BE CIVIL ENGINEERING

FINAL YEAR PROJECT

SOIL LIQUEFACTION ANALYSIS IN RISALPUR CANTONMENT AND DHA PESHAWAR SITE, NISATTA CHARSADDA

Project submitted in partial fulfilment of the requirements for the degree of
BE Civil Engineering

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This is to certify that

BE Civil Engineering

Final Year Project entitled

Soil Liquefaction Analysis in Risalpur Cantonment and DHA Peshawar Site, Nisatta Charsadda

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DEDICATION

To our dear Parents whose belief in us, support, prayers and best wishes always accompanied us during this span and accomplishment of this project

AND

Worthy Instructors whose dedicated, selfless assistance and guidance paved our way towards the completion of this project and accomplishment of our Civil Engineering degree.

ACKNOWLEDGEMENT

Starting with the name of ALLAH Who is the most Gracious and the most Compassionate. It is a great honor for us to present our civil engineering knowledge in its practical form. Greatest thanks to **ALMIGHTY ALLAH** for empowering us with the strength to complete this project and guiding us through all the difficulties.

After that, our sincere gratitude to our supervisor **Col Nawab Ali** for supporting, encouraging and guiding us through virtue of his life time knowledge and experience in the field of Geotech engineering. We are also grateful to all other faculty members of Geo-tech department for their guidance in our hour of needs.

ABSTRACT

Soil liquefaction is an important geotechnical Phenomenon prevalent in saturated granular soils. These soils upon receiving dynamic loading (like earthquake) consolidate and the excess water liquidates the soil resulting in loss of strength and eventual settlement and collapse. There is not much awareness about this phenomenon and its mitigation in Pakistan. The research aims to highlight the Soil Liquefaction phenomenon and it will eventually assist the government authorities to develop guidelines for sustainable land development and put forth the remedial measures for soil liquefaction.

Liquefaction potential of two sites, Risalpur Cantonment and DHA Peshawar site at Nisatta Charsadda, was assessed. These sites, within 20 km of each other, are located in zone 2B as per Building Code of Pakistan (BCP, 2007) and pose significant seismic hazard but they offer different liquefaction potentials. Liquefaction potential of these two sites was analyzed against earthquakes of magnitude 6.5 to 8.0 by the simplified Stress based approach using SPT which yields Factor of safety value of the site against liquefaction.

Typical profile of subsoil in Risalpur constitutes of Silty Clay (CL - ML) with a thin middle layer of poorly graded sand (SP). Water table is deep and not encountered during boring but for the purpose of this study, we are considering a hypothetical situation where Risalpur's water table suddenly rises during an event of flooding and heavy rains (2010 flooding). Results show that even when the water table is high, Risalpur soil is not Liquefiable except the middle SP layer which liquefies at earthquake magnitude 7.5 and above. However this layer will not cause any liquefaction related problems on the surface as its 16 ft deep bounded by non-liquefiable layers and only 3 ft thick.

DHA site is located on bank of river Kabul. Subsoil profile consists of poorly graded sand (SP) overlain by silty sand (SM). Water table is 7 ft deep. Results show that DHA soil is highly liquefiable and liquefaction was observed till 30 ft depth at M=6.5 earthquake with the depth of liquefaction increasing with the earthquake magnitude. Significant settlement and lateral spreading is anticipated on the surface.

Concluding the results, any construction project without considering liquefaction mitigation measures must be avoided at DHA Nisatta, Charsadda. Potential remedial measures are proposed in this study to counter the effects of liquefaction for future infrastructure Projects.

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Chapter 01

1 INTRODUCTION

1.1 Background

We selected two target area for our study of liquefaction analysis. These areas will be discussed below separately in detail. The reason of selecting multiple areas is to get diverse geological conditions and underground strata to make this study rich and acquaint ourselves and the readers to different set of circumstances related to the subject of liquefaction.

1.1.1 Liquefaction

Liquefaction takes place when loosely packed, water-logged sediments at or near the ground surface lose their strength in response to strong ground shaking. Liquefaction occurring beneath buildings and other structures can cause major damage during earthquakes. There is not much awareness about this phenomenon and its mitigation in Pakistan. The research will resultantly educate students by bringing out some possible remedies against liquefaction phenomenon against potential soils and sites.

Figure 1.1 Soil Liquefaction, June 16, 1964 Niigata Japan

1.2 Locations

1.2.1 Risalpur Cantonment

Risalpur is small city situated in the District of Nowshera, KPK, on Nowshera-Mardan Road. It is some 45 km East of Peshawar and situated at $34^0$4’52’’N $71^0$58’21’’E.

Located in a basin about 1014 feet (309 m) above the sea level, it is surrounded on the west and south by Kalpani and Kabul rivers, respectively

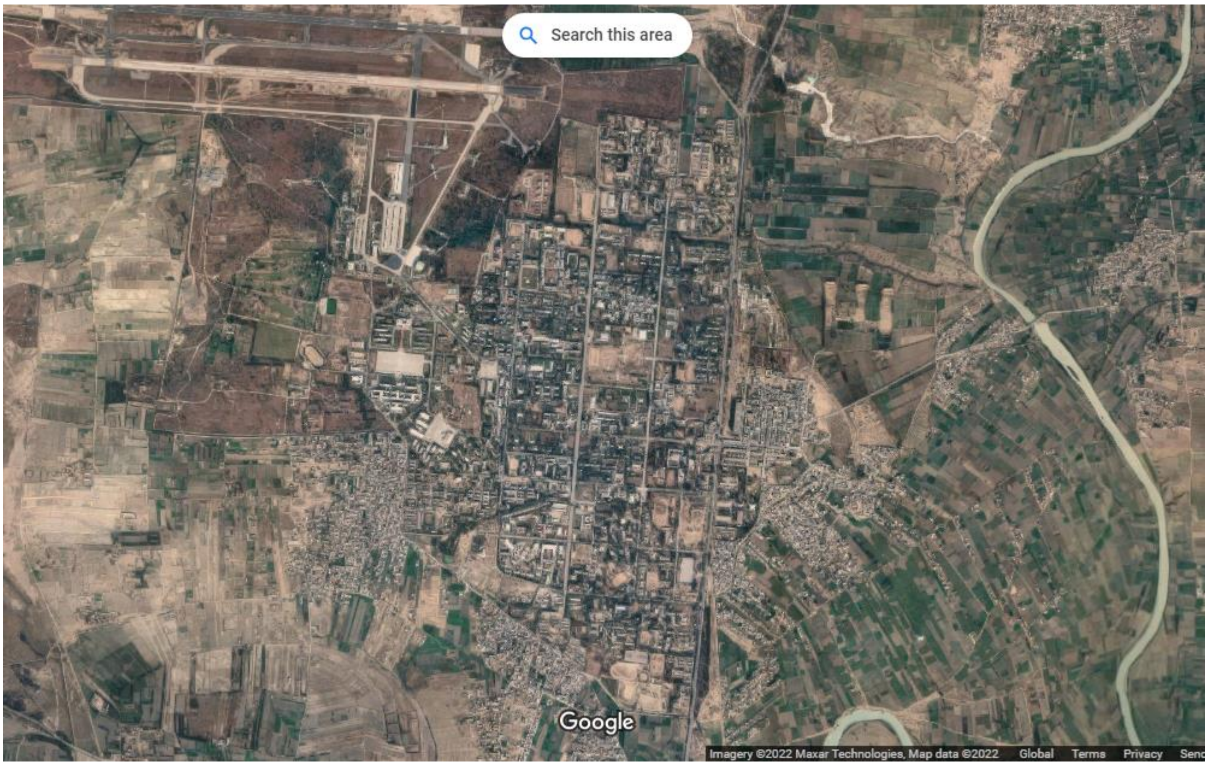

Figure 2.2 Satellite imagery of Risalpur cantonment

1.2.1.1 Importance of the Site

Risalpur is home to the Military College of Engineering, NUST. Besides, PAF base Asghar Khan and Air Force Academy, Engineers Centre and CAE NUST are all situated in Risalpur.

With a population of over 42,000 permanent residents according to 2021 census, it is a strategic city for Armed forces of Pakistan with multiple installations.

Lying 90 kilometers south of the famous Khyber Pass it served as the Staging post for Royal British Army and Royal Air Force from 1910 to 1947.

1.2.2 DHA Peshawar Site

DHA Peshawar is a proposed project on the site of Nisatta, a town located in Charsadda District, Khyber Pakhtunkhwa at a distance of 40 kilometers from Peshawar.

It is located on 34^{0}6’8’’N 71^{0}47’47’’E and at an altitude of 912 feet (277m). Four smaller rivers meet in Nisatta forming a larger river, River Kabul.

Figure 3.3 Satellite imagery of DHA Peshawar site Nisatta Charsadda

1.2.2.1 Importance of the Site

Having a local population of above 90,000, this site was selected for the proposed DHA Peshawar project. With three major cities Peshawar, Mardan and Nowshera around; this site can house a large population for the DHA housing scheme.

1.3 Data Acquisition

1.3.1 Risalpur Cantonment

Being in MCE, we collected the samples/data easily from the first site i.e. Risalpur Cantonment by hiring a local contractor for boring and performed the SPT on bore hole. The borehole was dug at the site of construction of Officer's Mess in Risalpur Cantonment.

The other site posed considerable constraints of logistics and schedule.

1.3.2 DHA Peshawar Site

As the second site, Nisatta Charsadda District, is quite remote and was off-limits; we were advised not to go there in person and rather use the bore log data of the site held with Geo-technical lab, MCE. The data was shared by the Lab and then analyzed by our group. The data comprised of 50 boreholes dug all around the proposed construction site of DHA Peshawar along the bank of Kabul River.

1.4 Aim

Main objective of this study is to analyze the liquefaction probability of the two said sites by reading the geological and seismic conditions and exploring the subsurface strata of respective areas. Upon encountering the liquefaction, remedial measures will be suggested for the respective site to help the local authorities in implementing the same.

Further, with the help of this study, we aim to highlight the importance of this naturally occurring phenomenon of liquefaction for the government authorities and general public. It will assist concerned authorities in developing recommendations for sustainability as well as mitigation strategies. In latest years, there has been a great increase in development and construction activity, and liquefaction consideration is becoming increasingly critical for land use design and infrastructure development.

As during the execution of even mega construction projects, only the bearing capacity reports are prepared and the assessment of liquefaction is generally neglected, by the means of this research paper, we request the concerned government authorities to make the preparation of liquefaction hazard analysis reports compulsory along with other necessary reports before the commencement of the project

To achieve the aim of analyzing the liquefaction potential, we assessed the two sites by using a simplified stress-based approach i.e. Standard Penetration Tests, SPT, for assessing the safety factor against liquefaction of soil.

1.5 Objectives

To achieve our aim, following are the objectives need to be fulfilled:

1. To Analyze the liquefaction potential of Risalpur Cantt and DHA, Nasiita Charsadda soils.
2. To Find the Factor of Safety (FS) of these sites against a deterministic Earthquake.
3. To Model these sites on the NovoLiq/ LiqIT software with collected data and analyze/compare the results.
4. To Suggest remedial measures to mitigate the effects of liquefaction.

1.6 Report Composition

This paper has been divided into six chapters describing the overall study in a stepwise procedure:

Chapter 01 Introduction:
A brief description of the project with aim and scope

Chapter 02 Soil Liquefaction:
Describes the phenomenon of Soil Liquefaction, its causes and effects.

Chapter 03 Literature Review:
Review of a case study of soil liquefaction (Niigata Earthquake of 1964) and a paper on analysis of liquefaction potential.

Chapter 04 Methodology:
Field and Lab tests, Data acquisition, Software modelling and plan of action is explained

Chapter 05 Analysis and Results:
Explains the analysis on the basis of test results and the outcome.

Chapter 06 Conclusion and Recommendations:
Comparison of the results and recommendations as per the findings

Chapter 02

2 SOIL LIQUEFACTION PHENOMENON

2.1 General

The concept of soil liquefaction feels new and exciting because we have only been studying it for the last 50 years or so which comparatively is not long ago at all if we think about science itself. The truth is that Soil Liquefaction has always occurred with earthquakes, but it really was not until the 1960s that we started to recognize it as an actual separate effect from the earthquake or the ground motions that were occurring and began to understand its mechanism.

Something was going on beneath the layers of ground and people did not understand what was actually happening. When people saw incidents like manholes bursting out of the ground that made them wonder what the forces behind this phenomenon are.

Figure 4.1 Uplifted buried structures after 2004 Niigata Chuetsu, Japan

2.2 Soil Liquefaction Phenomenon

The process by which a saturated soil loses a significant amount of strength and load carrying capacity in reply to applied stress, primarily seismic shaking or another rapid change in stress situation, leading it to behave like a liquid . This process is known as liquefaction.

2.2.1 Working Principle

Soil is made up of particles that are attached to one another. Due to gravity, these particles spontaneously stack on top of one another and form grid shapes based on their attributes (especially in loose granular soils). Empty spaces or pores are created in between the particles due the resulting grid which upon saturation are filled with water. Every particle creates its contact force with the particles around it. These attached forces subsequently keep all of the individual soil particles in their respective positions. Liquefaction in soil happens when these soil particles and the water between them are loaded rapidly and suddenly. These particles start floating in the water due to the rapid pressure of the water. As a result, when the soil loses its cohesiveness, it softens, dissolves, and loses its solid attributes, which are replaced by liquid attributes.

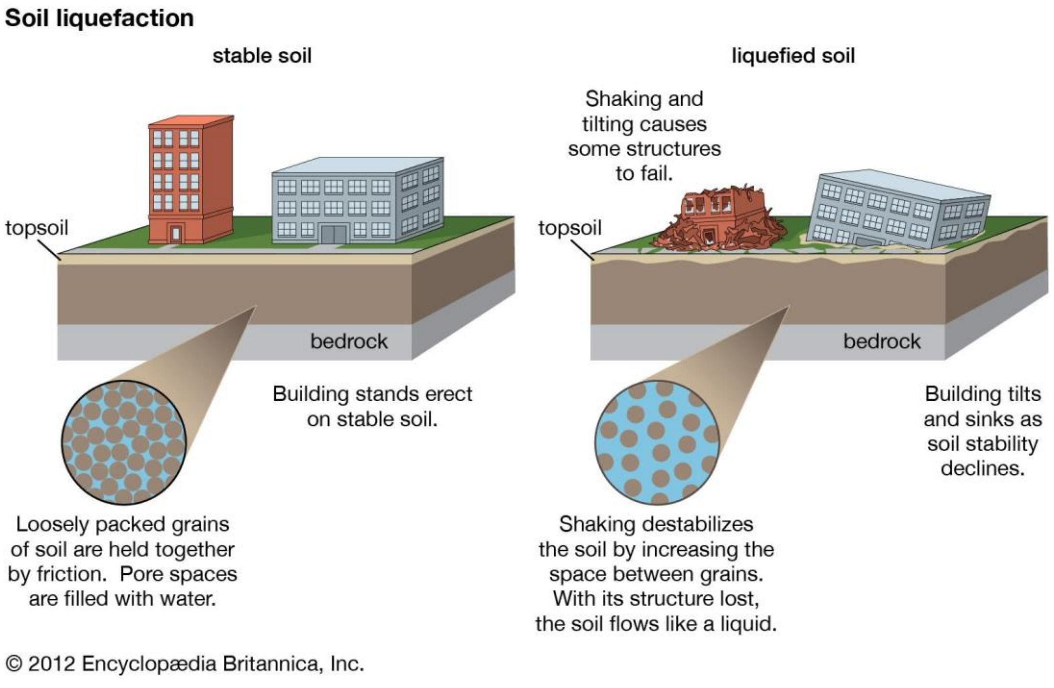

Figure 5.2 Working mechanism of Soil Liquefaction

2.2.2 Causes of Liquefaction

Soil liquefaction occurs when the effective stress or shear capacity in the soil are reduced to almost zero. This can occur due to monotonic loading (a single, abrupt change in stress – for example, an increment in weight on an embankment or the removal of toe support) or cyclic loading (periodic variations in stress state – for example, wave loads or earthquake movement). The most prone soil to liquefy is a soil in a loose saturated condition and one that may create enough pore water pressure in response to changes in load. This is due to the fact that sheared loose soil has a propensity to shrink in volume. The reduction in volume is related to the rearranging of soil particle geometry.(Papadopoulos and Lefkopoulos 1993). After this rearrangement, soil particles are more densely packed together leaving little space in between for pore water.

The decrease in volume generates large excess pore water pressure. There are two possibilities for the pore water pressure. If the soil layer is well drained, the excess pore water will drain away quickly leaving a dense and compacted layer of soil. But if that soil layer is poorly drained, the pore water has nowhere to go. As pore water pressure rises, it separates the soil particles from each other and they start floating in a suspension. Effective stress decreases and loss of bearing capacity occurs which results in quicksand.

Liquefaction is much more probable to occur in relatively newly formed deposits (Holocene period, last 10,000 years), mainly poorly graded sands and silts (well-sorted), in layers at least meters thick, and soaked with water. These deposits are most usually seen near riverbeds, beaches, dunes, and other locations where windblown sand and silt (loess) has formed.

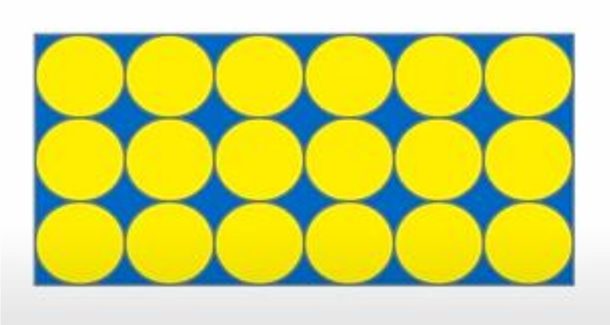

Figure 6.3 Grid Structure of Granular Soils saturated with water

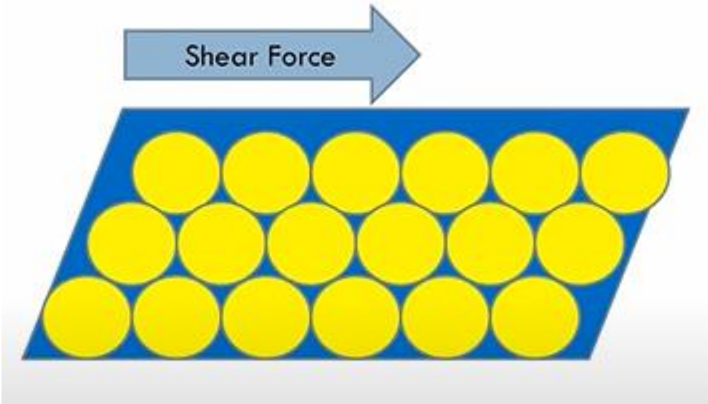

Figure 7.4 Shear force rearranges and densifies soil grains causing water to squeeze out of pores

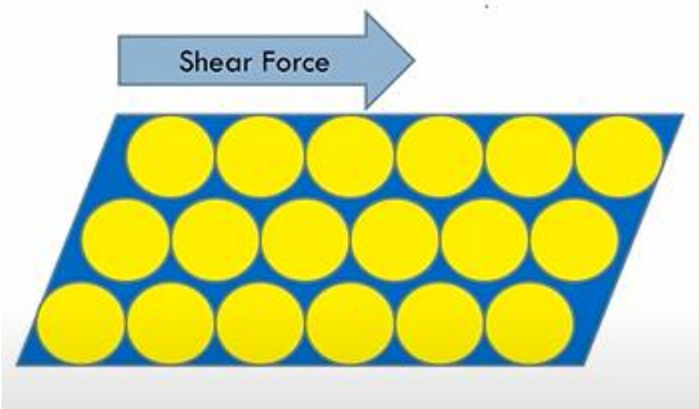

Figure 8.5 Particles float in water when there is no drainage and soil liquefies

2.2.3 Effects of Liquefaction

Two of the most important effects to consider in conjunction with soil liquefaction are seismic compaction i.e. settlement and lateral spreading. But there are numerous other possible destructive effects of soil liquefaction depending upon the landscape and soil characteristics like sand boils and bearing capacity failures.

2.2.3.1 Seismic Compaction

Another more common term for seismic compaction that we use today is liquefaction induced settlement.

One of the most common effects of liquefaction is settlement. As the soil particles tries to rearrange and contract they squeezes out the pores and pore water. Thus, significant volumetric strains could occur.

Technically, sand does not need to liquefy in order to experience these types of volumetric strains or settlements. In fact there is quite a body of research that predicts seismic compaction in dry sands. So we can experience seismic compaction in both dry and saturated sands.

As numerous other factors can contribute to the settlement, including the weight of the structure, the concept of seismic compaction is strictly related to the “Free-Field” compaction only.

2.2.3.2 Lateral Spread

Lateral spread is the horizontal deformation of the soil. We can think of it as this way that seismic compaction is the vertical deformation of the soil due to soil liquefaction and lateral spread is the horizontal deformation of the soil.

Lateral spread is one of the most damaging hazards associated with the soil liquefaction. It poses tremendous risks to the lifeline things like bridges, ports, roads, railways, pipelines etc. Anything which is not tolerant to stretching is likely to be damaged by lateral spreading.

Let’s say we have a cross section of a bridge built on a drilled shaft foundation with a subsoil profile shown in figure. So if the earthquake occurs, the loose sand layer will be liquefied and loses its shear strength.

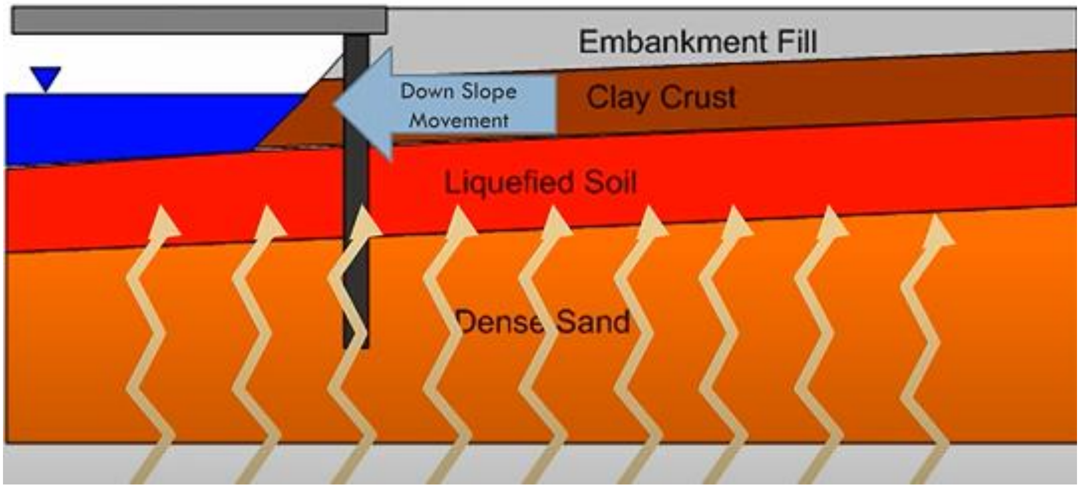

Figure 9.6 Fill Layer on top of Liquefiable layer

As there is an open face of overlaying clay layer on the river side, there is no resistance for it to move in that direction. As a result, there is a downward slope movement of the clay layer and the embankment fill on top of the weakened or softened liquefied soil and it could result in vertical cracks and sand boils leading the ground to shift in the direction of the river which can cause damage to the bridge and its foundations.

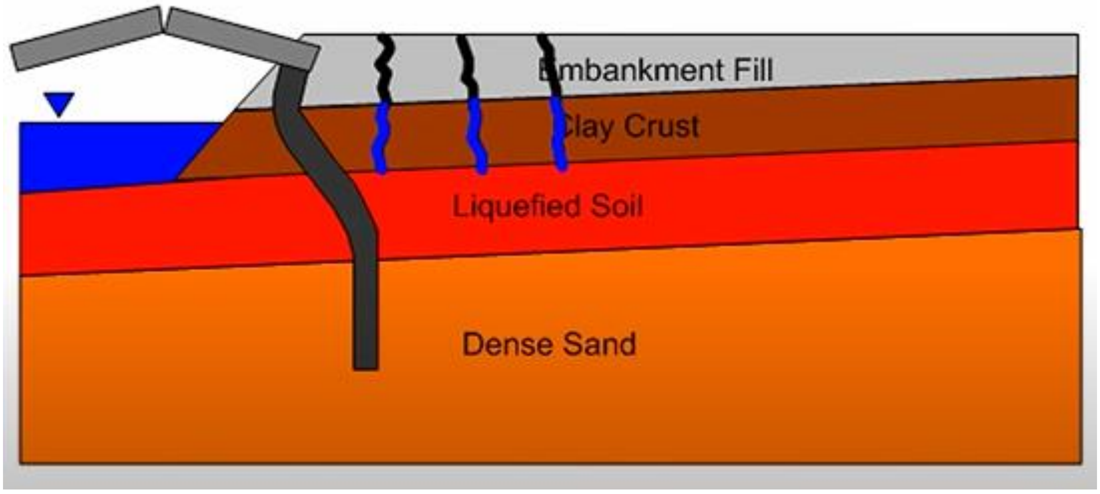

Figure 10.7 Lateral Spreading along the slope after Liquefaction

Figure 11.8 Lateral Spreading causing bridge spans to fall off from its supports, 1964, Niigata Japan

2.2.3.3 Sand Boils

When pressured water wells up through a layer of sand, sand boils, also called as sand volcanoes, erupt. The name comes from the appearance of water bubbling up from beneath the sand. The discharge of sand onto to the surface from a center well with gradients at the sand angle of repose creates a sand cone. At the summit, there is usually a crater. A cone appears to be a small volcanic cone with a diameter of a size varying from millimeters to meters.

In an earthquake, this mechanism is frequently combined with liquefaction in soil, which can be found in saturated sediments. When a saturated loose sand layer is underlain by a layer of low permeability e.g. clay and experiences liquefaction, excessive “pressure is released through cracks or weak zones present in the overlying clay layer. This rising water takes sand particles with it and deposit them on the surface giving the look of a sand volcano.

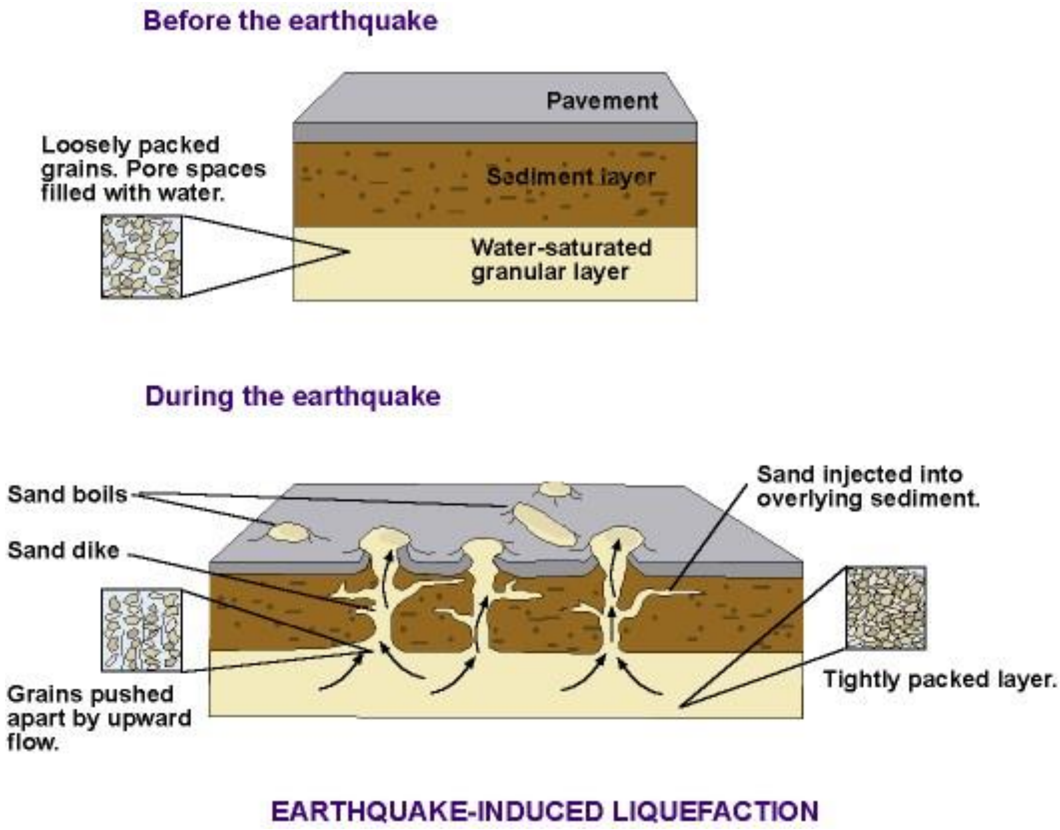

Figure 12.9 Sand boil during earthquake induced liquefaction

2.2.3.4 Bearing Capacity Failure

As a result of soil liquefaction, footings in cohesion less soils may experience a drop in bearing capacity, as well as experiencing settlement and tilting. As a result, structures with shallow footings are more likely to settle and tilt.

The fall in the value of the angle of internal friction becomes obvious during the liquefaction condition during the first few seconds of the earthquake, resulting in a quick bearing capacity failure. As with quicksand, the extra pore water pressure causes the soil particles to split apart and form a suspension.(Cubrinovski, Bray et al. 2011)

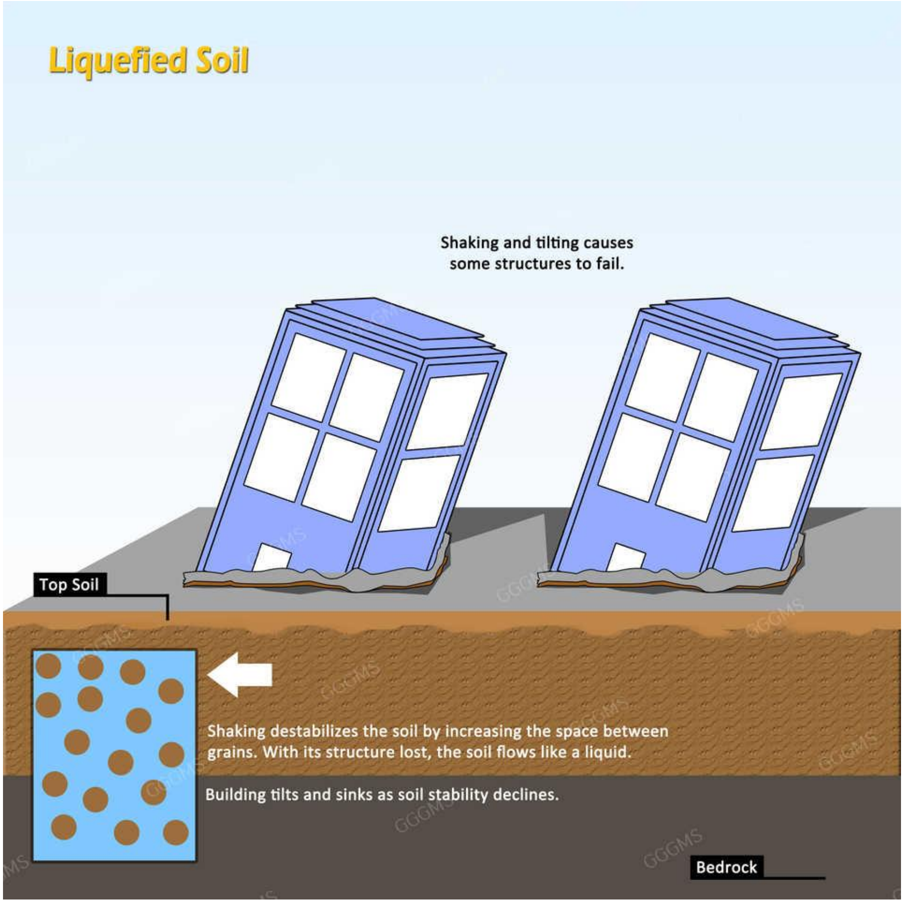

Figure 13.10 Bearing capacity failure during soil liquefaction

Chapter 03

3 LITERATURE REVIEW

3.1 General

Seismic occurrences, such as earthquakes, generate a succession of soil disturbances that can destroy or jeopardize a structure's structural stability, which could become hazardous. The liquefaction process generates a sudden change in motion which is not in synchronization with that of structure. It can result in variety of structural failures to the property, and also the loss of life. In soils which are saturated, liquefaction causes a quicksand phenomenon. It takes place as liquefaction is happening, causing the building or footing to be pushed into the soil, tilting and finally collapsing. Retaining walls, which depend on the strength and rigidity of ground, are used in the construction of structures which are near to water bodies. The retaining wall collapses when the ground liquefaction occurs, resulting in land sliding.

Seismic activity has a negative impact on the ground. Buildings suffer structural damage as a result of soil liquefaction. This occurs as a result of a variety of structural defects. The strains of liquefied soil's foundation load are too great for it to endure. As the foundation descends into the sand, the building will sag and inevitably collapse. Only places with saturated soils experience soil liquefaction. The bulk of these locations are near bodies of water like rivers, ponds, and lakes.(Seed and Idriss 1971)

Codes and regulations must be scrupulously followed when constructions are built in certain locations. Under normal circumstances, the terrain can support ground stresses. But an earthquake or intense ground movement causes water logging, which enhances the soil's liquid fluidity. Because the soils has lost its rigidity and can no longer withstand the weights, the structure above it sinks or collapses.

3.2 Liquefaction Criteria

There are various different procedure and methods to assess and evaluate the liquefaction potential of a soil deposit. Not every soil is liquefiable and most of the soil parameters have to be inside the limits of certain criteria for it to be liquefiable. The criteria is broadly divided into four major categories and these are discussed below.

3.2.1 Historical Criteria

We can deduce a lot related the liquefaction occurrences in various types of soils and locales by observing previous earthquakes. Soils which have already experienced liquefaction previously may liquefy again during subsequent earthquakes. When you're planning to build a structure and want to know if the site is prone to liquefaction phenomenon, look into previous earthquakes to see if they resulted in liquefaction. There are also maps depicting areas where liquefaction has occurred previously or may occur in the future time. Several municipal governments have prepared maps of risky zones, which include areas which are more likely to liquefy. These maps may be present in a nearby library or the building department of that locality.

3.2.2 Geological Criteria

The nature of geologic phenomenon that formed a soil strata has a significant impact on its vulnerability to liquefaction. The Saturated zone deposits (fluvial and alluvial deposits), accumulation of eroded material (colluvium deposits) or sedimentation caused by wind activity (Aeolian deposits) can all be susceptible to liquefaction. These mechanisms stack particles into regular sizes and form a loose state which can lead to densification when earthquakes occurs. Densification causes pore water pressure to rise and shear strength to fall. Man-made soil deposits, especially those formed by the hydraulic filling, may also be prone to liquefaction.

The compositional and state criteria are more important for a thorough understanding the liquefaction process, and are discussed in detail below.

3.2.3 Soil Compositional Criteria

The soil type affects liquefaction susceptibility. Clay soils, especially sensitive soils, also soften under strain in a same way to liquefied soil, although this liquefaction in clays is different to that of sandy soils.

Soils having a uniform particle sizes are quite more prone to liquefaction than soil having a diverse range of particle sizes. When a soil with a range of particle sizes is shaken, smaller particles fill in the spaces between the larger particles, minimizing the potential for density increase and the creation of excess pore pressure.

The geological process explained previously tend to create rounded particles. The friction will be higher in particles having angular shape than in rounded shape, so a soil having angular particles is stable and less prone to liquefaction.

As previously stated, a uniformly graded soil is somewhat more prone to soil liquefaction than a well-graded soil because the latter's reduced susceptibility for volumetric strain limits the amount of excess pore pressure that can form under un-drained strata.

Sandy strata were once understood to be the only type of soil that could liquefy but liquefaction has been reported in gravel and silt. Fine-grained soils can also be strain-softened to produce liquefaction-like phenomena. Fine-grained soils are likely to show this type of phenomenon if the criteria listed below is satisfied.(Seed and Idriss 1971)

- Fraction finer than (0.005 mm< 15%)
- Liquid Limit, (LL < 35%)
- Natural water content (> 0.9 LL)
- Liquidity Index (< 0.75)

3.2.4 Initial State Criteria

The initial condition of a soil is reflected by its density and effective stresses when it is subjected to sudden loads. At a given effective stress level, looser soils are far more prone to liquefaction than compacted soils. Soils associated with high effective stress are more vulnerable to destabilization than soils subjected to relatively low stress for the same density.

A range of factors might have an impact on the state of a soil deposit. Here are a handful that are particularly vulnerable to liquefaction. The liquefaction resistance increases with relative density, Dr, at a fixed confining pressure, and it increases with increasing confining pressure at a constant relative density. Several investigations have demonstrated that the presence of pre-existing shear static stress in a soil deposit is critical to its vulnerability to stable liquefaction. The larger the initial shear stresses are, the greater the liquefaction potential and the less disturbance is required to liquefy that soil.

3.3 Liquefaction Case Study of Niigata Earthquake, Japan, 1964

3.3.1 Background

The renowned 1964 Niigata Earthquake was chosen as the study's subject since it was one of the earthquakes that prompted geotechnical research into the liquefaction phenomenon and its mitigation techniques. As a case study, a past paper "Case Studies of Liquefaction in the 1964 Niigata Earthquake" (Ishihara, 1981) was studied. In this study, a variety of geotechnical approaches were employed to evaluate liquefaction effects and the hazards posed by liquefied soil.

3.3.2 1964 Niigata Earthquake

The devastating Niigata earthquake struck on June 16, 1964. It had a 7.5 magnitude and killed 36 individuals while injuring 385 others. Despite the fact that the earthquake devastated a large area of Japan, the Niigata Prefecture was the hardest hit in terms of destruction of infrastructure. As a result, this paper will concentrate on this specific piece of land. One of Niigata's most striking qualities was the fact that this was the first time an earthquake titled or overturned multiple structures. The fact that several of the afflicted structures exhibited no destruction of the superstructure was even more surprising.(Ishihara and Koga 1981)

Figure 14.1 Tilted Apartment buildings in Kawagishi Cho, Japan, 1964

3.3.3 Tests for Liquefaction Conducted by Ishihara

“Kenji Ishihara collected soil samples from two locations near the Niigata City acceleration monitoring station, one of which had seen substantial liquefaction and the other of which was relatively undisturbed. Both locations were near the Shinano River. The Southbank site is made up of shallow compact sand layers that had minimal ground damage or structure tilting. Kawagishi-cho, the second site, was made out of non-engineered fill that had not been compacted. The buildings on this location had settled more than 50 cm and were severely tilted.

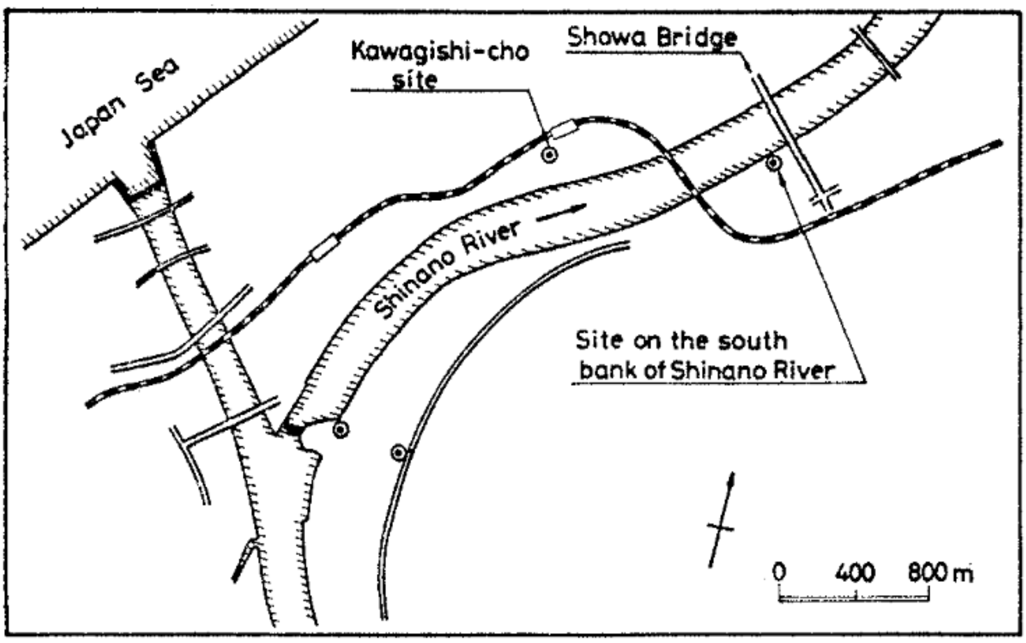

Figure 15.2 Location of Kawagishi Cho & South-Bank site, Niigata (Ishihara, 1997)

The soil at the Kawagishi-cho location was found to be constituted of a 20-meter deep layer of clean sand deposits, according to the standard penetration test. Figure shows that the first 11 meters were made up of medium sand, whereas the layer from 11 meters to 20 meters was mainly composed of medium fine sand. The SPT at the Southbank site, on the other hand, revealed a more varied soil profile, with medium sand, silt with sand, peat, less than 2 meters of medium fine sand, around 9 meters of fine sand, and a thin layer of medium sand (Figure). No samples were taken deeper than 5 meters because the sand was too dense. The Osterberg Piston sampler could only be used on loose to medium sand with a blow count of less than 20. The blow count of the Standard Penetration Test and the penetration resistance of the Cone Penetration Test both followed the same pattern with depth, according to the findings. To put it another way, a higher blow count was linked to a higher tip resistance score".(Ishihara and Koga 1981)

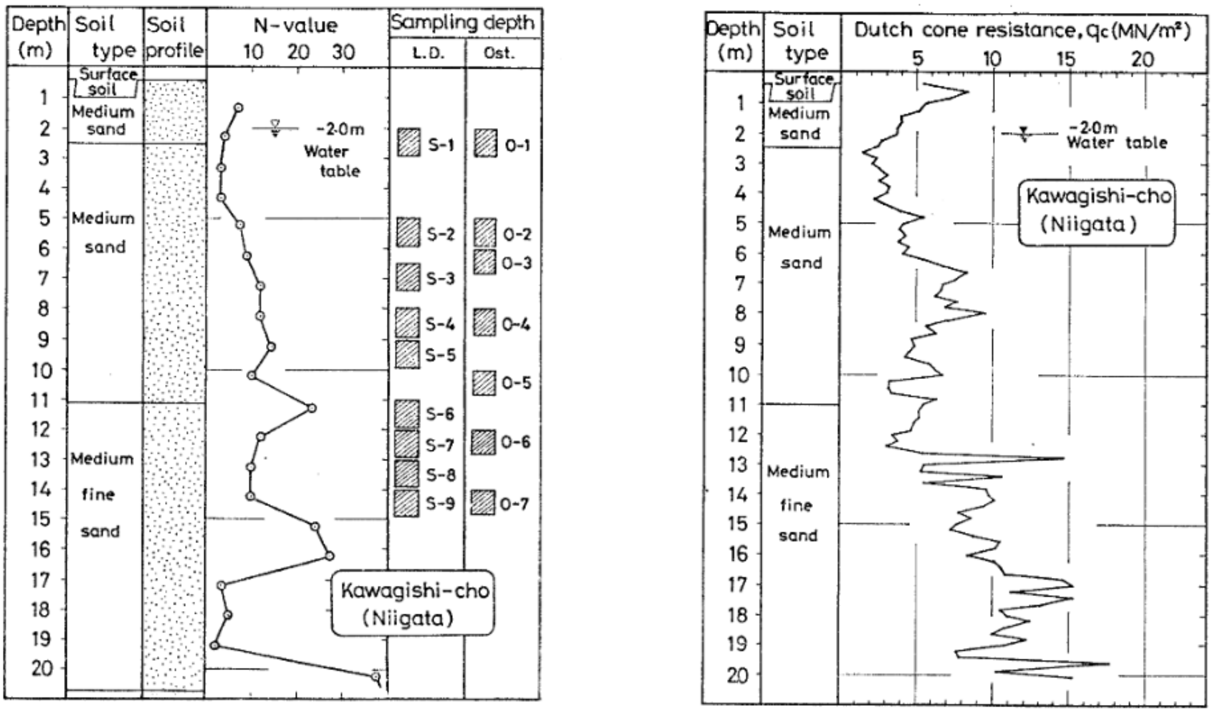

Figure 16.3 SPT & CPT results of Kawagishi Cho site (Ishihara, 1997)

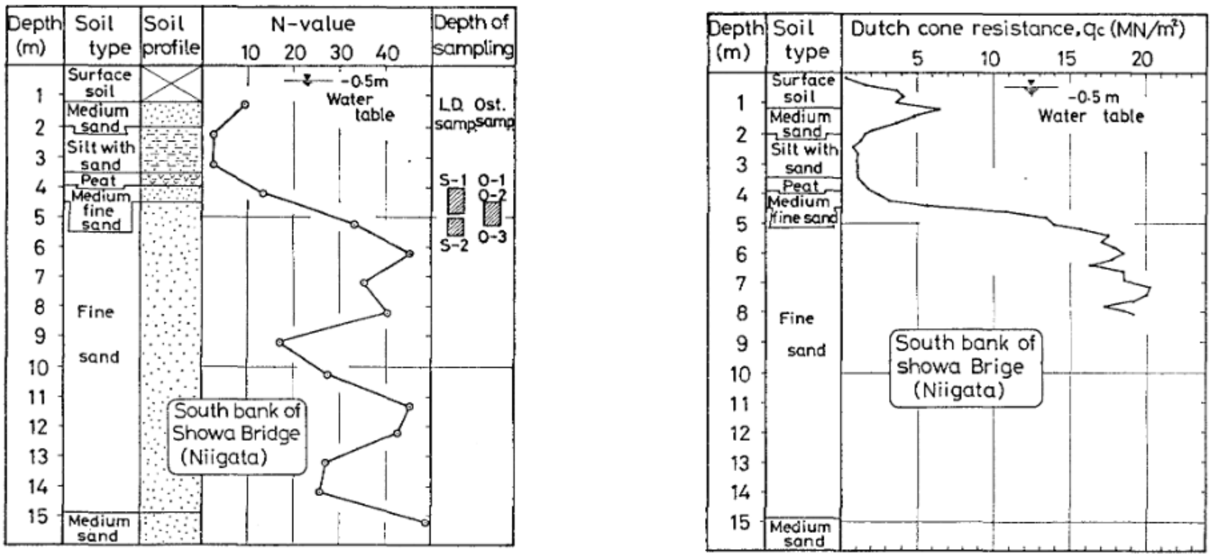

Figure 17.4 SPT & CPT results of South-Bank site (Ishihara, 1997)

The liquefaction potential or cyclic strength of the soils was determined using cyclic tri-axial testing. These tests measure the soil's ability to resist shear stresses when it is subjected to periodic loading, such as an earthquake. The cyclic stress ratio is calculated by multiplying the deviator stress by the reciprocal of two times the effective consolidation stress. The ratio of cyclic stress is plotted against the number of cycles in the tri-axial strength curves. The Factor of Safety can be computed by dividing the cyclic resistance by the cyclic stress. Acceleration data was used to calculate the cyclic stress ratio. All of this analysis was used to figure out how the soil's factor of safety changes with depth. When the factor of safety is less than one, it means that the soil in that stratum is prone to liquefaction.

3.3.4 Results

From roughly 5m to 11m, the Kawagishi-cho location was having a factor of safety less than 1. This layer of liquefied strata is around 6 meters in thickness. The reason of liquefaction at this site is the presence of loose sand which is having a low relative density.

The soil at south-bank site is non-liquefiable except at a depth of 4.5 meters. The factor of safety at this depth was less than one for this thin layer (1 meter) of soil (Figure). But, there was no sign of liquefaction destruction above the ground level at this location. The thickness of layer which has been liquefied impacts whether there would be any liquefaction related effects on the surface. So a lesser thickness of this layer can be a logical reason for the absence of any liquefaction related damage at this site.

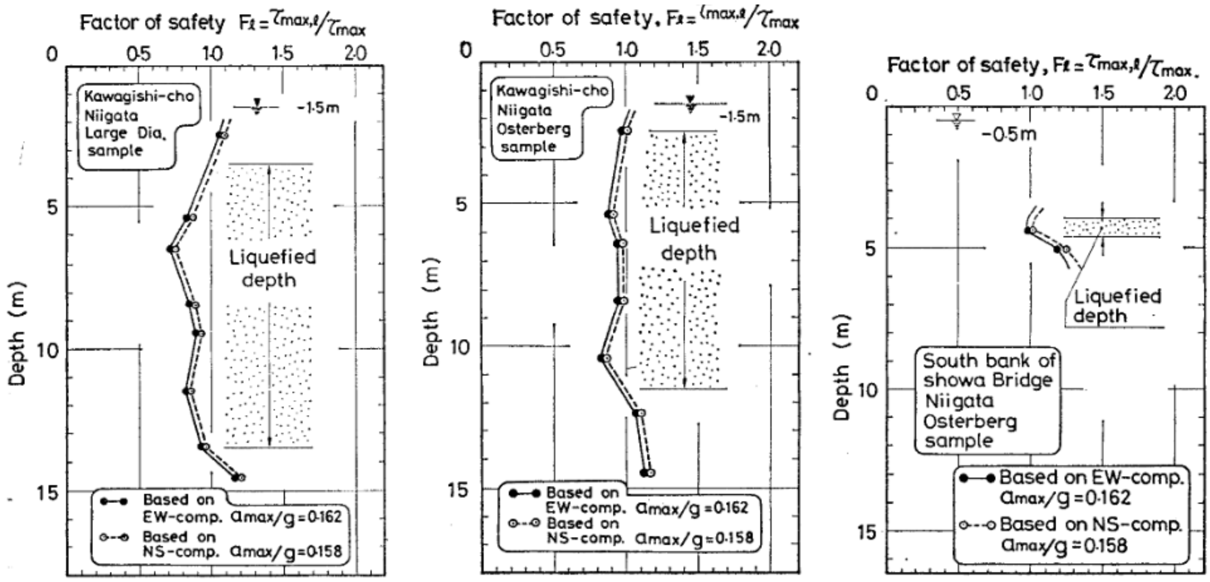

Figure 18.5 Factor of Safety at different depths at Kawagishi & South-Bank site

3.4 A Study of Liquefaction Potential in Kamra, Pakistan

3.4.1 Background

Kamra is notorious for its capacity for soil liquefaction, and numerous geo-tech experiments have been conducted there with similar outcomes. As a result, these researches can be used as a source of information and as a guide. This study reviews and discusses a recent research paper titled “Analysis of Soil Liquefaction Potential in region of Kamra, Pakistan” (Ahmed et al., 2018).

This paper mentions various methods for liquefaction evaluation proposed by different geotechnical engineers, but for the purposes of this study, the simplified stress-based approach was used.

3.4.2 Liquefaction Criteria

“The soil liquefaction potential is assessed using a combination of seismic measurements, field circumstances, and soil properties.

In terms of seismic factors, higher the earthquake's magnitude, the proximity of the epicenter, the longer the duration, and the higher the value of peak ground acceleration (PGA), the greater the liquefaction risk.

Liquefaction probability changes with fines percentage (FC), and when fines percentage rises up to 30%, soil liquefaction strength decreases; nevertheless, when fines percentage exceeds 50%, liquefaction probability diminishes and the soil is barely liquefiable. Greater average particle sizes (D50) have less capacity for soil liquefaction because it is difficult for this type of soil to develop excess pore pressure in the larger gaps of these particles, which also gives higher blow count values of standard penetration test (SPT). Several studies on sand samples with varied grain sizes in gap-graded and well-graded distributions demonstrated that gap-graded and well-graded distributions had the least and greatest resistance to liquefaction respectively.

The vertical effective stress value is higher when a soil deposit is placed at a deeper depth in the field. As a result, overcoming vertical forces is extremely difficult for pore water pressure, which minimizes liquefaction risk. In the engineering literature, there appears to be little or no study or verifiable occurrences of seismic liquefaction below 30 meter depth. Florin and Ivanov determined that liquefaction is practically impossible for an overload greater than 15 m, even in very loose sand. Furthermore, a deeper groundwater table (GWT) and thin susceptible layer of soil can reduce the likelihood of soil liquefaction to some extent.

The following parameters, which consists features of soils which can be liquefied, can be used to provide a preliminary evaluation of a soil's seismic liquefaction susceptibility in a major earthquake:

- Mean grain size (D50 = 0.02–1.0 mm)
- Coefficient of uniformity (< 10)
- Plastic index (PI < 10)
- Intensity of an earthquake (> VI)
- Depth of soil deposit (< 15 m)
- Relative density (< 75%)".(Ahmad, Tang et al. 2018)

3.4.3 Geology and Seismic Tectonic Setting

Kamra City lies in Pakistan's Punjab province's hilly north, with latitude 33°45′0″ N and longitude 73°31′0″ E. Kamra is the aviation city of Pakistan. Military firms like Air Weapon Complex (AWC) and more importantly, Pakistan Aeronautical Complex (PAC) are situated there. Kamra is situated towards the northwestern of Pakistan in Karakoram–Himalaya crystalline thrust region which is an active seismic zone.

Kamra is in an active seismic zone, with a recent major earthquake near the Pakistan–Afghanistan boundary in October 2015 (magnitude = 7.5), which generated a sequence of aftershocks with magnitudes larger than 6.5. The Ranja–Khairabad fault line, which stretches about 370 kilometers, is 3 kilometers away from the city. This fault tracks the main boundary thrust line with a northward dip. The fault is still dynamic, according to satellite photography and elevation data, and has distorted many quaternary deposits in the area. According to Pakistan's 2007 Building Code, the research region is located in 2B zone, with a peak ground acceleration (PGA) of 0.24g.

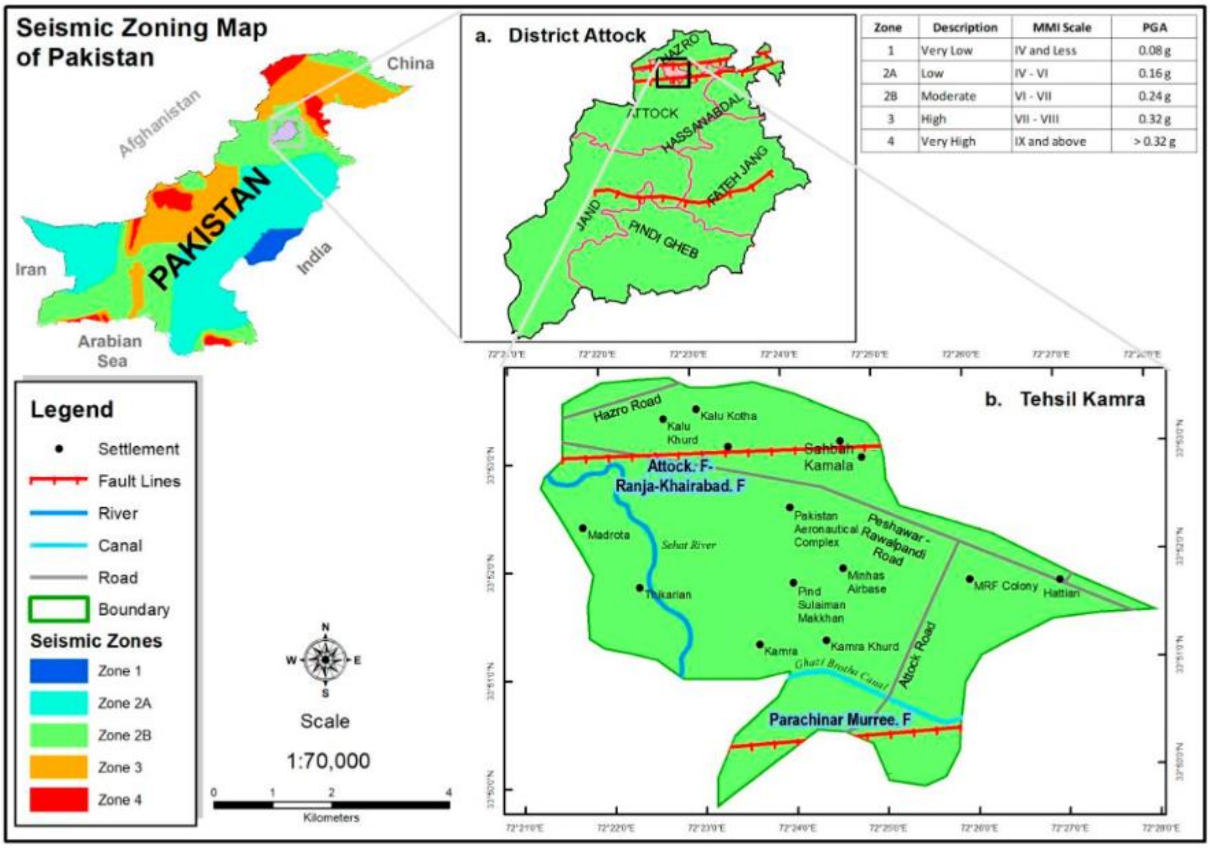

Figure 19.6 Seismic zonal map of study area in Pakistan

Ghazi Brotha canal and river Sehat flow through the research area, while the river Kabul and river Indus flow close to Kamra city, potentially increasing risk of soil liquefaction. Due to the abundance of groundwater in the proximity to water bodies like lakes, rivers, ponds, seas; liquefaction becomes more likely.

3.4.4 Geotechnical site Characterization

“Kamra's typical subsoil profile constitutes of silty gravel which is overlain by silty sand, poorly graded sand, and fill layers. Figure shows the stratigraphy and depth of the groundwater level.

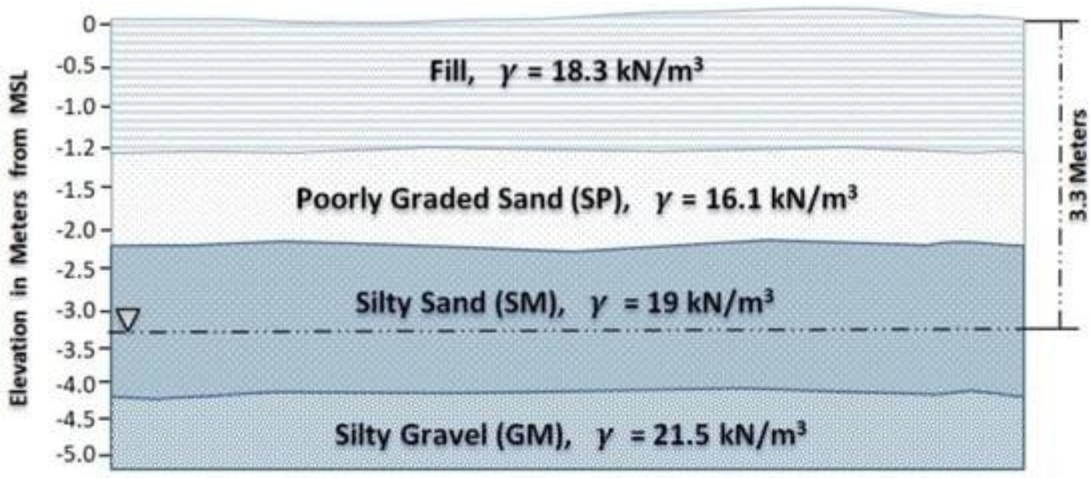

Figure 20.7 Typical subsoil profile of study area, Kamra

The required soil parameters like soil type, measured SPT blow numbers (Nm), fines content percentage, unit weight (γ), and vertical and effective stress were all derived from the SPT borehole findings.

The grain size study followed the American Society for Testing and Materials standard (ASTM D422-63). This method gives the size and category of sand. As per the Unified Soil Classification System, the sands from across all locations were classified as poorly graded sand (SP) and silty sand (SM) on the soil distribution curves (USCS). The USCS standards of a coefficient of curvature (Cc) less than 1 and a coefficient of uniformity (Cu) less than 6 for poorly graded sand classes were met by the poorly graded sand from all sites, which varied from 0.61 to 0.99 and 1.8 to 2.65, respectively. The fraction finer than filter no. 200 (F200) of silty sand throughout all locations were larger than 12 percent and the plastic index (PI) was more than 4. This indicated that the USCS guideline values were compatible and the sand class was silty sands".(Ahmad, Tang et al. 2018)

3.4.5 Evaluation of Liquefaction Potential

Practicing engineers utilize any of the three ways, mentioned below, to assess soil liquefaction capacity of a site: one is energy-based, another is stress-based, and alternative to these is strain-based method. Among practicing engineers, empirical methods and semi empirical techniques that are based on the simplified stress based approach, are more popular. Seed and Idriss pioneered the stress-based approach for determining soil liquefaction capacity in the year 1971 and it was periodically modified afterwards by many scientists.

The stress-based approach devised by Youd et al. was utilized in this research to determine the factor of safety (FS) value of the soil against seismic liquefaction at Kamra, Pakistan. SPT was conducted at ten distinct locations which consists of 50 boreholes throughout this study region to determine the soil parameters and to plot the subsoil profile.

In situ measurements are widely used in common engineering practice for quantitative estimation of soil liquefaction potential of a site. The cyclic stress ratio (CSR) is the earthquake induced stress on soil stratum, while cyclic resistance ratio (CRR) represents the soil liquefaction resistance capacity. The safety factor (FS) against soil liquefaction is computed using these two factors.

This equation gives the FS value against liquefaction susceptibility:

$$FS = \frac{CRR_{7.5}}{CSR} \cdot K_\sigma \cdot K_\alpha \cdot MSF$$

In this equation CRR7.5 is the cyclic resistance ratio for an intensity 7.5 earthquake, Kσ is the overburden correction, Kα is the correction for ground slope, and MSF is the scaling parameter for different earthquake magnitudes. Factor of Safety values were determined at various depths of the study area's subsurface profile.

3.4.6 Results and Discussion

“For the purpose of this study, liquefaction capacity was assessed at 10 different locations (50 bore holes) covering the study area for various earthquake magnitudes between M = 6.5 and 8.0 with a peak ground acceleration (PGA) of 0.24g. Two basic criteria of the factor of safety for soil liquefaction capacity were used for this study: liquefiable soil will have a FS ≤ 1.0 and non-liquefiable soil will be having FS > 1.0. The typical subsoil profile of the study area consists of silty gravels overlain by silty sand, poorly graded sand, and fill layers.

The factor of safety was calculated when the groundwater table rose from 3.3 to 1.2 meters during an earthquake. SPT blow counts for the specified soils were not greater than 11, indicating that the poorly graded sand and silty sand layers were of loose-to-medium density. Table summarizes the results and changes in the SPT values, as well as the factor of safety value against prospect of soil liquefaction.

Table 3.1 Summary of SPT samples and factors of safety (FS) against soil liquefaction potential results.

Depth (m)	N_m	$(N_1)_{60cs}$	r_d	*CSR*	$CRR_{M=7.5}$	K_σ	K_α	*FS*			
								M = 6.5	*M* = 7.0	*M* = 7.5	*M* = 8.0
1.2	6	9.7	0.993	0.155	0.111	1	1	1.03	0.85	0.71	0.61
1.6	7	10.9	0.990	0.179	0.121	1	1	0.97	0.80	0.67	0.57
2.0	7	11.2	0.987	0.199	0.123	1	1	0.90	0.74	0.62	0.53
2.4	8	16.2	0.984	0.212	0.172	1	1	1.17	0.97	0.81	0.69
2.8	8	15.3	0.981	0.223	0.163	1	1	1.05	0.87	0.73	0.62
3.2	8	16.5	0.978	0.231	0.175	1	1	1.09	0.90	0.76	0.64
3.6	10	19.2	0.975	0.238	0.206	1	1	1.25	1.03	0.87	0.73
4.0	11	21.7	0.973	0.244	0.238	1	1	1.41	1.17	0.98	0.83

As the factor of safety versus soil liquefaction susceptibility at an earthquake of magnitude M = 6.5 was less than 1 for only two subsoil regions, the poorly graded sand layers at depths of 1.6 and 2.0 m, these two are the zones of potential liquefaction. Likewise, after an earthquake of intensity M = 7.0, the silty sand layer could not liquefy at depths of 3.6 and 4.0 m because the soil stratum possessed FS > 1. (See Table). For higher earthquakes magnitude of M = 7.5 and 8.0, the poorly graded sandy and silty sand soil strata were totally in the zone of liquefaction probability. According to the results of the soil liquefaction potential study, the amplitude of soil liquefaction varies with different seismic earthquake orders of magnitude. The findings demonstrated that when an earthquake's intensity exceeds 7, the likelihood of soil liquefaction rises, and that the greater the seismic magnitude, the greater the liquefaction potential and more damage it produces. During an earthquake with a magnitude of 7.5 to 8.0 at Kamra, the middle layers (SP and SM in the subsoil profile) are highly vulnerable to liquefaction.

Figure shows the same tabulated data in form of a graph. It depicts the factor of safety versus liquefaction and the corresponding SPT blow counts (N1)60cs corrected for clean-sand which is correlated to earthquake orders of magnitude M = 6.5, 7.0, 7.5, and 8.0".(Ahmad, Tang et al. 2018)

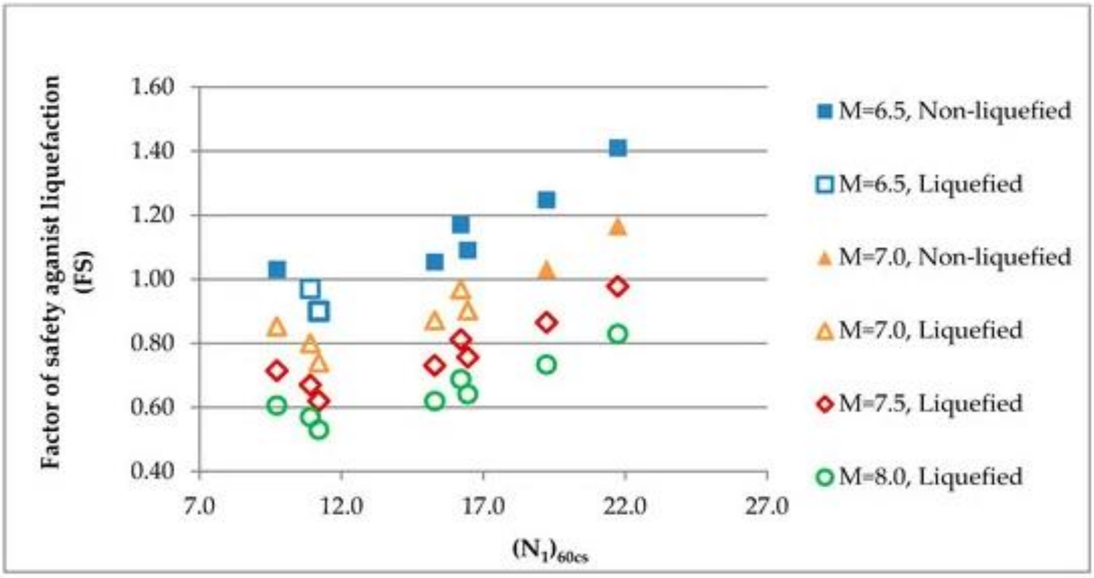

Figure 21.8 Correlation of FoS & Corrected SPT blow counts for Kamra Soil

Chapter 04

4 METHODOLOGY

4.1 General

If appropriate geotechnical research and designs are not performed, structures built on liquefied soils might be fatally damaged. When the site is situated in a seismically active zone and whose soil stratigraphy is prone to liquefaction, a complete geotechnical examination should be done to address liquefaction hazard. Examination in the research area are comprised of two steps which include laboratory test for site delineation and initial screening of liquefiable soils and afterwards SPT-based field inspection for soil liquefaction assessment.

Geotechnical site characterization including subsoil profile, grain size analysis, Fines Content and Atterberg limits and finally soil type is discussed in this section along with the degree of importance of geology and seismic tectonic setting of the site.

There are many approaches for liquefaction evaluation which are developed by different geologists in different times. Being simple and largely adopted by contemporary scientists, Stressed based approach is utilized in this study and it will be discussed in detail in this section.

Further investigation for the evaluation of soil liquefaction and the Codes/Standards utilized in this study are also mentioned.

4.2 Geotechnical Site Characterization

4.2.1 Subsoil Profile

Exploring subsoil stratification is necessary to ascertain the different soil layers, their depths and other parametrs. Subsoil profile is plotted as per the borehole data and the depth of groundwater table is also noted.

The SPT borehole data consists of soil type, number of blow counts (Nm), percentage of fines, unit weight (γ), and effective stresses.

4.2.2 Grain Size Analysis

American Standard (ASTM D422-63) is utilized for the grain size examination. This method gives the idea of range of soil particles as per the Unified Soil Classification System (USCS).
Grain size examination includes:

- Soil gradation curves
- Coefficient of Curvature (C_u)
- Coefficient of Uniformity (C_c)
- Mean Grain Size (D_{50}).

4.2.3 Fines Content (FC) and Atterberg Limits

Soil liquefaction resistance may vary depending on fines percentage in a soil sample. When the fines percentage increases up to 30%, soil's capacity to resists liquefaction increases; however, when fines percentage is above 50%, soil liquefaction capacity diminishes and that soil is barely liquefied. Furthermore, when fines percentage increases, soil type or group also changes.
Soil fines percentage is calculated, and the Atterberg limit test is used to find plasticity index (PI) and assess the category of the soil whether it is the plastic or non-plastic in line with ASTM D4318-10 standard. Because the soil liquefaction probability is directly connected to the Atterberg limits of the soil, any sample with significant content of plastic particles must be judge using the Atterberg limits.

4.3 Geology and Seismic Tectonic Setting

"The site's location in relation to seismic zones and the geological setting of the Earth's crust is critical for the site's soil liquefaction potential as well as the history of recent seismic activity. The amplitude of ground accelerations and the distance from surrounding fault lines are directly related to the magnitude of the earthquake. The National Engineering Services of Pakistan (NESPAK) issued a seismic map of Pakistan which was published in 2010 based on the Building Code of Pakistan (BCP, 2007), which depicts five zones based on predicted ground movement. The study areas are both located in zone 2B with the ground acceleration of 0.24g. With the help of the Uniform Building Code (UBC, 97), the BCP is accepted and developed as a reference code.

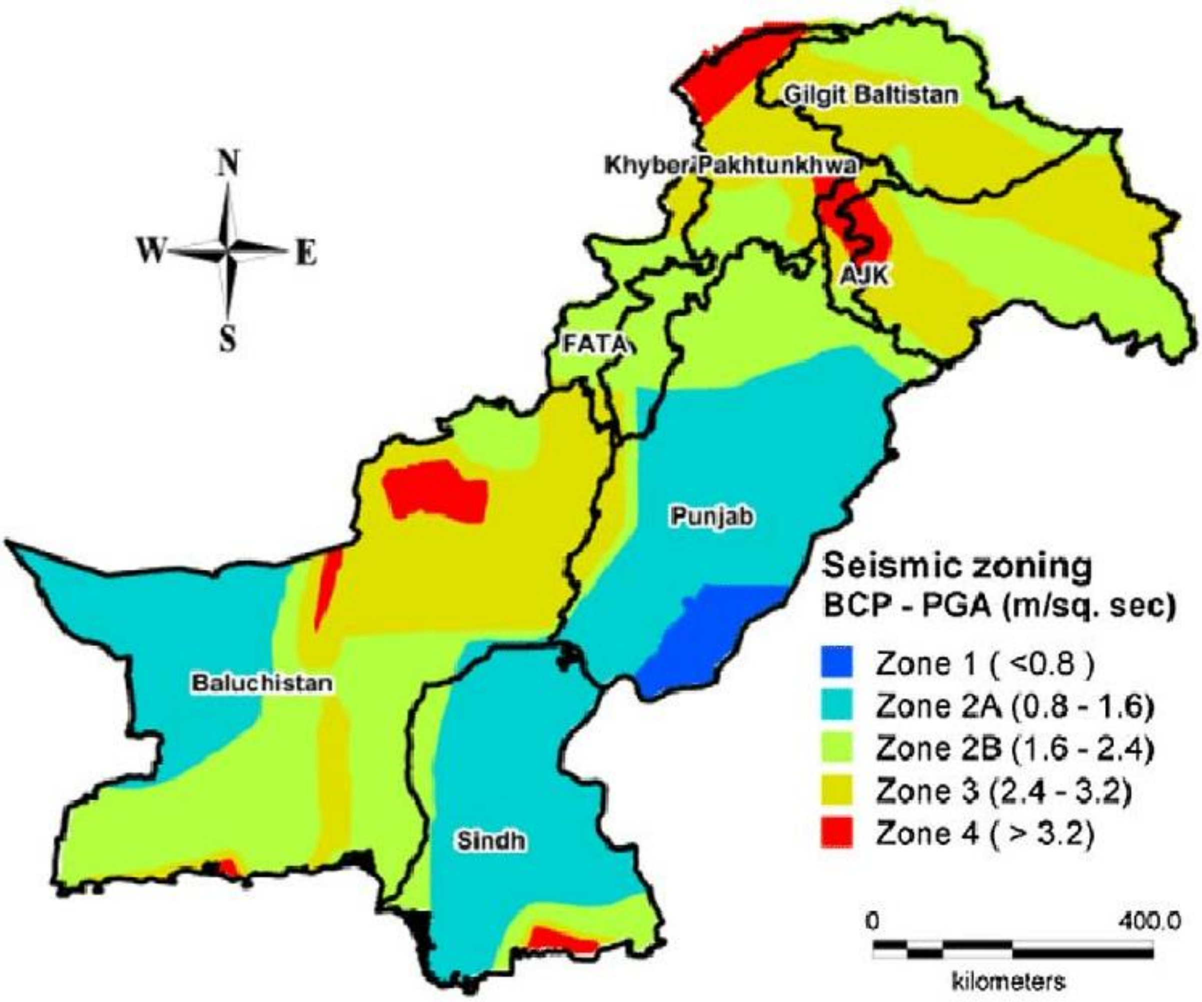

Figure 22.1 Seismic zoning map of Pakistan (NESPAK, 2010)

Length of the fault is the main criteria for the magnitude of the earthquake. Earthquake magnitude is evaluated from the fault length with the help of following equation which is given by Tocher as follows:

$$LogL = 1.02M - 5.77$$

Here L refers to fault line length and M denotes the magnitude of the earthquake. So earthquake of a magnitude $M \leq 8.0$ can occur at the area of study. In the study, various magnitudes of earth quake between 6.5 and 8.0 were used in the evaluation.

Rivers or other nearby water bodies are other geological elements that impact the site's soil liquefaction potential. Because of the abundant amount of groundwater in these areas, soil liquefaction is common, although it's difficult to quantify”.(Ahmad, Tang et al. 2018)

4.4 Liquefaction Assessment Approaches

Practicing engineers use three methods to evaluate soil liquefaction potential: (1) based on energy, (2) based on stress, and (3) based on strain. Among practicing engineers, empirical methods and semi empirical approaches based on the stress methods are more common and are the standard.

4.4.1 Stressed Based Approach

In 1971, Seed and Idriss developed an approach based on stress which is "simplified technique" for finding out soil liquefaction. Shibata Tokimatsu and Yoshimi, Seed et al. (1985) and Idriss and Boulanger (2004) have all modified and updated the simplified procedure. The simplified technique, as used by some studies, analyzes the ratio of the CSR of the soil with CRR to find out the liquefaction potential .The safety (FS) factor against liquefaction was found out in both Risalpur Cantonment and DHA Peshawar Nisatta, Charsadda, using the stress based approach devised by Youd et al. The safety factor versus soil liquefaction capacity is given below:

$$FS = \frac{CRR_{7.5}}{CSR} \cdot K_\sigma \cdot K_\alpha \cdot MSF$$

“CRR is the cyclic resistance ratio for an earthquake with the magnitude of 7.5, $K\sigma$ refers to the overburden correction factor , $K\alpha$ denotes to the the ground sloping correction, and MSF is the factor for magnitude scaling. Safety factors were found out at varying depths”.(Ahmad, Tang et al. 2018)

4.1.1.1 Calculation of Cyclic Stress Ratio (CSR)

In order to compute CSR, equation was referred by Seed and Idriss which is given below:

$$CSR = 0.65 \frac{a_{\max}}{g} \frac{\sigma_v}{\sigma'_v} r_d$$

Here a_{max} is maximum horizontal acceleration occurring at ground level, g refers to acceleration of gravity, σ_v total stress and σ'_v denotes effective vertical overload stresses, and r_d represents a stress reduction component for depth and it depicts elasticity of the soil , for rigid body r_d = 1). The stress reduction factor is calculated using the following relationship.

$$r_d = 1 - 0.00765z \text{ for z } \leq \text{ 9.15 m}$$

$$r_d = 1.174 - 0.0267z \text{ for 9.15 m } < \text{ z } \leq \text{ 23 m}$$

Z denotes to depth under surface.

4.4.1.1 Calculation of Cyclic Stress Ratio (CRR)

The CRR required to be corrected of the fines percentage and value of equal clean sand, (N1)60cs, according to the approach which is given by given by Youd .Using liquefaction case history data following equation is achieved:

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{[10 \cdot (N_1)_{60cs} + 45]^2} - \frac{1}{200}$$

This relation is accurate for $(N1)_{60cs} < 30$. In case of $(N1)_{60cs} \geq 30$, granular soils are very much compacted and are categorized as non-liquefiable.

4.4.1.2 SPT Blow Counts Correction

The SPT blow values were standardized for the effective overload stress at the depth of the test and rectified to a standard value.

Blow counts values were calculated from the below relation.

$$N_{1(60)} = N_m C_N C_E C_B C_R C_S$$

Here, N_m is the penetration resistance, and C_N, C_E, C_B, C_R, and C_S are factors of effective overload stress correction, ratio of hammer energy, dia of borehole, length of rod, and with or without liners sampling, accordingly.

4.4.1.1 Fines Content Correction

Correcting the Fines percentage was firstly initiated by Seed and Idriss, with the assistance of R. B. Seed, the following equation incorporates this correction criteria:

$$N_{1(60)cs} = \alpha + \beta (N_1)_{60}$$

Here, α and β can be calculated from the following calculations:

$$\alpha = 0 \text{ for FC} \leq 5\%,$$

$$\alpha = \exp\left[1.76 - \left(\frac{190}{FC^2}\right)\right] \text{ for } 5\% < \text{FC} < 35\%,$$

$$\alpha = 5.0 \text{ for FC} \geq 35\%,$$

$$\beta = 1.0 \text{ for FC} \leq 5\%,$$

$$\beta = \left[0.99 + \left(\frac{FC^{1.5}}{1,000}\right)\right] \text{ for } 5\% < \text{FC} < 35\%,$$

$$\text{and } \beta = 1.2 \text{ for FC} \geq 35\%.$$

4.5 Investigation

Lab tests for targeted area depiction and initial testing of liquefiable soils were conducted in the research region, based on SPT field computation of soil for liquefaction evaluation. SPT was carried at 4 distinct boreholes at the Risalpur Cantonment site and 50 boreholes at the DHA Nisatta, Charsadda site to evaluate N values, subsoil characteristics, constitute a subsoil profile, and recover soil samples for lab testing.

The research area's tectonic and seismic zoning maps were adjusted using data from the National Engineering Services of Pakistan (2010) and geological survey maps of Pakistan (2006), respectively. If the groundwater table raises up from 7 m to 2 m depth during an earthquake, the soil liquefaction capacity in the research area, which was having PGA of 0.24g and an earthquake ranges from 6.5 and 8.0, was evaluated. The relation between safety factor and comparable clean sand corrected SPT blows is also constructed.

4.6 Designs Codes and Regulations

“In this study, codes and the guidelines utilized for the analysis of soil liquefaction process are as follows:

- ASTM D4318-10 The American Society for Testing and Materials standard
- ASTM D422-63 The American Society for Testing and Materials standard
- USCS Unified Soil Classification System
- BCP, 2007 Building Code of Pakistan
- UBC-97 Uniform Building Code for seismic resistant design”.(Ahmad, Tang et al. 2018)

4.7 Project Sequence

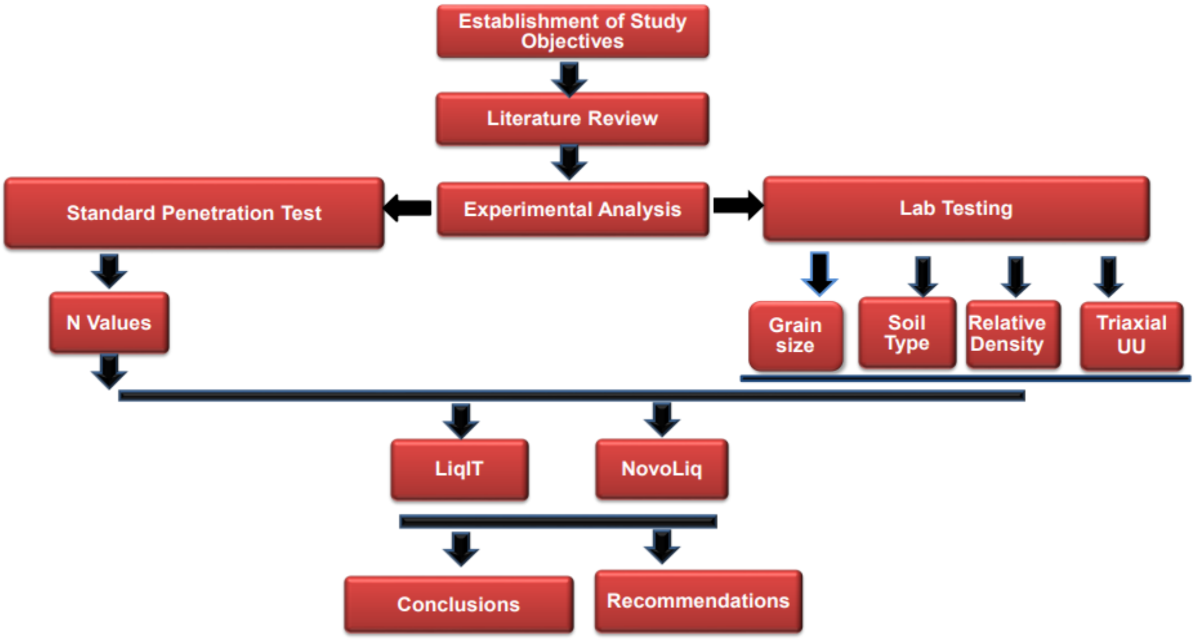

Figure 23.2 Flow Chart of Research Methodology

Chapter 05

5 ANALYSIS AND RESULTS

5.1 Risalpur Cantonment Site

5.1.1 Analysis

5.1.1.1 Geology and Seismic Tectonic Setting

Risalpur is situated towards northwest of Pakistan in the Karakoram–Himalaya crystalline thrust region, according to Pakistan's tectonic map. It is a seismic active area with a high risk of earthquakes.

The fault line of Ranja–Khairabad, which is having a stretch of 370 km, is 20 kilometers away from Risalpur Cantonment, as illustrated in Figure. As a result, a seismic activity measuring of Magnitude 8.0 could strike research site according to the equation provided below. In this study, liquefaction potential was assessed at various different earthquake magnitudes, from M = 6.5 to 8.0.

$$LogL = 1.02M - 5.77$$

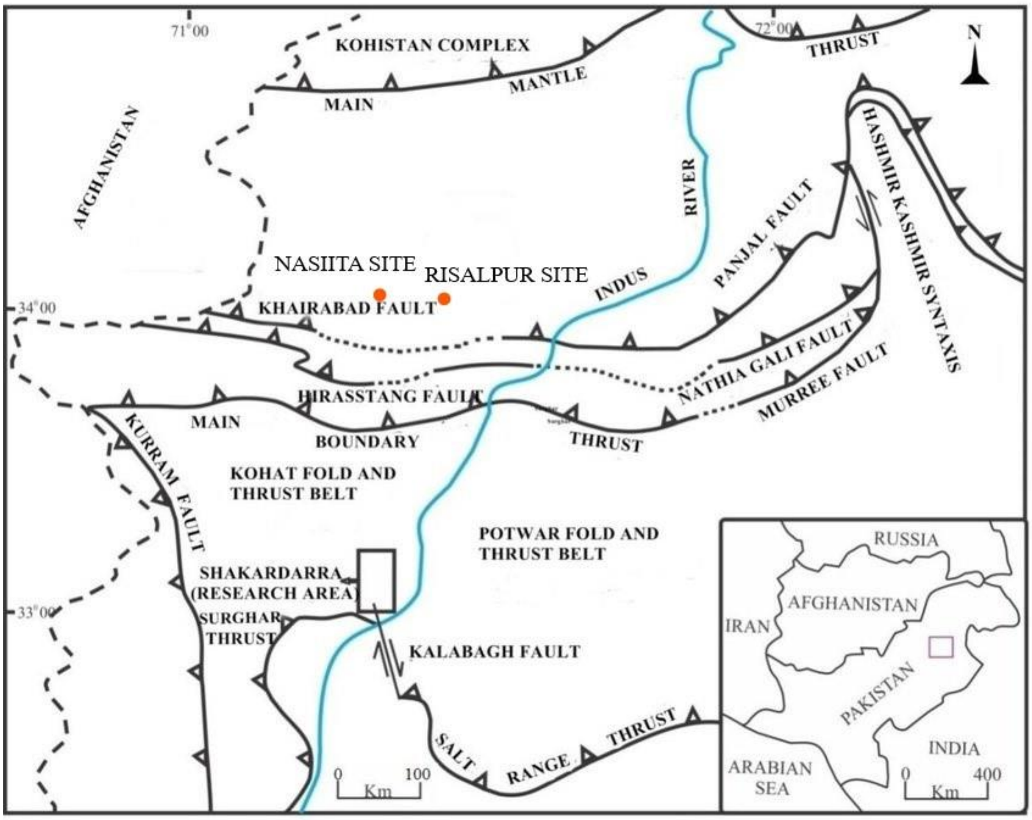

Figure 24.1 Location of Risalpur site w.r.t the fault line

This specific fault followed the trend line of main boundary thrust with a northward dip. This fissure is still active, according to satellite photography and topographic maps, and has distorted many quaternary sediments in the area. The water level in Risalpur, on the other hand, is really shallow and was not discovered during borehole operations. This incidence could be attributed to the ever-increasing number of tube wells extracting the groundwater.

Soil liquefaction does not occur when the groundwater table is too deep but we are considering a hypothetical situation in which there is a sudden rise in Risalpur's groundwater table in event of Flash floods combined with heavy rains as seen in 2010 Flooding.

5.1.1.2 Geotechnical Site Characterization

The typical sub ground profile of Risalpur comprised of predominantly silty clay with a thin layer of poorly graded sand at the depth of 14 to 17 feet.

A typical bore log of Risalpur Cantonment is shown below:

Table 5.1 Typical bore log of Risalpur site

Material Description	Classification	Sample No.	Depth (ft)	SPT Blows			N Value	Remarks
				1st 6"	2nd 6"	3rd 6"		
L.L=25.3% P.L=21.8% PI=3.5%	ML		1					
			2					
			3					
		DS/SS	4	3	3	3	6	
			6					
			8	3	4	6	10	
			10					
		DS/SS	12	4	5	6	11	
SAND =94.0 % FINES =6.0%	SP		14					
			15					
			16	6	7	8	15	
L.L=24.0% P.L=18.8% PI=5.2%	CL-ML		17					
			18	7	8	10	18	
			20					
		DS/SS	22	9	12	13	25	
			25					
			26					
			28	10	12	14	26	
			30					

According to the Soil classification, the strata were classified to be mainly cohesive soil i.e. silty clay (CL-ML) with a plasticity index (PI) above 5%, on soil grading curves (USCS). There a 3 feet thick middle layer at 16 feet comprising of poorly graded sand (SP) having curvature coefficient (Cc) and uniformity coefficient (Cu) varied from 0.61 to 0.99 and 1.8 to 2.65, accordingly.

The proportion finer than filter no. 200 (F200) and the plastic index (PI) of low plasticity silt at all sites was greater than 70% and the plastic index (PI) was 3.5. Silty Clay has a PI of 5.2. The intended Liquefaction requirements were not met due to the excessive fines content.

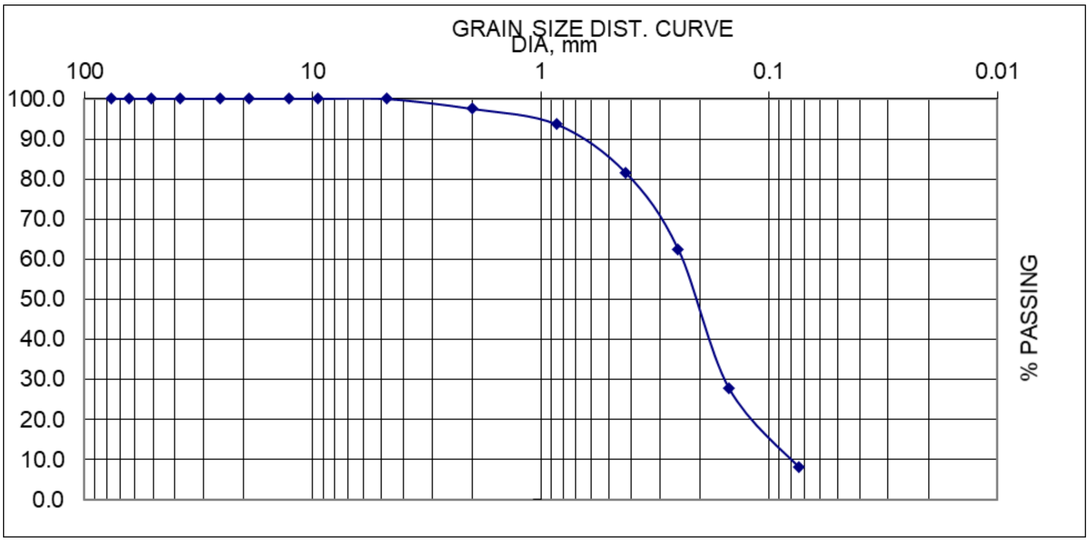

Figure 25.2 Gradation Curve of Risalpur soil

5.1.2 Results

As mentioned in the previous section, Risalpur soil does not fulfill the liquefaction criteria which depicted by the results achieved through the software results. The safety factor, FS is greater than the threshold value of unity which is the testament that the soil experiences no liquefaction even at an Earthquake of magnitude 8.0.

Table 5.2 Factor of Safety Vs depth of Risalpur site

No	Depth	Gamma	% FC	u	Sigma	Eff. sigma	Nspt	N1(60)	N1(60)cs	CSR	CRR	F.S.
1	4.00	120.95	80.00	0.00	0.24	0.24	6.00	4.82	10.78	0.11	0.12	1.10
2	8.00	120.95	80.00	0.00	0.48	0.48	10.00	7.42	13.90	0.11	0.15	1.43
3	12.00	120.95	80.00	0.00	0.73	0.73	11.00	7.17	13.61	0.11	0.15	1.41
4	16.00	102.49	6.00	0.00	0.93	0.93	15.00	9.14	9.21	0.10	0.10	0.97
5	18.00	120.95	80.00	0.00	1.05	1.05	18.00	10.50	17.60	0.10	0.19	1.84
6	22.00	120.95	80.00	0.00	1.29	1.29	25.00	13.46	21.16	0.11	0.23	2.16
7	28.00	120.95	80.00	0.00	1.66	1.66	26.00	12.63	20.15	0.11	0.22	1.98

The only exception is at the depth of 14 to 17 feet where the value of FS drop below the unity to 0.97. This may be because this thin layer constitutes of poorly graded sand and the $N1(60)_{cs}$ value also drops at this point.

However, as this layer is at a greater depth of 16 feet, having a thickness of only 3 feet and is bounded by non-liquefiable soil both above and below it, it is unlikely that this layer will cause any liquefaction induced problems on the surface. (Ishihara and Koga, 1981).

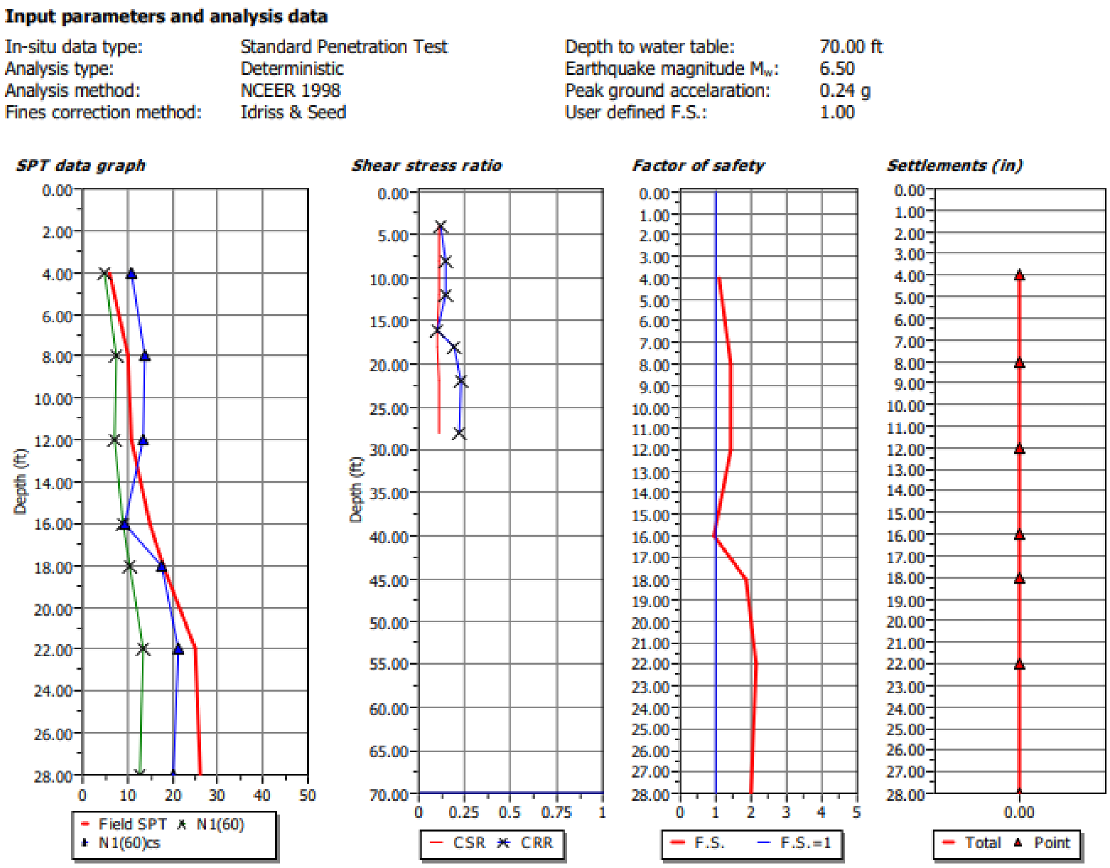

Figure 26.3 Graphical representation of FoS & SPT N values of Risalpur at M=6.5

The factor of safety curve is also developed and all the points are plotted on the right of FS=1 line which is no liquefaction region. The only stand out point is point 4 depicting the SP layer from 14 to 17 feet which plots to the left of FS=1 line which is liquefaction region.

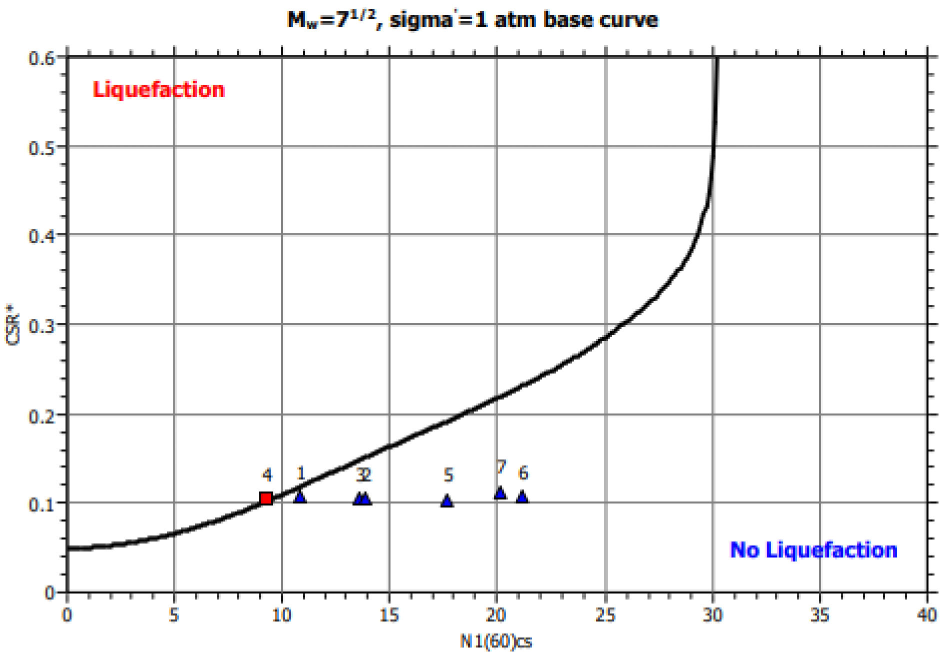

Figure 27.4 FoS Curve of Risalpur at M=6.5

Factor of Safety against different magnitudes of earthquakes at Risalpur site is analyzed. Risalpur soil is Non-Liquefiable even at M= 8.0 Earthquake except the 3 feet thick middle layer which liquefies at M=7.5 earthquake. SPT & FS values are summarized in the table below:

Table 5.3 Factor of Safety against various magnitudes at different depths

Depth Ft	Nm	(N1)60cs	CSR	CRR	FS			
					M=6.5	M=7.0	M=7.5	M=8.0
4	6	10.78	0.178	0.18	1.77	1.44	1.19	0.99
8	10	13.9	0.224	0.27	2.17	1.76	1.45	1.21
12	11	13.61	0.243	0.32	2.41	1.95	1.59	1.32
16	15	9.21	0.255	0.21	1.49	1.2	0.97	0.81
18	18	17.6	0.258	0.8	3	3	3	3
22	25	21.16	0.259	0.8	3	3	3	3
28	26	20.15	0.256	0.8	3	3	3	3

5.1.2.1 Relative Density Check

Many empirical relations between soil index properties and Liquefaction potential are proposed by different scientists for easy and direct liquefaction assessment. One of the most important and common relationship is given by (Seed and Idriss, 1971) which compares the Relative Density, D_r, of soil with the liquefaction potential of that soil.

Table 5.4 Relationship between relative density and liquefaction potential

Approximate relationship between earthquake magnitude, relative density, and liquefaction potential for water table 1.5 m below ground surface*

Earthquake acceleration	High liquefaction probability	Potential for liquefaction depends on soil type and earthquake acceleration	Low liquefaction probability
0.10*g*	$D_r < 33\%$	$33 < D_r \leq 54$	$D_r > 54\%$
0.15*g*	< 48	$48 < D_r \leq 73$	> 73
0.20*g*	< 60	$60 < D_r \leq 85$	> 85
0.25*g*	< 70	$70 < D_r \leq 92$	> 92

*From Seed and Idriss (1971).

Relative density for Risalpur soil is 75% and earthquake acceleration is 0.24g. Which categorizes it in second category and “Liquefaction Potential depends on soil type”. The soil type has already been assessed as non-liquefiable by various criteria in previous sections. So the results of our previous approach are supplemented by this technique.

5.2 DHA Peshawar Site

5.2.1 Analysis

5.2.1.1 Geology and Seismic Tectonic Setting

Charsadda is positioned at Karakoram–Himalaya metamorphic thrust zone, near the north and west of Pakistan, according to Pakistan's tectonic map. It is a tectonically active region with a high risk of earthquakes.

“DHA Peshawar, Nisatta, as stated in Figure, is situated round about 25 kilometers from the fault line of Ranja–Khairabad which stretches to about the 370 kilometers. As a result, a seismic activity measuring of Magnitude 8.0 could strike research site according to the equation provided below. In this study, liquefaction potential was assessed at various different earthquake magnitudes, from M = 6.5 to 8.0.

$$LogL = 1.02M - 5.77$$

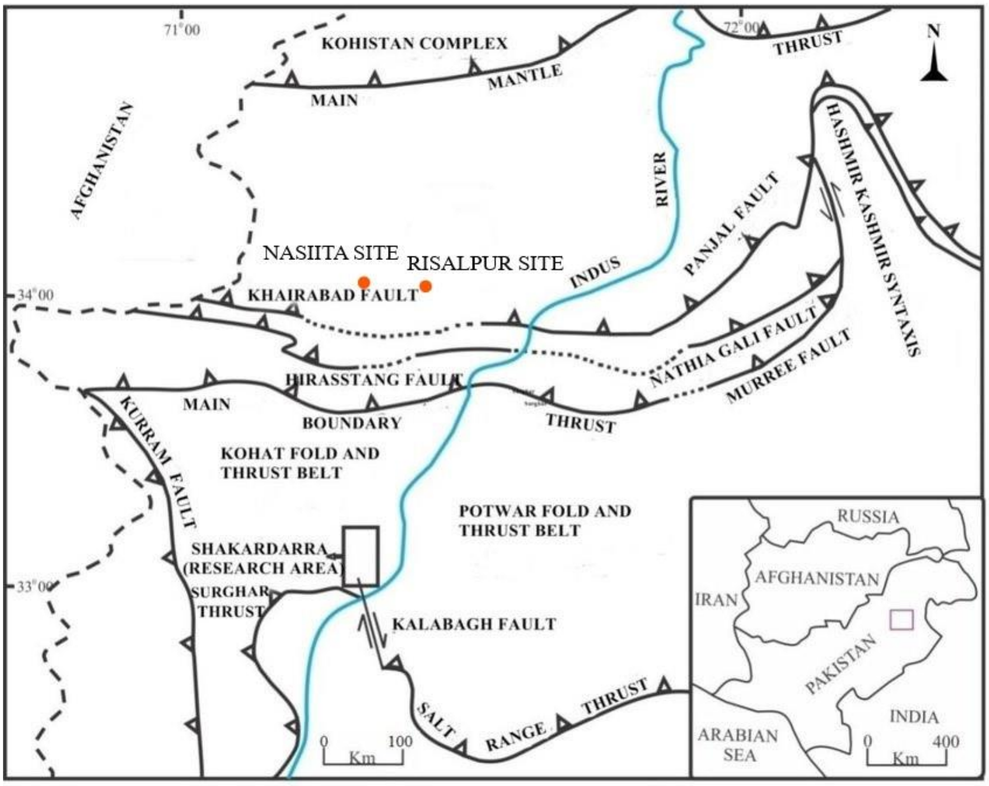

Figure 28.5 Location of DHA site w.r.t the fault line

This fault continues along the trend line of the main border thrust line with a northward dip. The fault is still active, according to satellite photography and digital elevation models, and has distorted many quaternary strata in the area”(Seed and Idriss 1971).

The study area is situated just at the bank of river Kabul and Swat River falls into Kabul just upstream of the site. The groundwater table is very shallow, only 7 feet deep which increase the probability of soil liquefaction. During the Earthquake, groundwater table may be lifted up to 3 feet below the surface.

5.2.1.2 Geotechnical Site Characterization

The typical subsoil profile of DHA site consists of predominantly poorly graded sand overlain by a thin layer of silty sand which is 7 feet deep.

A typical bore log of DHA site at Nisatta, Charsadda is shown below:

Table 5.5 Typical bore log of DHA site

Material Description	Classification	Sample No.	Depth (ft)	SPT Blows 1st 6”	2nd 6”	3rd 6”	N Value	Remarks
			1					
L.L=18.8% PL=16.2%			2					
		DS/SS	4	1	1.5	1.5	3	
Gravel=0.0% Sand= 90% Fines= 14.0%			7					
	SM	DS/SS	8	1	2	2	4	GWT
			9					
			10					
		DS/SS	13	2	2	3	5	
			15					
		DS/SS	18	2	3	4	7	
			20					
			24					
		DS/SS	25	3	4	4	8	
			27					
			28					
		DS /SS	30	3	4	4	8	
			32					
		DS/SS	35	4	5	6	11	
			38					
		DS/SS	40	4	5	6	11	
	Sand		42					
		DS/SS	45	4	6	6	12	
			48					
		DS/SS	50	4	6	6	12	
			52					
			55					
Gravel=0.0% Sand= 92.0% Fines= 8.0%		DS/SS	60	5	7	8	15	
			62					
			65					
		DS/SS	70	5	7	8	15	
			72					
			75					
			78					
			80					

“According to the Unified Soil Classification, sands samples from all stations were classified to be low plasticity SP and SM on soil distribution lines.

Coefficient of curvature (Cc) smaller than unity and a coefficient of consistency (Cu) smaller than 6 for coarse grained sand classes/group were met by the coarse grained sand from all stations, which varied from 0.61 to 0.99 and 1.8 to 2.65, correspondingly. Figure depicts typical illustrative gradation curves of DHA soil. The fraction finer than filter no. 200 (F200) and the plastic index (PI) of silty sand at all stations were larger than 12 percent and 2.6, respectively, indicating that unified soil classification system guide data were complementing and sand class/group was silty sand”.(Seed and Idriss 1971)

The Atterberg’s Limits and the Fines content satisfied the proposed Liquefaction criteria.

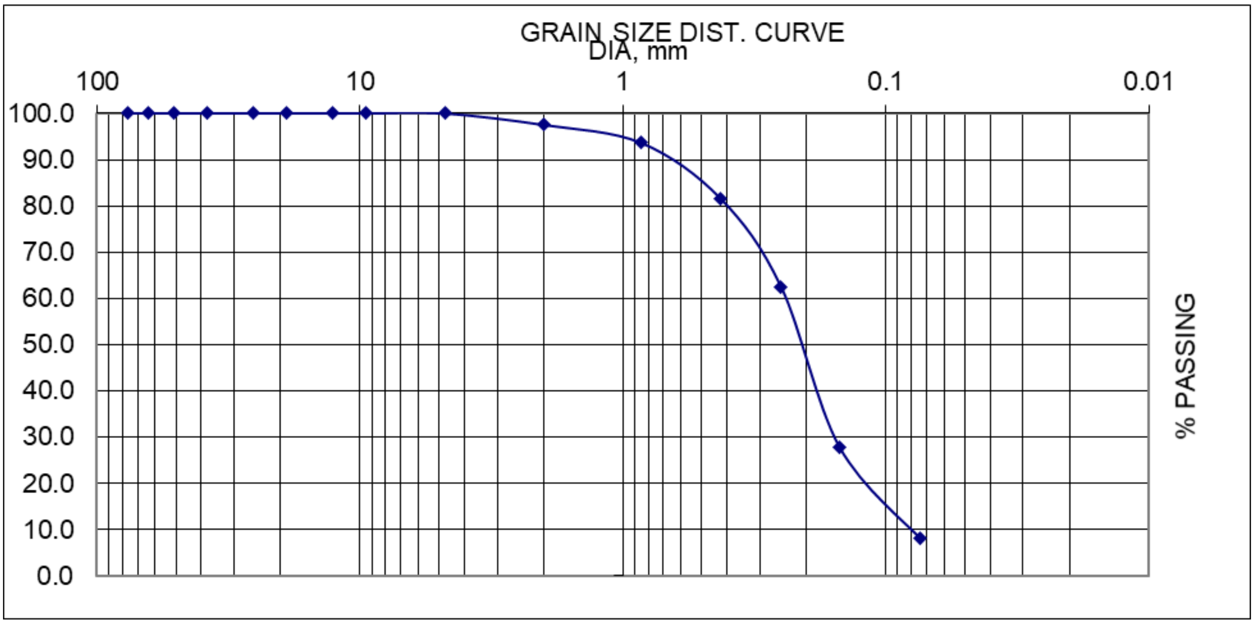

Figure 29.6 Gradation Curve of DHA soil

5.2.2 Results

As mentioned in the previous section, soil of DHA site fulfill all the liquefaction criteria. The SPT N values are not greater than 15 even at the depth of 80 ft which shows that that SP deposits are very loose. The loose state of the sand is also depicted by the values of relative density which is around 50%.

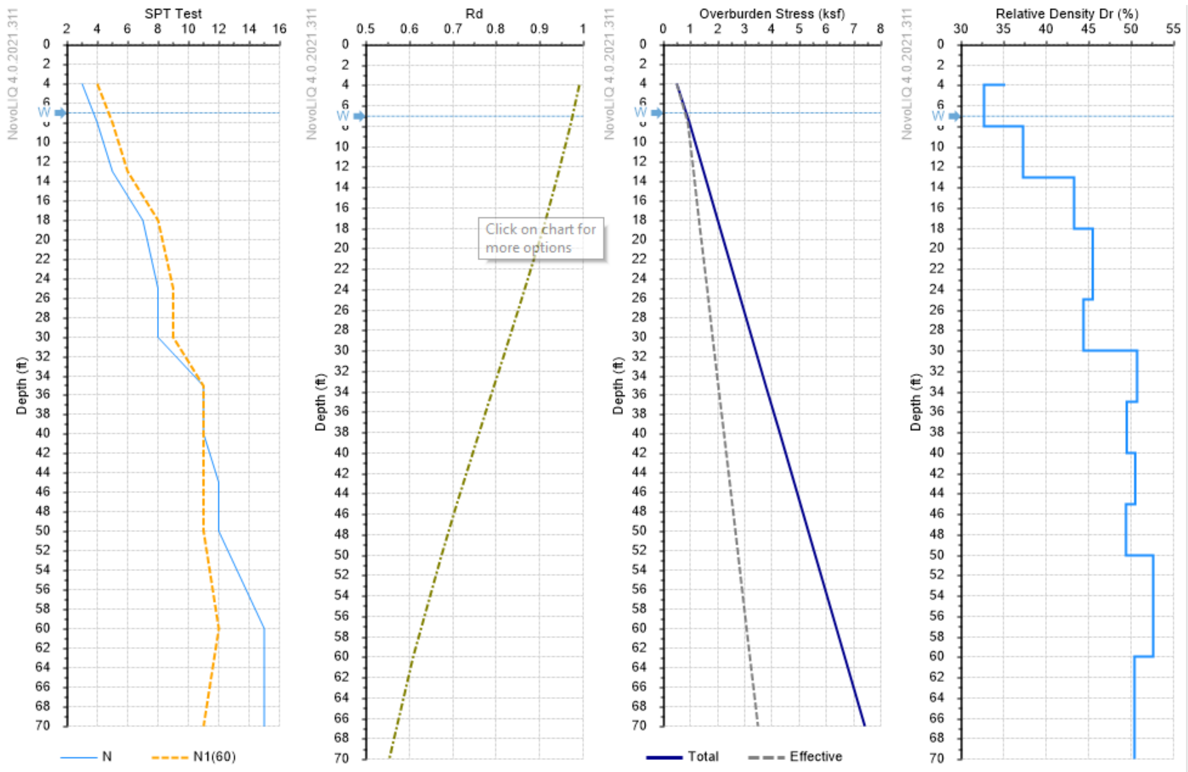

Figure 30.7 Graph of Dr & SPT N values of DHA soil

The liquefaction susceptibility of DHA site is also depicted by the results achieved through the software analysis. The factor of safety, FS is significantly lower than the threshold value of unity which is the testament that the soil will experience liquefaction at an Earthquake of magnitude 6.5. Top 30 feet of soil is liquefied at M = 6.5 and the depth of liquefaction increases as the earthquake magnitude increases with all the soil layer till 70 feet depth being liquefied at M = 7.5 and above.

Table 5.6 Factor of Safety Vs depth of DHA site

Depth (ft)	Rd	Rd_I&B	Overburden Stress (ksf)		Fines Content (%)	SPT Test				Relative Density Dr (%)	Simplified CSR	CSR_I&B	CRR7.5	CRR7.5 (ave)	Safety Factor	Safety Factor	Probability of Liquefaction PL(%)	
			Total	Effective		N	Co	Cn	N1(60)				Boulanger & Idriss		Boulanger &		Youd & Noble	Cetin et al. 2004
4	0.991	0.991	0.49	0.49	12	3	0.79	1.7	4	35.2	0.155	0.155	-	-	-	-	-	-
8	0.97	0.97	0.96	0.9	8	4	0.79	1.45	5	32.7	0.162	0.162	0.09	0.09	0.98	0.98	5.6	95.4
13	0.94	0.94	1.48	1.11	8	5	0.9	1.33	6	37.3	0.196	0.196	0.1	0.1	0.88	0.88	4.9	99.6
18	0.908	0.908	2	1.31	8	7	0.95	1.24	8	43.3	0.216	0.216	0.11	0.11	0.91	0.91	4	99.2
25	0.858	0.858	2.73	1.61	8	8	1.01	1.13	9	45.5	0.227	0.227	0.12	0.12	0.91	0.91	3.6	99.6
30	0.821	0.821	3.25	1.81	8	8	1.02	1.07	9	44.4	0.229	0.229	0.11	0.11	0.87	0.87	3.7	99.9
35	0.783	0.783	3.77	2.02	8	11	1.03	1.01	11	50.7	0.228	0.228	0.13	0.13	1.02	1.02	2.8	97.1
40	0.746	0.746	4.29	2.23	8	11	1.03	0.95	11	49.5	0.224	0.224	0.13	0.13	1	1	2.9	98.8
45	0.709	0.709	4.81	2.44	8	12	1.04	0.91	11	50.5	0.218	0.218	0.13	0.13	1.05	1.05	2.8	97.8
50	0.674	0.674	5.33	2.65	8	12	1.04	0.86	11	49.4	0.212	0.212	0.13	0.13	1.05	1.05	2.9	98.8
60	0.609	0.609	6.37	3.06	8	15	1.05	0.78	12	52.6	0.198	0.198	0.14	0.14	1.21	1.21	2.4	88.9
70	0.554	0.554	7.41	3.48	8	15	1.05	0.71	11	50.4	0.184	0.184	0.13	0.13	1.22	1.22	2.6	94

The FS value increases with the increase in depth and crosses into safe territory after the depth of 45 ft which is depicted in the graph. The overburden stresses increase with depth which reduces the liquefaction potential by negating the pore water pressure which is the reason that the soil is not liquefied beyond the depth of 45 ft.

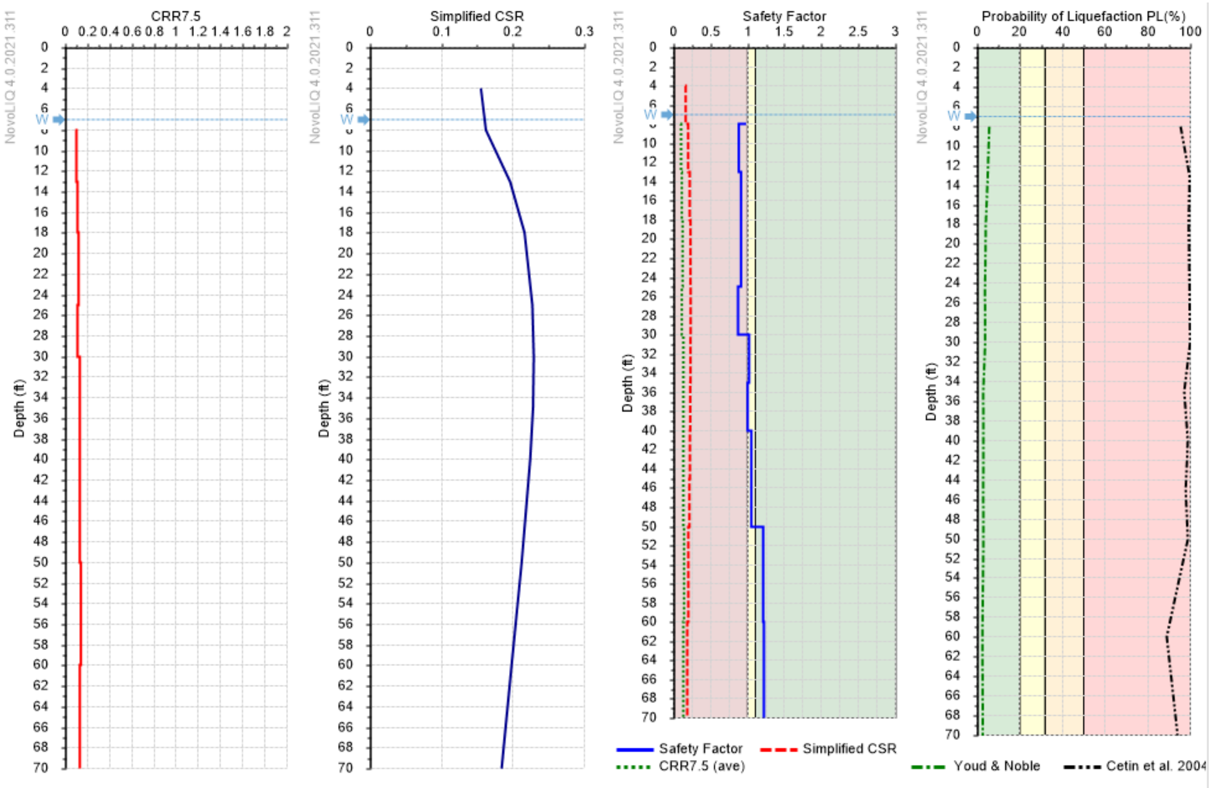

Figure 31.8 Graph of FoS & SPT N values of Risalpur soil at M=6.5

The below graphs show the effects of a liquefaction event at a Magnitude 6.5 earthquake at DHA site. A significant amount of vertical settlement (20 inches at the top) in the soil is depicted here along with even greater amount (80 inches for topmost layer) of horizontal displacement towards the river bank.

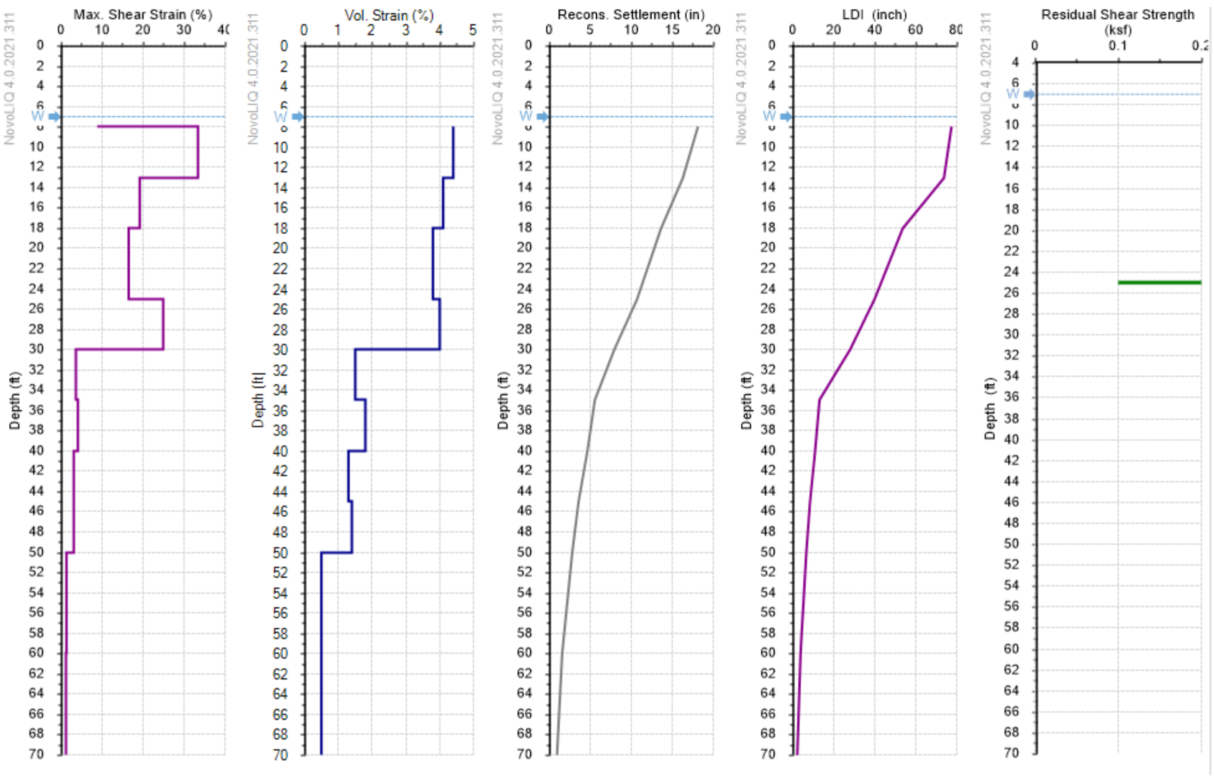

Figure 32.9 Graph of Settlement & Lateral Displacement of DHA soil at M=6.5

The factor of safety curve is also developed and all the points up to 45 feet deep are plotted on the left of FS=1 line which is the liquefaction region depicting severe liquefaction in the soil.

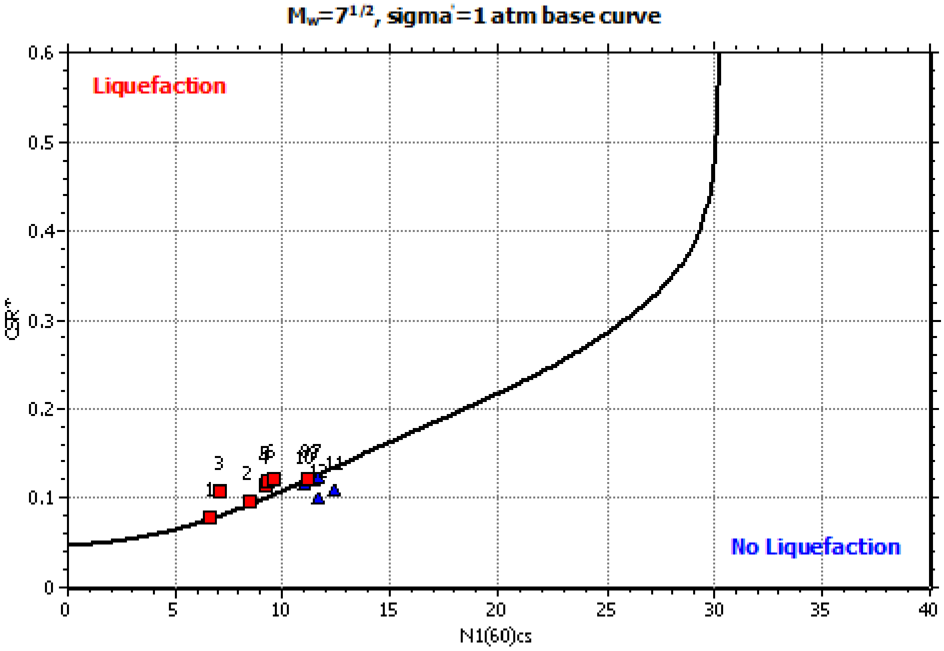

Figure 33.10 FoS Curve of DHA site at M=6.5

Factor of Safety against different magnitudes of earthquakes at DHA site, Nisatta is analyzed. DHA soil is Liquefiable till 30 ft at M=6.5 Earthquake but the depth of liquefaction increases with the earthquake magnitude and is completely liquefied at M=7.5. SPT & FS values are summarized in the table below:

Table 5.6 Factor of Safety against various magnitudes at different depths

Depth	Nm	(N1)60cs	CSR	CRR	FS			
Ft					M=6.5	M=7.0	M=7.5	M=8.0
4	3	4	0.155	0.1	1.11	1.07	0.89	0.74
8	4	5	0.162	0.09	0.98	0.83	0.68	0.57
13	5	6	0.196	0.1	0.88	0.81	0.66	0.55
18	7	8	0.216	0.11	0.91	0.88	0.71	0.59
25	8	9	0.227	0.12	0.91	0.89	0.72	0.59
30	8	9	0.229	0.11	0.87	0.87	0.69	0.56
35	11	11	0.228	0.13	1.02	1.01	0.8	0.65
40	11	11	0.224	0.13	1	0.99	0.78	0.62
45	12	11	0.218	0.13	1.05	1.09	0.85	0.67
50	12	11	0.212	0.13	1.05	1.07	0.83	0.65
60	15	12	0.198	0.14	1.21	1.2	0.92	0.71
70	15	11	0.184	0.13	1.22	1.19	0.9	0.69

5.2.2.1 Relative Density Check

Many empirical relations between soil index properties and Liquefaction potential are proposed by different scientists for easy and direct liquefaction assessment. One of the most important and common relationship is given by (Seed and Idriss, 1971) which compares Relative Density, D_r, of soil with the liquefaction potential of that soil.

Table 5.7 Relationship between relative density and liquefaction potential

Approximate relationship between earthquake magnitude, relative density, and liquefaction potential for water table 1.5 m below ground surface*

Earthquake acceleration	High liquefaction probability	Potential for liquefaction depends on soil type and earthquake acceleration	Low liquefaction probability
$0.10g$	$D_r < 33\%$	$33 < D_r \leq 54$	$D_r > 54\%$
$0.15g$	< 48	$48 < D_r \leq 73$	> 73
$0.20g$	< 60	$60 < D_r \leq 85$	> 85
$0.25g$	< 70	$70 < D_r \leq 92$	> 92

*From Seed and Idriss (1971).

Relative density for DHA Peshawar soil is 55% and earthquake acceleration is 0.24g. Which categorizes it in first category ie. “Highly Liquefiable”. This result is the same as deduced by the standard procedure in previous sections so the results of our previous approach are supplemented by this technique.

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Chapter 06

6 CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

After research of the case study and analyzing the two sites for their soil liquefaction potential through simplified analysis method, it can be concluded that:

- Simplified Stressed Based procedure is the most common and easy contemporary approach for soil liquefaction analysis.
- Being close to the seismic active zones, both the sites face significant seismic hazard. Though, they exhibit different soil liquefaction potentials.
- The soil in Risalpur Cantonment is predominantly silty clay with Fines content exceeding the criteria and a high enough relative density which makes it highly unlikely to liquefy. Further, the groundwater table is too deep to cause any liquefaction.
- DHA Peshawar site on the other hand pose significant liquefaction hazard and is highly susceptible to soil liquefaction. The strata is loose sand which is poorly graded with very less fines content. Groundwater table is very shallow. So it checks all the criteria of soil liquefaction.
- With ever increasing requirement to claim new land for construction in recent years, it is imperative to consider soil liquefaction potential for land use and planning.
- This study highlights the importance of soil liquefaction phenomenon that will encourage the people and eventually assist the national agencies to develop laws and regulations for economical land development and put forth particular remedial measures for soil liquefaction.

6.2 Recommendations

The soil of DHA Peshawar site is found to be highly liquefiable and any construction without considering liquefaction mitigation measures must be avoided. Generally, the most easy and effective strategy to avoid liquefaction associated damage could be to avoid construction on liquefiable soils altogether. But this strategy is not always possible due to ever increasing demand for new land for construction.

To ensure the safety of the structure and sustainable development, certain structural or geotechnical remedial measure should be put in place in such cases to avoid the liquefaction induced instabilities.

6.2.1 Deep Foundation (Piles)

Deep foundation or piles is an underground rigid column like structure on which the superstructure is resting. It is another strategy to avoid the liquefiable soils indirectly by bypassing it and transferring the structural loads to the more competent layer of the soil lying under the liquefiable layer. It is virtually impossible for a soil to liquefy beyond the depth of 15 m due to the overburden stresses (Florin and Ivanov). Thus by transmitting the loads beyond this depth via piles, we can avoid liquefaction hazards.

Piles is the only practicable solution for the DHA site because mitigating liquefaction by other approaches like dewatering will have little effect as the site as right on the river bank and a constant recharge of groundwater through the river is possible.

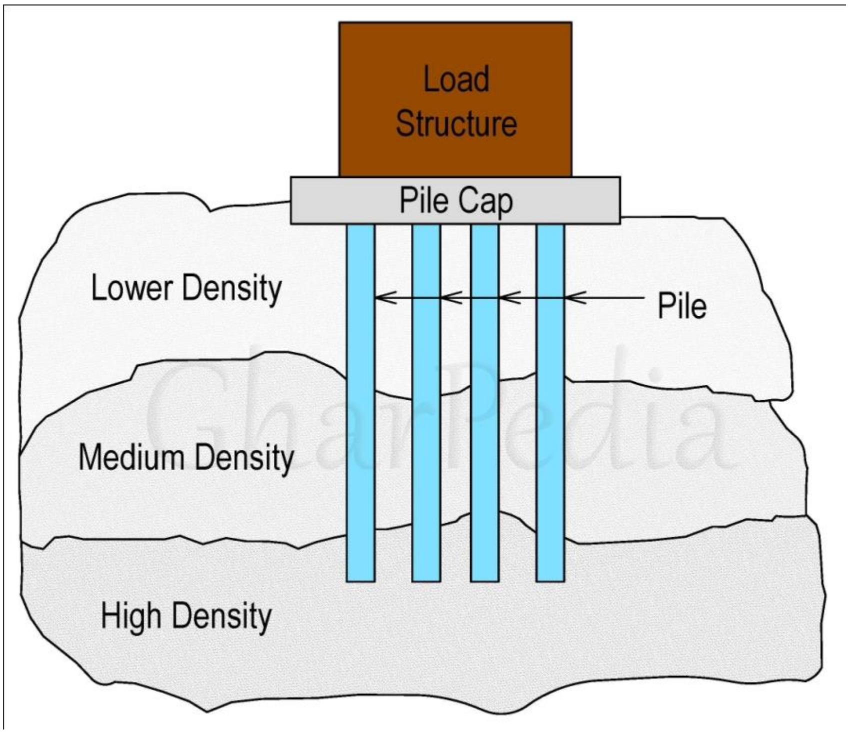


Figure 34.1 Deep foundation Piles

6.2.2 Compaction Grouting

Grouting is a process where low slump cement mortar is pumped into the ground surface (granular soils, fissured rocks etc.) to lower the permeability, increase the bearing capacity and shear strength of the soil etc.

Compaction grouting is a type grouting where the mortar is pushed into the soil, under high pressure, through a nozzle; forming a bulb which densifies the surrounding soil. The nozzle is then gradually pulled back while forming successive bulbs on top of each other. Thus forming a rigid column and densifying the surrounding soil.

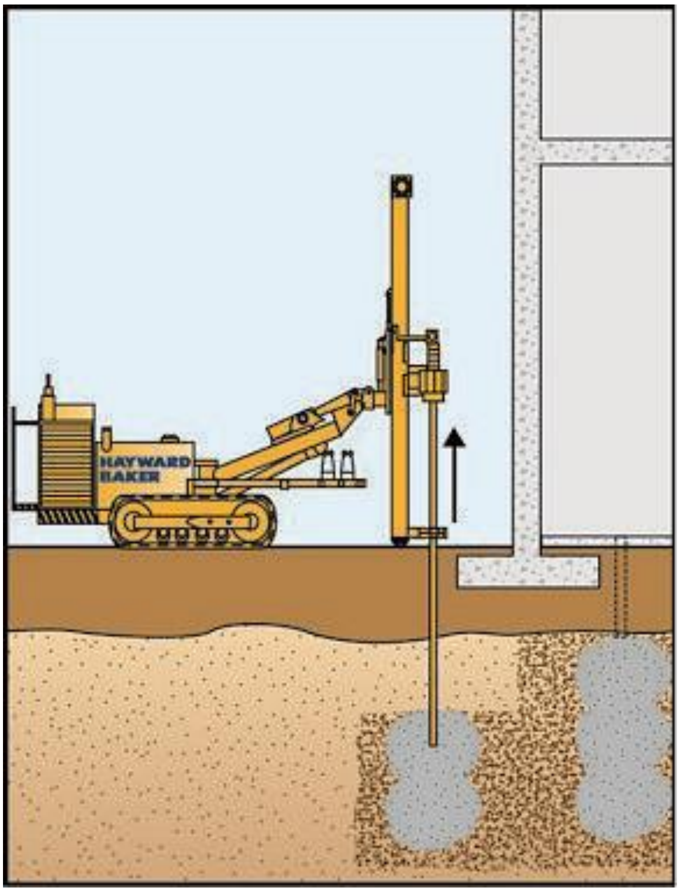


Figure 35.2 Compaction Grouting

6.2.3 Vibro-Floatation / Vibro-Replacement

Vibro-floatation is also known as vibro-compaction. It increases the relative density which results in reduction of settlements and hence increases liquefaction resistance. It involves the rearrangement of soil particles into denser state with vibrations by using vibrating probes. It is applicable to granular soils such as sands gravels and slags with fines content less than 10%.It depends on several factors including equipment size and quality, spacing and pattern.

Vibro-replacement is also known as stone column construction. It is an approach to improving poor soils through the construction of cementations gravel columns.. The technique applies up to a depth of 45m. It increases the bearing capacity, reduction of settlement and also mitigate the risk of liquefaction to the design depth in case of seismic events.

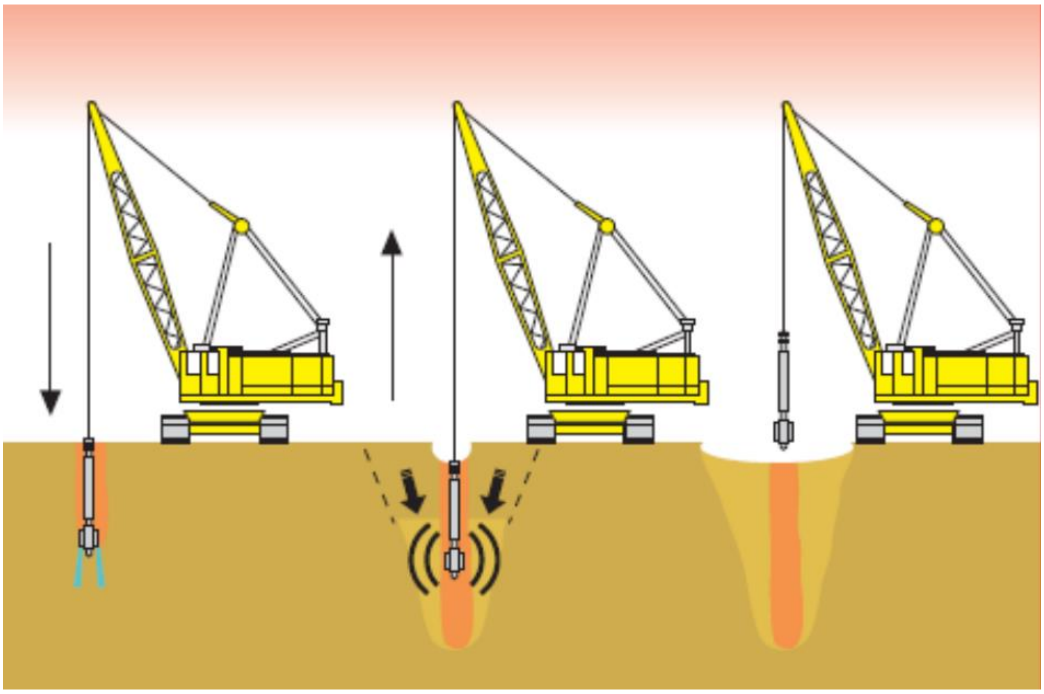


Figure 36.3 Vibro Floating

6.2.4 Groundwater Drainage

Lowering the groundwater table by draining is very effective to counter liquefaction as shallow groundwater table is necessary to initiate liquefaction. When groundwater table is deep enough, it becomes hardest for pore pressure to reduce the overburden stresses of overlying soil and thus liquefaction is not initiated even though the soil fulfills the geotechnical liquefaction criteria.

However, in case of DHA Peshawar, site is positioned just on bank of Kabul river. So any attempt to lower the groundwater table would be extremely difficult as the adjacent river will continuously recharge the groundwater in the permeable soil. In this case, other remedial measure are advised which are discussed below.

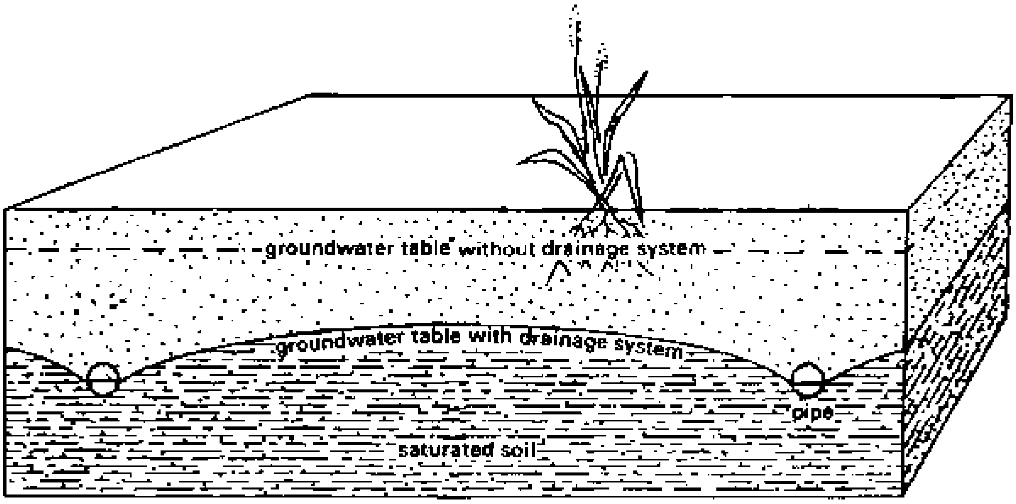


Figure 37.4 Groundwater Drainage

6.2.5 Soil Improvement by Adding Admixtures

Inorganic soil stabilization, also known as admixture, is the process of changing the physical and chemical qualities of soil by adding additives like cement, fly ash, or lime, or a combo of these.

There are two primary mechanism by which chemicals modify soils.

- • Cementation increases particle size, which increases internal friction and so improves the capacity of the soil and provides resistance to liquefaction.
- Absorption of excess moisture and locking free water in chemical bonds. This will also reduce liquefaction potential.

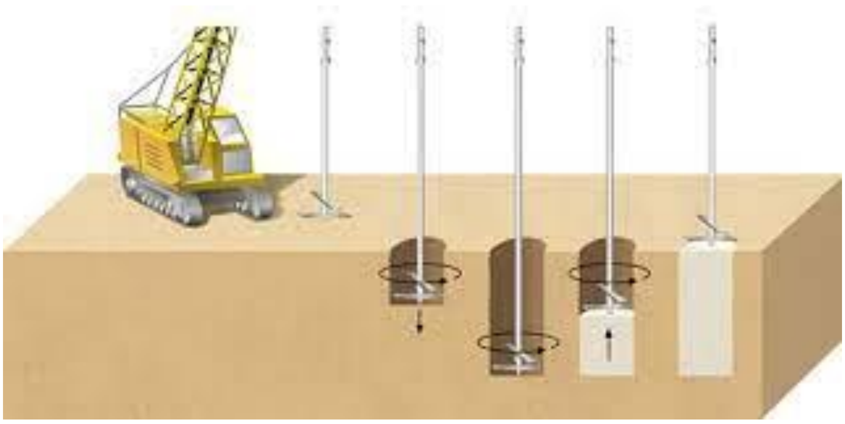
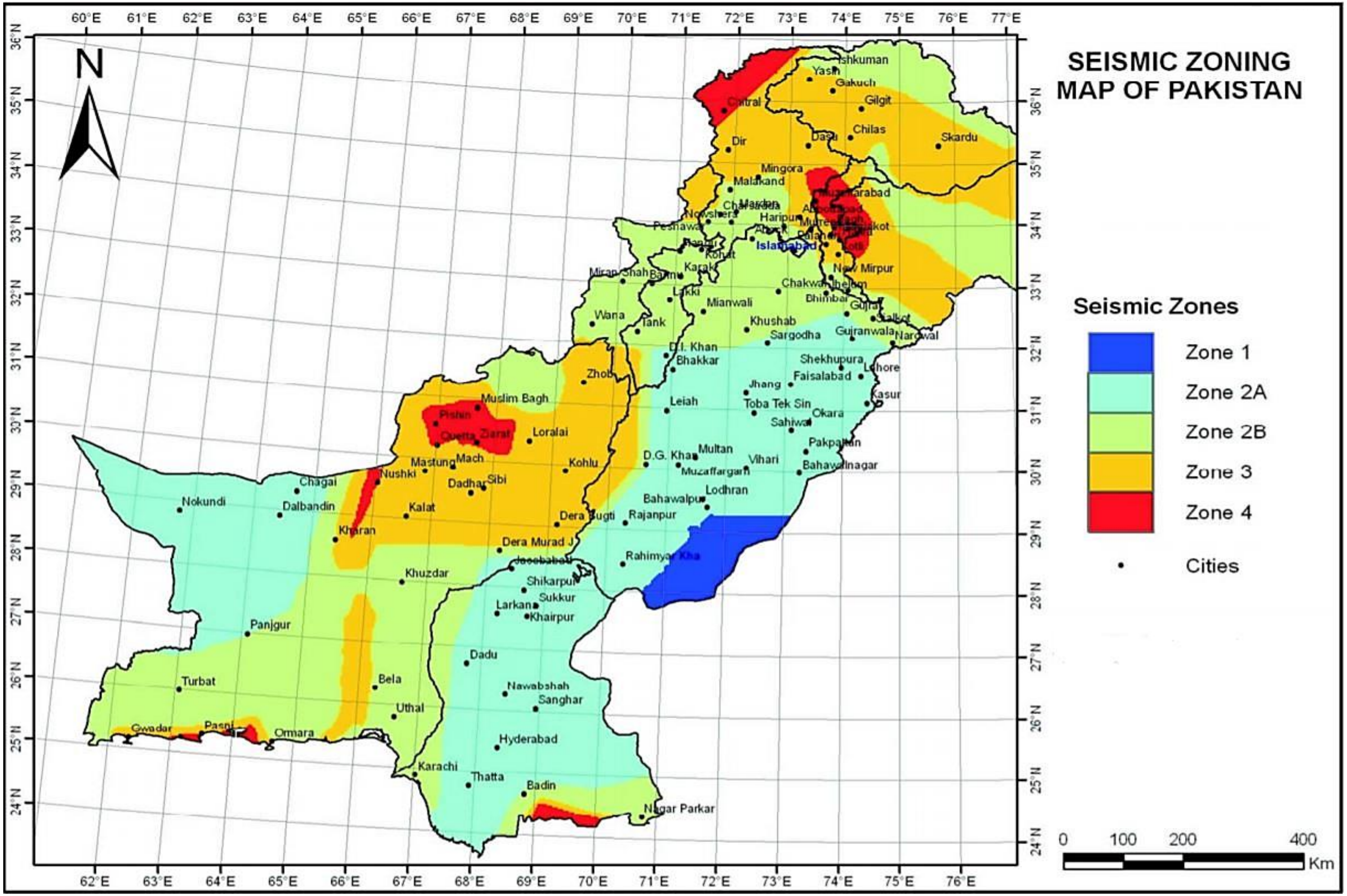


Figure 38.5 Deep soil mixing

ANNEXURE A

(Seismic Zoning Map of Pakistan)

ANNEX A


SEISMIC ZONING
MAP OF PAKISTAN
Seismic Zones
Zone 1
Zone 2A
Zone 2B
Zone 3
Zone 4
Cities
0 100 200 400
Km
N

REFERENCES

Ahmad, M., et al. (2018). "Assessment of soil liquefaction potential in Kamra, Pakistan." Sustainability **10**(11): 4223.

Cubrinovski, M., et al. (2011). "Soil liquefaction effects in the central business district during the February 2011 Christchurch earthquake." Seismological Research Letters **82**(6): 893-904.

Ishihara, K. and Y. Koga (1981). "Case studies of liquefaction in the 1964 Niigata earthquake." Soils and foundations **21**(3): 35-52.

Papadopoulos, G. A. and G. Lefkopoulos (1993). "Magnitude-distance relations for liquefaction in soil from earthquakes." Bulletin of the Seismological Society of America **83**(3): 925-938.

Seed, H. B. and I. M. Idriss (1971). "Simplified procedure for evaluating soil liquefaction potential." Journal of the Soil Mechanics and Foundations division **97**(9): 1249-1273.