hwang Y-W, Dashti S and Grabe J 2021 Seismic site response of layered saturated sand: comparison of finite element simulations with centrifuge test results Int. J. Geo-Engineering 12

[12] Hu J 2021 A new approach for constructing two Bayesian network models for predicting the liquefaction of gravelly soil Comput. Geotech. 137

[13] Gao B, Ye G, Zhang Q, Xie Y and Yan B 2021 Numerical simulation of suction bucket foundation response located in liquefiable sand under earthquakes Ocean Eng. 235

[14] Buffo M M, Ferreira A L Z, Almeida R M R G, Farinas C S, Badino A C, Ximenes E A and Ladisch M R 2021 Cellulolytic enzymes production guided by morphology engineering Enzyme Microb. Technol. 149

[15] Ponomarchuk E M, Rosnitskiy P B, Khokhlova T D, Buravkov S V, Tsyasar S A, Karzova M M, Tumanova K D, Kunturova A V, Wang Y-N, Sapozhnikov O A, Trakhtman P E, Starostin N N and Khokhlova V A 2021 Ultrastructural Analysis of Volumetric Histotripsy Bio-effects in Large Human Hematomas Ultrasound Med. Biol. 47 2608–21

[16] Padmanabhan G and Shanmugam G K 2021 Liquefaction and reliquefaction resistance of saturated sand deposits treated with sand compaction piles Bull. Earthq. Eng. 19 4235–59

[17] Kusumawardani R, Chang M, Upomo T C, Huang R-C, Fansuri M H and Prayitno G A 2021 Understanding of Petobo liquefaction flowslide by 2018.09.28 Pulu-Donggala Indonesia earthquake based on site reconnaissance Landslides 18 3163–82