TREATMENT OF LIQUEFACTION SUSCEPTIBLE LOOSE SATURATED CLEAN SAND WITH MAGNESIUM OXIDE

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B.S, Civil Engineering, Middle East Technical University, 2016

Submitted to the Institute for Graduate Studies in
Science and Engineering in partial fulfillment of
the requirements for the degree of
Master of Science

Graduate Program in Civil Engineering

Boğaziçi University
2021

ACKNOWLEDGEMENT

I would like to thank my supervisors Assoc. Prof. Dr. Özer Çinicioğlu, my father Prof. Dr. Murat Teker and my consultant Prof. Dr. Mustafa Küçükislamoğlu for all their help and advice with this study. I would also like to appreciate to my sister Hilal Gülnur Teker (MSc.) and my mother Hülya Teker, whom without this would have not been possible. I also appreciate all the support I received from the rest of my family

ABSTRACT

TREATMENT OF LIQUEFACTION SUSCEPTIBLE LOOSE SATURATED CLEAN SAND WITH MAGNESIUM OXIDE

Static liquefaction, more risky than cyclic one in some cases, is a strain-softening case of the granular soil as sudden loss of effective stress due to quick development of pore water pressure due to static and dynamic actions. Under zero effective stress, soil strength is lost, so it behaves like a liquid. To diminish its effects, there have been proposed traditional and recently developed improvement methods including chemicals, synthethic fibers, blast densification in the aim of lowering the saturation degree, filling pore space of soil and combining soil particles together. However, these have some drawbacks such as toxicity and disturbance to ambiance, being expensive and inoperable for specific cases. As an alternative, magnesium oxide (MgO) was offered in this study due to the vast usage on medical, agricultural and constructional industries. To prevent liquefaction occurrence, the hydration mechanism of MgO, producing magnesium hydroxide precipitates was used to reduce water content and to connect soil grains. Its impacts on the compressive strength of loose, fine, fully saturated liquefiable clean sand with MgO doses of % 0.5, %1, %2, %4, %6 respectively, were investigated. Treated cylindirical soil samples were cured under 23 ^{0} C and 41% humidity for 3,7 and 14 days. Several unconfined compression tests (UCT) were made on samples under strain-controlled condition. The addition of MgO after sufficient curing period removed to water content, provided to be failed earlier and made them stiffer and denser. Increase in MgO doses enhanced to unconfined compressive strength and reduced total and failure deformation capacity. Longer treatment supported to this by making them more brittle with higher Elastic Modulus, so ductility and durability was dimished. Finally, liquefiable soil was improved mechanically along with the preventetion of excessive deformation and saturation, so the possibility of liquefaction occurrence was decreased.

ÖZET

SIVILAŞMAYA YATKIN GEVŞEK SATURE TEMİZ KUMUN MAGNEZYUM OKSİT İLE İYİLEŞTİRİLMESİ

Sıvılaşma, bazı durumlarda depremlerden daha yıkıcı olan akış sıvılaşması olarak gözlenir. Akma sıvılaşması, statik ve dinamik hareketlerin neden olduğu boşluk suyu basıncının hızlı gelişimi nedeniyle anlık efektif gerilme kaybı olarak granüler zeminin yumuşayarak şekil değiştirmesi durumudur. Sıfır efektif gerilme altında zeminin direnci kaybolur, dolayısıyla sıvı gibi davranır. Sıvılaşmanın etkilerini azaltmak için, doygunluk derecesini düşürmek, zeminin gözenek boşluğunu doldurmak ve daneleri bir araya getirmek amacıyla, kimyasal enjeksiyonu, sentetik lif güçlendirmesi, patlatarak sıkılaştırma gibi yöntemleri içeren geleneksel ve son zamanlarda geliştirilmiş iyileştirme yöntemleri önerilmiştir. Ancak, toksisite ve çevresel örseleme, maliyet ve belirli durumlar için uygulanabilir olmamaları bunların negatif etkileridir. Alternatif olarak magnezyum oksit (MgO) medikal, tarım ve inşaat endüstrilerinde yaygın kullanımı nedeniyle bu çalışmada önerilmiştir. Sıvılaşma oluşumunu önlemek için, su içeriğini azaltmak ve zemin daneleri arasında bir bağlantı kurmak gayesiyle, magnezyum hidroksit çökeltileri üreten MgO'nun hidrasyon mekanizması kullanılmıştır. Bu nedenle, magnezyum oksitin sırasıyla %0.5, %1,%2,%4,%6 MgO dozları ile sıvılaşabilen düzgün, ince ve tamamen doymuş temiz kumun kompaksiyon mukavemetine etkisi araştırılmıştır. İyileştirilmiş numuneler 23 ^{0} C ve % 41 nem altında 3,7 ve 14 gün küre tabi tutulmuştur. Bu süre sonundaki iyileştirilmiş numuneler üzerinde çok sayıda tek eksenli serbest basınç (UCT) deneyi yapılmıştır. Bu sayede uzun kür süresi ve MgO eklenmesi, su muhtevasını düşürmüş ve erken kırılmasını sağlamış, kum daha sert ve yoğun hale gelmiştir. MgO artışı, zeminin serbest sıkıştırma mukavemetini artırır. Kırılma ve toplam deformasyon kapasitesini düşürür. Kür süre artışı bunu desteklemiş ve Elastik modulüsü artışı ile numunenin kırılganlığı yükselmiştir. Daha yüksek MgO içeren numunelerin tek eksenli basınç altında süneklik ve dayanıklılığı azalmıştır. Sonuçta, sıvılaşma riski ve aşırı oturma azalmış, zemin mekanik olarak güçlendirilmiştir.

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LIST OF SYMBOLS

Å (Ansgtrom)	Angstrom
Al_2O_3	Aluminium Oxide
B	Skempton's Factor
c	Cohesion
C_c	Coefficient Of Conformity
C_u	Coefficient Of Uniformity
CaO	Calcium Oxide
$Ca(OH)_2$	Calcium Hydroxide
CO_2	Carbon Dioxide
D_r	Relative Density
e	Void Ratio
e_{max}	Maximum Void Ratio
e_{min}	Minimum Void Ratio
e_o	Void Ratio of Soil
E_a	Axial Strain
EtOAc	Ethyl Acetate
Fe_2O_3	Iron(III) Oxide
Gmax	Maximum Shear Modulus
Gs	Specific Gravity
H_2O	Water
J	Joule
K	Kelvin
$MgCl_2$	Magnesium Chloride
$MgCl_2.6H_2O$	Magnesium Chloride Hexahydrate
$MgCO_3$	Magnesium Carbonate
MgO	Magnesium Oxide
$Mg(OH)_2$	Magnesium Hydroxide
N	Loading Cycle Count

N_{Liq}	Cycle Counts For Liquefaction
N/ N_{Liq}	Loading cycles generalized by Counts For Liquefaction
q	Deviatoric Stress
q_{max}	Minimum Deviatoric Stress
q_{min}	Maximum And Minimum Deviatoric Stress
q_{peak}	Peak Deviatoric Stress
$q_{residual}$	Residual Deviatoric Stress
q_u	Unconfined Compressive Strength
R_u	Porewater Pressure Ratio
$R_{u,max}$	Maximum Porewater Pressure Ratio
SiO_2	Silicon Dioxide
Sr	Degree Of Saturation
S_u	Undrained Shear Strength
u	Pore Water Pressure
W	Water Content
Δu (Delta)	Excess Pore Water Pressure
φ (phi)	Friction Angle
μ (mu)	Micron
ρ_{dry} (Rho)	Dry Bulk Density
$\rho_{saturated}$	Saturated Bulk Density
σ (sigma)	Total Stress
σ_1 (sigma)	Major Principal Stres
σ_3 (sigma)	Minor Principal Stress
σ' (sigma)	Effective Vertical Stress
$\sigma'o$ (sigma)	Effective Confining Pressure
$\sigma'o /\Delta U$ (sigma)	Generalized Excess Pore Pressure
$\sigma'vo$ (sigma)	Initial Vertical Effective Stress
τ (Tau)	Shear Strength
Ω (Omega)	Ohm

LIST OF ABBREVIATIONS

3D	Three dimension
ASTM	American Society for Testing and Materials
BET	Brunauer-Emmet-Teller
BS	British Standard
CBR	California Bearing Ratio
Cbz	Carboxbenzly
CD	Consolidated Drained Test
CP	Confining Pressure
CS	Collodial Silica
CSC	Collodial Silica Content
CSH	Calcium Silica Hydrate
CSR	Cyclic Stress Ratio
CRR	Cyclic Resistance Ratio
CU	Consolidated Undrained Test
EDX	Energy-dispersive X-ray
EICP	Enzymatically Induced Calcite Precipitation
FA	Fly Ash
FL	Fiber Length
GO	Graphene Oxide
GSD	Grain Size Distrubition
LL	Liquid Limit
MICP	Microbially Induced Calcite Precipitation
MOC	Magnesium Oxychloride Cement
MSH	Magnesium Silica Hydrates
PC	Portland Cement
PI	Plasticity Index
PL	Plastic Limit
RPM	Rate per minutes

SEM	Scanning Electron Microscope
SP	Poorly Graded Sand
TAS	Triaxial Automated System
TGA	Thermogravimetric Analysis
UCS	Unconfined Compressive Strength
U.S.	United States
USCS	Unified Soil Classification System
UCS	Unconfined Compressive Strength
UCT	Unconfined Compression Test
UU	Unconsolidated Undrained Test
XRD/F	X-Ray Diffraction/ Fluorescence

1. INTRODUCTION

In 2008, there was an earthquake with magnitude 8.0 happened at Wenchuan. The consequences were very intense and shocking as much as an earthquake. Over an extended band-like zone close to dynamic fault, ten-million-hectare area was affected and approximately 70,000 lives were lost along with the ground collapse, a number of fractures in the utility lines,highways and buildings. Instead of the shaking, main reason belonged to the liquefaction as anoutcome of the earthquake. Nearly 120 sites throughout the region were come up with this destructive event. Like Wenchuan, the earthquakes on Kocaeli, Bhuj, Chile and Kobe endangered to many lives and properties. Numerous roads, bridges, ports, retaining walls were extremely damaged or destroyed in consequence of lateral spreading, ground mobility, excessive permanent horizontal and vertical deformation, and loss on bearing capacity offoundation systems. Moreover, the effects of the liquefaction were observed in the environmentby leading to land instability such as landslide on the sloping ground, boiling in sands, water eruption, ground oscillation, and the displacement around the rivers edge or at the water courses.In the 62% of every 50 destructive earthquakes, the ground surface was subjected to the liquefaction. Even, 15-30% of whole earthquake casualties was by reason of this problem.

As the definition, liquefaction is the progressive or rapid loss of shear strength and stiffness of loose saturated granular soil, behaving like a liquid at the end due to sudden or many repeated changing in the stress conditions caused by large static and dynamic loadings.

As mechanism of liquefaction, the support system of soil includes the frictional contact forces between grains based on the building and the overburden loads of soil layers and thus, soil shear strength is created, which depends on the fabrics and the level of cementation between particles. For fully saturated loose granular soils, the generation of excess pore water pressure nearly does not happen because pore water finds the path to dissipate easily under the steady loadings lower than shear strength of soil within sufficient time. On the other hand, when changes in stress condition on the soil due to monotonic sudden strong loadings or cyclic repeated seismic actions was happened, water could not be squeezed out within inadequate andvery short time in advance of another loading cycle. At this moment, as soil response against compression, the volume becomes nearly constant due to the entrapment of porewater, showing incompressible characteristics. Therefore, external pressures are transmitted from the soil skeleton to porewater under undrained condition, so porewater pressure starts to be built. As water pressure increases, the soil particles start to leave from each other along with prevention of soil aggregation. Hence, there is a reduction in the contact forces between grains. This breakage in the soil structure leads to softening and gradual loss of effective stress in the soil. Depending on the condition whether all external loading equals to pore water pressure or not, shear strength of soil would be diminished or totally lost, known as liquefaction initiation. In absence of shear resistance, soil grains are suspended in the water and soil behaves like a liquid.In case of reduction in shear strength, static and cyclic shear stress becomes greater than steady-state shear strength of soil, so the liquefied soil could be largely deformed under very low staticor cyclic shear stress because of an inadequacy of the conveyance capability of shear stress on the soil.

The possibility of liquefaction is affected by several factors associated with the soil such as relative density of soil, in-situ vertical effective stress, grain-size distribution, type of soil, seismic history, age, shape and origin of the soil, deposition, saturation level, permeability and consolidation condition, thickness and state of cementation, sedimentation characteristics of the soil and drainage condition in the soil.

Besides, the characteristics of earthquake loading such as magnitude, existence of stress return, period of shaking, distance to earthquake center are also constitutive elements for liquefaction. Regarding type, gravels and clays are not appropriate for liquefaction, whereas loose saturated sandy and non-plastic silty soils having low density and poor drainage are most risky types for liquefaction potential. Moreover, if the sediments placed in the coast or streambed are very young and formed last 10,000 years, liquefaction susceptibility becomes greater. As an example of liquefiable soil, fully saturated clean sand near to ground surface having low permeability, uniform gradation and rounded soilparticles, very loose state, newly deposited and no cementation between grains, no preloading or seismic shaking before, the location close to fault rupture of major earthquakes could be liquefied most easily compared to others. The shear resistance of clean sand only depends on the friction angle and level of vertical effective stress and under undrained shearing coming from seismic shakings or sudden static surcharge, the absence of cohesion leads clean sand to have very low or zero residual strength at liquefied state, depending on its properties stated in above. At this point, based on previous investigations and reports, most susceptible soil to liquefaction was used in this study.

The liquefaction is divided into two types as flow and cyclic liquefaction. The flow liquefaction occurs in the situation that steady-state and residual strength of liquefied soil is lessthan the shear stress needed for a static equilibrium within the slopes or foundations under structure due to static loadings such as extra loading on the embankments or loss of toe supporton the slope or dynamic loadings such as pile driving and blasting. It is very sudden and devastating type of liquefaction. After initiation of flow liquefaction, there is a softening for thepoorly drained soil. At this state, soil's effective vertical stress becomes constant and deformation in the soil continues with low constant residual shear strength along with the duration of liquefaction. Therefore, depending on the dimension of region and kind of a soil, flow liquefaction leads to large shear deformations and settlements, landslide and flowing on the slopes, collapse of soil under the foundation during dissipation of excess porewater pressure. Moreover, soil mass, displaced after first flow liquefaction can experience another liquefaction occurrence due to inadequate strength compared to external actions. The soils susceptible to flow liquefaction are low permeable saturated silty sands, fills, gravelly sands having low stability in the slope or under the footings. Additionally, for the occurrence of flow liquefaction, it is not needed that effective vertical stress of liquefied soil becomes zero. However, flow liquefaction is not possible for the dilative soils such as medium and dense sands.

In cyclic liquefaction, effective stress of soil is gradually decreased until it becomes zero under only many repeated cyclic actions rather than static ones such as earthquake. Unlike to flow liquefaction, shear strain after every cycle of the soil is incremented and collected in the soil. Next, effective stress of soil becomes zero at the initiation of cyclic liquefaction, where steady-state shear strength of soil is less than cyclic shear stress acting on soil. Since the repeated loading mechanism exist in the cyclic liquefaction, gradually increased shear strains atthe liquefaction initiation are generally greater than that of flow liquefaction. After zero effective stress, leading to loss of shear strength in the soil, great shear deformations and ground failures are observed during the dissipation of excess porewater pressure. Therefore, cyclic liquefaction causes to sand boiling at ground surface due to upward movement of excess pore pressure, collapsing under the foundation due to loss of bearing capacity of soil and landslide asthe result of flow failures. Similarly, when there is continual decline in effective stress which does not become never zero during liquefaction induced by earthquake, the type of cyclic softening is known as cyclic mobility. In here, cyclic shear stress is less than the steady-state shear strength or residual shear strength of liquefied soil. Nevertheless, if cyclic shear stress exceeds the permissible level, there may be permanent deformation and cyclic failures such as ground oscillation and lateral spreading, occurred in the very gentle slopes with 1-2 angles.

Failures for both types of soil are related with physical properties and type of soil, period and intensity of earthquake, distance to earthquake center and the presence of stress recovery, where effective stress of soil can reach to zero and cyclic liquefaction may be triggered instead of cyclic mobility. All kind of soil can experience to cyclic softening, but such failures could not be possible for dilative soils such as dense sands or stiff clays due to strength development during cyclic actions. Additionally, after cyclic softening occurrence in the soil by massive shakings, the ability of water to carry all external actions is not probable anymore. Therefore, soil skeleton could start to resist against vibrations and the impact of earthquake into structuresis reduced by this way.

When the effect of liquefaction on the soil and the structure above is considered, the damages have important effect on the economy, because the level of casualties may exceed to billion dollars. Therefore, mechanical characteristic of liquefiable soils should be treated. There were several previous traditional methods including removal and replacement, densification, reinforcement, grouting and mixing and drainage with various application procedures forvarious types of soil. Vibro-compaction and blast densification, vibro-stone columns andcompaction piles, permeation and jet grouting with deep soil mixing, earthquake drain and dewatering are some examples of these.

Although all traditional methods are extensively used to enhance mechanical, hydrologicaland drainage properties of soil, several disadvantages such as negative effect on environment by chemicals, disturbance to existing buildings by shaking and deformations during applications, greater expense for wider area treatment are revealed. Moreover, some of them arenot applicable near to groundwater table, existing structures or underground and others requirenecessary gap to be implemented. Hence, when the concerns about applicability, compatibility and feasibility over real site cases are thought, newly developed techniques are needed. Recenttechniques have the purpose to decrease the pore space, to combine the soil grains together andto reduce the saturation level.

There are also methods such as partial saturation method, usage of short synthetic fibers, chemical grouting, microfine cement, colloidal silica injection, laponite and bentonite-based treatment, improvement by biological matters and the utilization of other additives as fly-ash, zeolite, graphene oxide nano-sheet under these recent techniques. As the application process over the site, some techniques need the high pressurized infiltration and other requires the mixing of binders with soil. Based on this, pore size of soil may be obstacle for microfine cementand other additives for grains higher than 16 μ or biologically induced improvement appropriate for soil smaller than 0.4 μ, except that all of these are effectively diffused into soil. Regarding partial saturation except very economical compared to chemical grouting, the depth and type ofsoil needed to be improved may require specific conditions because partial saturation with air infusion way cause to increase flow speed through the compressor and thus, disturbance to ambiance may arise. In the soil with short-synthetic fibers which is economical and high durablein the soil compared to chemical grouting, variable specifications exist.

Besides, because of excessive drainage pressure needed for injection of microfine cementand other additives, the disturbance to closer buildings, irregular distribution of chemicals on desired depth, high cost of implementation for finer soils and toxicity for environment. Even, some chemicals with expansive characteristics cause to ground heaving under the building base.Nonetheless, the level of disruption to nearby structures for chemical grouting is less than that for traditional ones.

In microfine-based improvement, manufacturing cost is not economical, but overall cost compared to Portland Cement is less when permeability and resistance after improvement is considered. Although the usage of nanomaterials provides great strength and permeability, disturbance to existing structures, high cost and environmental impact still exist. For biologically based treatment as biogas and bio-solidification, biological procedures in the site condition are not known well and applicable for certain soil types, even if environmental dangerand the cost is reasonable in this method.

Additionally, most of these new methods are at the stage of laboratory investigation and there is no any real implementation on the site. The treatment level of soil, the period and expense of project, delivery and extracted contaminants are changing for different countries, sothe estimation of certain cost for these techniques is hard to obtain. Likewise, there are several specifications such as unit cost, ratio of chemicals, effectiveness, implementation process, machinery types and impact on ambiance, which makes cost calculation difficult. For durability evaluations of newly developed techniques, investigations in the treated soils for longer period are difficult to conduct and many of techniques at laboratory stage are not realistically implemented in the site. Lastly, the production of Portland or microfine cement gives large CO_2 amount to the environment.

When the disadvantages of traditional and newly developed methods are considered, alternative methods with different types of additives are needed to reduce the civil, environmental, economic consequences of liquefaction. For this purpose, the usage of magnesium oxide additive can increase the physical and hydrological properties of liquefiable soils. When general utilization of magnesium oxide is investigated, it can be observed that thereare many investigations about the effect of magnesium oxide on cement and soil. In general, MgO as known as periclase is white solid mineral, which has very high melting and boiling points (2800 ºC and 3600 ºC) and two ions as Mg^{+2} and O^{-2} joining with ionic bonding. Magnesium oxide is mostly produced from the calcination of magnesium carbonate, $MgCO_3$. Depending on the calcination condition as the period and temperature, specific surface area and activity degree of magnesium oxide are determined. Hence, the types of magnesium oxide are obtained.

Regarding types, very reactive light-burned magnesium oxide is formed at 700-1000 ºC temperature, whereas hard-burned one produced at 1000-1500 ºC temperature has low reactivity. For dead-burned magnesium oxide generated at 1500-2000 ºC temperature, the activity is the least and it is used for refractory purpose. Magnesium oxide is the materials extensively used in variable industries from nuclear area to medical and drug sectors involving chemical, medical and agricultural scopes. Besides, other utilization is related with the goal of treatment of water, fertilizers and isolator, waste extermination. In addition to environmental effect, magnesium oxide has important role on groundwater reclamation due to its basic pH value by preventing the effect of heavy metal and acidic substances, which are toxic to the livings and nature. Hence, its effect towards the stabilization of metals is more dominant to Portland cement, lime and dust kiln because of greater safety and more economical aspects.

Despite those practices, the usage of magnesium oxide has been observed in civil engineering materials as cement industry. In case of addition of MgO with the content less than 5% of dry weight of Portland cement, there is an enhancement in swelling, permeability, and compressive strength properties of Portland Cement along with the decrease in water adsorption. Although the reactive magnesium oxide may lower the mechanical outputs of Fly Ash by damaging to the packaged system, dead-burnt type or lower reactive one does not have such impact on Fly Ash. The main reason is due to specific surface area and water demand of MgO, leading to porosity increase through the volumetric expansion in the system, so larger pores are not filled with magnesium hydroxide (brucite) and even, some cracks may arise during compressive testing on the treated Fly Ash. Hence, reactive MgO also causes to raise in water adsorption. However, water demand for dead burned MgO is less than reactive MgO and Portland Cement due to low specific surface area.

In fact, unconsumed MgO behaves as filler for void space of soil, so the porosity increase in the system is diminished and compressive strength is enhanced with the usage of dead burnedMgO. In another actions of MgO substituted by the Portland cement with 10%,15%,20% of dry weight of PC with sand aggregates, crystalline solid $Mg(OH)_2$ or brucite after MgO hydration with water is responsible for strength gaining at early age (28 days), but brucite connects to the silicate molecules of sand in order to generate Magnesium Silicate Hydrates (M-S-H), whose resistance is much more than that of brucite at longer age (60-70 days). It is known that reactivityand quantity increase in MgO leads to rise in water demand and absorption, whilst more porous and poor interlinking and reticular system happens, ensuing reduction in mechanical performance. Furthermore, increasing in porosity on the system make carbonation easier and it is the fact that carbonation of MgO in cement has positive impact on the strength, water adsorption and durability. In addition to magnesium oxide on Portland Cement, the advantagessuch as low CO_2 emission, energy preservation, CO_2 absorption from atmosphere, being environmentally friendly give a reason to produce MgO-based cements.

Furthermore, there is utilization of MgO in Magnesium Oxychloride Cement (MOC), which contains magnesium oxide (MgO), magnesium chloride ($MgCl_2$) and water. In the certain ratios between components and the characteristics of Phase-5 crystals, competitive compressive strength with respect to pure Portland Cement would be observed. Again, MgO reactivity rather than Phase-5 Crystal or water content has great impact on the strength and decrease in specific surface area provides better network towards porosity. Additionally, the reason to increase in strength of MOC in case of reduction in reactivity of MgO is the unconsumed MgO which complete the pores as a filler. It is also revealed that addition of silica-based material augments the strength, drops the water adsorption and porosity degree of MOC, because silica-based material as a filler gets into the pores and thus, lowers water content along with raising the density.

In addition to Magnesium Oxychloride Cements, there are also reactive MgO cements, Magnesium Phosphate Cements, Magnesium Silicate Hydrate Cements, Magnesium Oxysulphade Cements under deeply investigations nowadays. In short, magnesium oxide is one of significant materials used in civil engineering industry for the period exceeding 150 years.

Also, based on studies concerning MgO, the influence of MgO on the various soil such as fine loose sand, high and low plastic clay, sand silty sand, clayey silt is very beneficial,because physical, mechanical and hydrological performance are advanced. It is obvious that nano-size MgO addition provides the reduction in deformation and porosity through filling the void between grains, ensuing greater interlocking and thus, strength and stiffness of soil increases. Also, there is an output about whether strength gaining and hydraulic conductivity reduction after addition of $Mg(OH)_2$, brucite exist. Next, it is known that nano-MgO particles controls theexpansive behavior of plastic clays by the chemical reaction between the ions from MgO and clay and thus, endurance and resistance of clay treated with nano-size against surrounding actions has development.

However, swelling potential and plasticity could be reduced due to the cations exchange and pH of complex through the addition of magnesium hydroxide precipitation to clay. For the sand with clay and silt, both standard and nano-size MgO additives lower the plasticity and liquidity of soil along with the increase in compressive strength.

Moreover, it makes clayey sand stiffer and very stiffer at only 1% content or less without the grain size level. There is also finding related with the positive impact of MgO on the density and compressive resistance of silty sand treated for certain periods and in the presence of CO_2 in the environment, mechanical result of silty sand cured with MgO and CO_2 has 20 times increment compared to that in the absence of CO_2. Thus, in a very short period of time, MgO treated soil under CO_2 curing can give the compressive strength more than two times of that of soil cured with Portland Cement. To obtain desired result through CO_2 curing, low water content, high porosity and MgO content sufficient enough are needed and carbonation process is only efficient at the beginning of the hydration, not after the precipitation of brucite. Besides,expansion of MgO after hydration increases due to carbonation. Hence, there are several significant limitations of carbonation as much as positive impacts on the MgO. In this study, carbonation of MgO is not applicable due to saturation degree of soil, in which all pores are filled with water [1-37].

Even though MgO is extensively used from civil engineering industry to the investigation of treatment of soil, there is a still lack of research about its effect on the liquefiable soils. Based on various advantages in terms of building materials and geotechnical engineering, the question about how MgO influences the physical, mechanical and hydrological properties on a fully saturated sand with fine size and uniform gradation is unknown. For the conditions in the previous efforts, the water content, which is below water content for fully saturation, is changing and the compaction with variable degree exist after the mixing.

On the other hand, as mentioned in above, one of liquefaction condition is the close distance of liquefiable soil to the ground surface and another situation is the fully saturation. Under these circumstances, the effect of liquefaction into lives, properties and environment reaches to the maximum level. Hence, to observe the impact of MgO on liquefiable soil at best,MgO additives should be mixed with saturated liquefiable sand without compaction.

For materials and preparation, necessary checks about whether or not the soil used in this study conforms to the properties of liquefaction susceptible ones based on the results of critical friction angle, relative density, the parameters from grain size distribution, both geological and physical features of a soil and UCSC classification. Necessary comparison is needed to be madeaccording to previously proposed data. After this step, loose clean uniform sand and hard-burntMgO in powder form are used. The content of MgO is successively 0.5%, 1%, 2% ,4%, 6% of dry weight of soil and water content is found by corresponding formulation, after void ratio and specific gravity under lab condition are determined. Then, soil mixed with different MgO content in dry condition is poured into the certain amount water by wet-pluviation method to make loose state as much as possible.

Afterwards, loose, and fully saturated MgO-sand complexes in mold are formed without any compaction. By this way, the most critical situation for liquefaction is generated along with zero compaction case and the effect of magnesium oxide on the soil could be easily observed in terms of improvement against liquefaction.

Next, all samples are tripled and cured for 3, 7 and 14 days separately under room condition.Then, specimens are analyzed on the Unconfined Compression Test (UCT) under strain-controlled condition. Therefore, there are 15 samples involving magnesium oxide waited for different curing periods.

Necessary outputs such as maximum compressive strength, failure strain, end strain, water content along with physical assessment on the sample after curing period are taken into account to conclude the effect of MgO on liquefiable soil. Accordingly, the liquefaction susceptibility of used clean sand and change in UCS and deformations through MgO addition and curing periods are discussed.

To sum up, the effect of magnesium oxide on the static liquefaction performance of the clean liquefiable saturated sand is investigated in this study and the aims are sorted as removingwater content through the hydration of MgO and connecting soil particles to each other by precipitation of magnesium hydroxide so that saturated clean sand has no longer liquefaction risk and improved geotechnical properties thanks to MgO powder, which is very economical and safe for environment.

As the presentation of parts, the investigations are divided into four parts. In first part, thedefinition and mechanism of liquefaction is performed with the types, causes and evaluation ofliquefaction and liquefaction susceptible soils are specified. In second part, general knowledge and hydration process of magnesium oxide with the utilization on both material and geotechnical type of civil engineering is stated. In third part, traditional and newly developed methods for liquefaction mitigation are described with the feasibility studies. Then, materials and indexes are mentioned with implementation mechanism in Method part.

Lastly, unconfined compression test is conducted on clean and treated sand samples with different magnesium oxide contents as 0.5%, 1% ,2%, 4%, 6% of dry soil weight as percentage under fully saturation. At the end, necessary outputs are submitted to make a conclusion about how magnesium oxide affects the mechanical properties of liquefiable clean sand.

2. CONCEPT OF LIQUEFACTION

2.1. Definition

Liquefaction is the matter of fact which is rapid and quick loss or reduction of shearing resistance of soil behaving like a liquid under static and dynamic forces. When shear stress required for stability of a soil mass is more than shear resistance of the soil under liquefied condition, liquefaction happens. It generally occurs in loose, saturated cohesionless soil depositssuch as clean or silty sands [1-4].

2.2. Mechanism

Soil deposits consist of the unification of individual soil particles contacting with each other. The overlying soil mass provides contact forces between the soil grains. Therefore, overburden loads of upper soils are transferred into these contacts which keep each particle in stable place and by this way, shear strength of soil to support soil system is formed.

In general, inner support of soil over overburden loadings may be destroyed by liquefaction under fully saturation, even if it seems that it is very strong. Main reason is due to quick and powerful seismic or static actions caused by an earthquake or dead overburden loads. During an application into the soil within short period of time, solid skeleton of soil cannot respond against this situation. For this reason, volumetric compression cannot happen under quick condition, whereas excess pore water pressure is raised up until pore water pressure is lessthan or equal to the total stress called as an initiation of liquefaction. As a result of this, effective stresses may be lowered or zero, thereby stiffness and shear strength based on effective stress would be lost or diminished.

Regarding the detailed microstructural mechanism of liquefaction, the sudden static or repeated vibrational actions only force porewater to be contracted at initial, but time is not adequate for porewater to be squeezed out as the response against contraction known as entrapment of porewater pressure. Therefore, porewater pressure takes external surcharge onto itself and thus, it begins to increase.

At the same time, loosely packed soil grains cannot react over rapid loading due to its solid configuration. Hence, entrapped water pressure, increasing gradually, causes soil grains to diverge from their original location so the soil elements do not move into closer or denser formation themselves and contact forces between particles decrease.Therefore, debilitating of the soil aggregation due to contact forces reduction causes softening and lowering in the shear strength of soil.

Moreover, in case of too much increment in pore water pressure, the soil grains get lost their support, thereby disunited from each other. Thus, the soil resistance gets smaller and even,it goes down below the shear stress required to a stability of soil deposit. Eventually, the powerful and rapid static or multi-cycled dynamic actions induced by an earthquake or dead loads cause to the reduction in the connection of soil particles, so soil is suspended into the water and behaves like a liquid [3,5,6].

Briefly, liquefaction is related with pore pressure build-up followed by reduction in soil'seffective stress and by this way, the shear strength of the soil is decreased into the level under shear stress needed for an equilibrium of a soil deposit [1].

2.2.1. In Sand

The decrease in shear resistance because of pore pressure development within the soil skeleton is caused by the liquefaction occurrence.

Based on Mohr-Coulomb envelope, the shear resistance (τ) and vertical effective stress are expressed in Eq. 2.1 and Eq. 2.2 as follows [6]

$$\tau = \sigma' \tan \varphi \tag{2.1}$$

$$\sigma' = \sigma - u \tag{2.2}$$

where

τ = shear strength,

φ = friction angle,

σ' = effective stress,

σ = total stress,

u = pore water pressure.

For clean sand with no fine content, the cohesion becomes zero and thus, shear resistance,is affected by only effective stress condition and angle of friction related with arrangement of soil particles. Under undrained situation caused by sudden monotonic forces or repeated earthquake shaking, all external actions cause the shock inside soil system. Even though chemical and morphological structure of clean sand is not appropriate to retain water, soil skeleton does not have any response against a shock-based changes. Therefore, instead of passing by adjacent soil deposits, water pressure in the pores develops under constant volume exerted by sudden loadings. For this reason, based on Mohr-Coulomb formula in above, the liquefaction occurrence on the clean liquefiable sand is resulted with loss of shear strength of particles depending on only the friction angle and confining effective stress in the absence of cohesion [1].

2.3. Types

As stated in above, liquefaction is the situation of change in soil's state from solid into liquid as a result of increase in porewater pressure with the decrease in effective stress of soil. The increase in pore water pressure is due to the compression tendency of the granular soil subjected to shear strain due to static and cyclic effects. The porewater can not move out becauseof the rapidity of these loadings and thus, it happens that there is an increase in the pressure within the pore water forced into compression

There are two types of liquefaction in the nature depending on the soil type, the way of loading, relative density, the presence of inclination and etc.

2.3.1. Flow Liquefaction

The flow liquefaction, as first, happens under the condition that the shear strength of liquefied soil is below the shear stress needed for static equilibrium. After the flow liquefaction is occurred rapidly, there is a softening with loss on the shear strength leading to large shear deformations under the static shear stress within the soil. Moreover, when shear deformations reach to some level, flow failure or collapse may be observed.

In addition to this, flow liquefaction is happened in the loose granular soils having low-permeability such as silty sands, less permeable fills, gravelly sand where porewater is not drained. Likewise, soil in the slopes or under the foundation are often subjected to flow liquefaction, so flow failure may be caused.

Specifically, large settlements in loose soils are derived from the compression during thedissipation of excess porewater pressure.

2.3.2. Cyclic Liquefaction

For cyclic liquefaction, it happens when the seismic actions such as earthquake exist in the ground. Therefore, the cyclic shear stress is applied into the soil during the earthquake. At this point, shear stress required for static equilibrium of soil mass is less than the shear strength of liquefied soil.

Nevertheless, temporary strain softening with incremental cyclic deformations. In case ofthat shear stress exceeds the permissible level, it causes to permanent shear deformations and cyclic failures within the soil such as the ground oscillation and lateral spreading. For the soil having dilations features against cyclic shear stresses, either the loss in the shear strength or huge shear deformation is not observed.

In general, cyclic liquefaction is observed in the medium dense soils with very small inclination.

2.4. Susceptibility

2.4.1. Features of Liquefaction Susceptible Soils

The factors causing liquefaction are specified as magnitude of earthquake and distance tocenter, the composition and geological history of soil layer, natural effective vertical stress and relative density of the soil.

According to all liquefaction events observed in past, the liquefaction probability depends on the magnitude of shallow earthquake and the distance to the center which are defined with the curve. Soil in the sites close to the epicenter or location of fault rupture of a major earthquake has great liquefaction risk.

Several features of a soil layer such as geological history, hydrological condition and deposition type give significant notice about the possibility of liquefaction. Based on this, uniformly graded, newly deposit and loose relative density soils formed during sedimentation process bear the liquefaction risk.

In addition to this, the depth of ground water table is very important for liquefaction occurrence. Since liquefaction occurs because of an excess pore pressure increase under static or cyclic loading, grain size distribution affecting the volume change and permeability of soil isvery important for the liquefaction susceptibility. In opposition to general opinion about whether liquefaction only occurs on the cohesionless sandy soils or not, it was observed from recent investigations that low cohesive silts and gravelly soils may undergo the liquefaction.

In general, it is accepted that there is no liquefaction in the clayey soils. For evaluation ofliquefaction in the soil having low plasticity and fine grain size, some properties of a soil such as liquid limit and clay content must be considered. From several observations from various site conditions, different criterions including liquid limit, the index of plasticity, liquidity and grain size, the natural water content, clay percentage are defined as in Table 2.1.

In addition to this, liquefaction occurrence is affected by the relative density and confining pressure or initial effective stress of soil, because those cause to increment in pore water pressure which triggers liquefaction.

In briefly, clean sand having uniform gradation with rounded soil particles, very loose-loose density state, recently deposited with no cementation between soil grains, no prior preloading or seismic shaking events and locations close to the epicenter of fault rupture of a major earthquake carries liquefaction risk as mostly [7].

Table 2.1. Primary characteristics of liquefiable soil [7].

PROPERTIES	FEATURES OF LIQUEFIABLE SOIL
Types	Clean Sand (Non-Plastic , PI = 0) or Sand With Low-Plastic Fines
Gradation	Medium to Fine Sizes (For Non-Plastic Clean Sand) or Uniformly Graded (For Non-Plastic Clean Sand) or Percent Finer at 0.005 mm < 15 % (Sand with Low Plastic Fines)
Relative Density (%)	Loose Sand (D_r % : 15-35) or Very Loose Sand (D_r % : 0-15)
Permeability	Very Low
Shape	Rounded Same Size Particle
Cohesion	c = 0 or c is not much (Low -Cohesion Soil)
Deposition	Newly Deposited (Lacustrine, Alluvial, Marine Deposit)
Depth	Under Groundwater Table or Near to ground surface
Pre-condition	No Prior (Past) Loading or No Seismic Shaking Before
Consistency Limit	LL < 35% ; LI < 0.75 ; Finer % < 15% ; W > 0.9 LL or LL < 30% ; LI < 1.0 ; Finer % < 10 % ; W > LL

2.4.2. Conditions for Liquefaction Susceptibility

According to field and laboratory evaluations and examinations, liquefaction features of cohesionless soils are influenced by several factors.

2.4.2.1. Grain-Size Distribution and Soil Types. In soils where the resistance depends only on particle's friction, liquefaction susceptibility is the highest. In case of similar shape of particles, uniformity constant and relative density, the frictional strength is reduced as the size of particles gets smaller. According to efforts made in the alluvial and diluvial soils regarding effect of grain size on the liquefaction susceptibility, there are outcomes separating the size range of liquefiable soils from the one of non-liquefiable soils. Based on this, a soil with a gradation between two inner lines is most liquefiable such as sands and silty sands, whereas a soil with agradation ranging between the outer to inner lines is less susceptible to liquefaction. If gravel amount increases, resistance becomes greater during shearing due to quicker dissipation of excess pore pressure compared to sands, but from past events, there was the occurrence of liquefaction on the gravelly soils confined by impermeable stratum under critical ground vibration. Also, graphs state that increasing fines content in sand diminishes the compression behavior during vibrational shearing. Nevertheless, the fines content provides cementation and cohesion within sand, so liquefaction possibility gets lower. Also, non-plastic, low-plasticity, clay-size or silt-size soils may have large liquefaction possibility like sands, but the degree of pore pressure development diminishes, while plasticity in those soils is enhanced. Nonetheless,plasticity indices must be higher than 5 to have adequate resistance against liquefaction. Even if clay sediments are not classified as liquefiable soil, significant slide motions due to transformation of wide mass of clay into a severe fluid during seismic actions are observed. Apart from this, powerful vibration destroys the resistance of quick clay mainly. For other, several degrees of reduction in the strength are revealed which also causes to slope failures. Generally, for clay sediments including silt or sand content in slope regions, landslides are started by the initiation of liquefaction within silt or sand fraction before clay content, because the strain needed to liquefy sands is much lower than the strain needed to accomplish the peak strength of cohesive soils. Uniformly graded soils with a coefficient of uniformity lower than 5 are most likely to meet liquefaction compared to well-graded soils. In the uniformly graded soils, fine sand is the maximum risky type with respect to coarse sands, gravels, silts and clays.

2.4.2.2. Relative Density (D_r). In the light of information from previous laboratory and field case results, relative density or natural void ratio of soil is very significant for liquefaction initiation. Soils such as saturated clean sands and silty sands with the relative density under 50% are subjected to liquefaction mostly. Further, liquefaction occurrence is not considered if relative density is more than 75%. Concerning this issue, liquefaction within dense soil is less likely to happen, because dense sand produces the negative pore water pressures during seismic or static shearing due to dilation behavior and thus, the resistance against shear stress is raised.

2.4.2.3. Earthquake Loading Characteristics. The weakness of all cohesionless soil regarding liquefaction during an seismic action is based on the cycles number of stresses or strains by earthquake vibration, magnitude, the intensity, major frequency and period of time of ground vibration.

2.4.2.4. Vertical Effective Stress and Over consolidation. The rise in effective vertical stress greatens the shear resistance and bearing capacity of soil and hence, this diminishes to liquefaction susceptibility of soil. As the results from previous observations, saturated sand at the range of 15-18 m depth is not liquefiable. Similarly, there is not any liquefaction in saturated sandy soils having effective vertical stress more than 190 kPa. Also, liquefaction occurrence is diminished when over consolidation ratio increases, because it causes lateral earth pressure to become greater at rest condition and by this way, shear stress needed for liquefaction occasion is raised.

2.4.2.5. Age and Origin of the Soils. The soil grains having alluvial and fluvial based depositshas loosely packed structure which are new, trembling and out of additional resistance because of aging and cementation. Alluvial sediments earlier than Pleistocene era are impossible to liquefy excluding in case of critical earthquake vibrations, whereas deposits from late Holocene era are the most probable to liquefy. Also, earlier Holocene deposits are somewhat liquefiable.

2.4.2.6. Seismic Strain History. According to former reports, previous seismic strain experience has considerable influence on the shear strength of soil against liquefaction. The degree of seismic subjection of soil may limit the excess pore water development, thereby enhances soil strength during next seismic action. The reason to increase in liquefaction resistance of soil comes from the homogenous compaction of the soil or from superior engagement of the grains in the natural arrangement because of removal of low domestic weakness at the connections without taking place any difference within the building. On the other hand, when a soil is subjected to prior liquefaction at full state, there may huge pore pressure build up, leading to greater strains and instability areas within soil because of irregular compression and redispersal of water content. Under this condition, the pore pressure is easily built up during next vibrations.

2.4.2.7. Degree of Saturation. In dry soils, there is no liquefaction occurrence, but a few settlements may be observed due to compaction during vibrating of the soil. Similarly, the possibility of liquefaction on partially saturated sands is not well-known. On the other hand, when the saturation level of soil is increased, liquefaction durability gets lower. Even, sand at low saturation level could liquefy under critical and extended period of earthquake shock.

2.4.2.8. Thickness of Sand Layer. The thickness of sand layer to cause the breakage within surface as sand boils or fracturing as large damage due to liquefaction should be adequate for required lift-up pressure and mass of water extracted from liquefied zone. When the liquefiable sand stratum is thin and embedded within a soil profile, the impact of liquefactionat deep into the surface is prohibited in the existence of non-liquefied surface layer. The necessary thickness of non-liquefied surface layers to prevent the destruction within soil was studied based on three case experiences, but this is not suitable for designing of structure, because there are some settlements, not appropriate for settlement-susceptible constructions, even though the thickness of non- liquefied surface layer is more than proposed value. Nevertheless, it must be counted as introductory proposals for this matter [6].

2.5. Evaluation of Liquefaction Potential

The triggering of liquefaction initiation is associated with obtaining shear resistance of soil before liquefaction and determination of resistance loss after liquefaction.

Liquefaction flow failure occurs when a loose, saturated soil loses most of its shear resistance, due to undrained monotonic loading, and flows like a liquid, because the shear stresses acting on it turns to be equal to soil's shear strength which is reduced in liquefied state. Based on this, liquefaction flow failure is only possible when the steady state strength of the soil is less than the driving shear stress [4]. In other saying, the possibility of flow liquefaction is related with the strain softening during undrained shearing since flow liquefaction causes to a strain-softening and reduction ofstrength considered as soil response against liquefaction.

According to earlier studies, very loose sands and very sensitive low–PI clays may be subjected to sudden reduction of resistance at low shear strains, having very limited undrained shear resistance. Even though several native high-PI clays can undergo some strength reduction, at greater shear strains the progressive resistance reduction is caused due to their ductile properties.

For the cohesive soils with 5% or more shear strains at the point of peak deviatoric stress flow liquefaction occurrence is possibly low, even if there is a progressive reduction in the strength after this peak. At this point, the presence of very loose coarse grain soils has crucial role for flow liquefaction occurrence. Therefore, sandy silts, silts, silty clays, clayey silts and clays could experience to flow liquefaction except very loose clean sands.

Previous evaluations are derived from the concept of both a critical void ratio and steady-state condition which gives the liquefied resistance and reduction within shear strength of soil. According to these concept, void ratio and effective stress affect the state of soil.

Based on this,the state parameter is obtained by the subtracting critical state void ratio from natural void ratioof soil under constant effective stress. As an instance, defining a state parameter around 0.05, if a soil in the steady-state compared to the critical state has more denser arrangement, dilation happens and thus, the behavior of strain-hardening situation exist during undrained shearing.

However, a soil in the steady-state with respect to the critical state has more looser configuration, contraction occurs, so strain-softening may be revealed in case of undrained loading condition. Even, loose soil can undergo strain-softening during undrained shearing regardless of type of state. Furthermore, to observe flow liquefaction potential of soil, defininga state parameter around 0.05 gives positive results as a screening method.

According to numerous studies, under undrained shearing, the deviatoric stress of loose soil reaches to the peak at very low strain and reduce considerably in post-peak region until ultimate state is attained at greater strains in axisymmetric condition. When steady state becomes critical state, ultimate state is reached.

Based on the graph including deviatoric stress and axial strain during undrained shearing,line arrives to a peak and afterwards, it becomes constant which is specified as critical state line. Therefore, an axial strain increases as deviatoric stress is kept steady within this region where soil is under critical state [8].

In one of past study, under specific shearing in drained condition, the volumetric deformation of sand depends on confining pressure and relative density. Moreover, critical voidratio line as a boundary state curve was specified. During shearing, soil above the line shows contractive behavior, whereas soil below the line presents dilative behavior. From similar research conducting static and cyclic undrained triaxial test and analyzing the effect of confining pressure and relative density of soil, in spite of such that steady-state similar to critical void ratio line was defined, it was obtained that flow liquefaction may occur only on the sand above steady-state line. In contrast, cyclic liquefaction or cyclic mobility can happen in the soil either above or below this line.

In general, it is assumed that the soil above critical state line is very loose sand which are subjected flow liquefaction during undrained shearing leading to large settlements, whereas the soils below critical line are medium dense sands which are subjected to cyclic liquefaction during undrained shearing causing to lateral spreading or ground oscillation.

For liquefaction occurrence, a liquefiable soil should be subjected to strong shaking and during analysis, the features of ground movement induced by earthquake need to be considered. To identify the liquefaction initiation, the relationship between the value of effective stress within soil under cyclic loading and the boundary of flow liquefaction occurrence may be used. Therefore, increase in pore pressure under cyclic shearing would be the indication of liquefaction initiation.

Also, sands under drained dynamic shearing undergoes to volumetric change, whereas under undrained condition, only excess pore pressure is developed. From previous observations, the main reason is due to shear strain rather than cyclic shear stress. At this point, there is a logical connection between liquefaction occurrence and cyclic shear strains. However, the primary obstacle is to determine this shear strain during an earthquake. Finally, the liquefaction potential was calculated by connecting the relationship between increment in porewater pressure, number of cycles, cyclic shear stress and strains [7].

Based on the experiences from actual events on before, loose, newly deposited, non-plastic or low-plastic soils compared to denser, plastic and previously deposited has more potential to undergo quick strength reduction.

Certain type of plastic clays is generally more ductile than non-plastic or low-plastic, very loose soils, even if these meet with considerable strength reduction during undrained shearing. Therefore, those soils attain the peak and residual shear strength at great shear strains. Moreover, sensitive fine-grained soils having low plastic limit and peak undrained shear strength at lower axial strains may undergo flow failures [8].

2.6. Causes

2.6.1. Effects of Liquefaction on Soil

At the beginning of liquefaction initiation, loose sands are subjected to huge deformations or flowing situation with no resistance against deformation. Buildings above liquefied soil mass have considerable settlements and tilting. Despite this, water goes up to ground level initiating sand boiling and pipelines or granary type structures float in the surface.

On the other hand, for dense sand, too many iterations of cyclic stresses or cyclic stresses at much more intense level is needed to initiate the liquefaction, but soil reacts against this situation as decrease in pore water pressure and increase in shear strength because of the dilation characteristics. During the evolution of this durability, there are some settlements within the soilmass.

The degree of the settlements for stable state increases with the continuity of seismic loading, but total settlement level does not exceed the permissible limit. Therefore, the liquefaction in the dense sands is called as initial liquefaction with confined strain potential or cyclic mobility.

Liquefaction causes the four different failure types within soils. These are respectively flow failure, bearing capacity loss, ground oscillation and lateral spread. Besides, there are also different problems derived from liquefaction such as eruption of water, sand boiling and others. Sand boiling is not soil failure but also the indicator of liquefaction occurrence proving the building up within pore water pressure at certain depth.

2.6.1.1. Flow Failures. Flow failures or landslides, causing most harmful soil failure, happenin inclined regions having 5% or more sloped loose granular soil. In the following of initiation, cohesionless soils lose its resistance and discharges as massive liquid generating huge deformations. When steady-state or residual resistance is mobilized in the soil in following of deformations, it is indication that flow based deformation is begun. Flow failures,therefore, can dislocate massive size of soil for hundreds of meters. Its occurrence may exist in native territory, artificial soil structures as mine-tailing dams and fills behind retaining structures. Generally, entirely liquefied soil or unaffected soil blocks sitting on a liquefied soillayer may undergo the flow failures. For the case history, flow failures was happened in the 1811-1812 New Madrid Earthquake in Missouri and lately, in the Lower San Fernando Dam at 1971 San Fernando, California.

2.6.1.2. Lateral Spreading. It is well-known kind of soil failure launched by liquefaction. Lateral spreading includes basically lateral displacement of wide, unaffected, surface soil masses over the layer subjected to liquefaction. Under the lateral spreading state, there are rotational motions and collapse within soil. It is usually appeared on the gentle slopes having 3 degrees level, disrupted river channel and it causes to the permanent displacements changing from few centimeters to 10 m realized in the U.S, Japan and so on. During the lateral spreading, embankments, pipelines, bridges, foundations of structures, canals and roads was costly affected. For example, in Alaska earthquake, it dislocated 266 bridges and spreads of flood plain deposits around river channels. According to previous investigations, in case of magnitude of earthquakes under 6, the possibility of considerable lateral displacement due to liquefaction reduces and there is no remarkable lateral spreading occurrence for dense to very dense sands under the magnitude of earthquake below 8 as a degree.

2.6.1.3. Ground Oscillation. Ground oscillation is the back-forth movement of soils masses on the liquefied layer caused by liquefaction at depth splitting soil masses above. It generates the vibration in ground through crushing of soil masses. Ground surface layer fissured is shaken at a dissimilar mode than the underlying and adjacent hard surface and it leadsto cracks between oscillating masses and other hard surface. Ground oscillation, in cultivated region, together with sand boiling gives a huge clearance trouble. Moreover, when ground settlement happen with ground oscillation, severe breakdown and huge loss may be observed.

2.6.1.4. Loss of Bearing Capacity. If the initiation of liquefaction happens in sand a few meters under a building, it goes up along the overlying sand layers. By this way, the soil bracing the structure is emasculated so, there is loss of bearing capacity within soil [6].

2.6.2 Effects of Liquefaction on Structure

Based on case studies, it was observed that considerable strength reduction, due to strain softening happens in vital parts of a soil complex during sudden failures without any warning.

From previous studies, triggering to liquefaction could be very insignificant, so for the soils, susceptible to strength reduction, triggering to liquefaction is considered. Any structures upon liquefiable soils must be constructed with caution by taking the strength reduction and great deformations into account [8].

The liquefaction occurrence within the soil has several impacts on the construction and ground surface. These are such as structural devastation and significant damages on the live and property. The primary factor causing those are sorting as bearing capacity reduction, excess permanent horizontal deflection known as lateral spreading, ground mobility at high amplitude,significant vertical soil settlement. Moreover, as the examples of structures, roads, bridges, ports, retaining walls may be damaged.

Regarding the nature, landslides are caused because of the sand boiling. According to statistical outputs, liquefaction is occurred within 62% of 50 devastating earthquakes around theworld. Moreover, 15-30% of total earthquake casualties is came from the liquefaction occurrence. Therefore, those damages as a consequence of liquefaction has important effect onthe economy as much as the level exceeding billion-dollars. Some of the earthquakes were given as an example in below [1-4, 9].

- Wenchuan Earthquake, 2008,
- Niigata Earthquake, 1964,
- Kocaeli Earthquake, 1999,
- Kobe Earthquake, 1995,
- Bhuj Earthquake, 2001,
- Chi-chi Earthquake, 1999.

The risky structures towards liquefaction and the kind of structural failures were summarized as seen in Table 2.2. By this way, classification of liquefaction-based failures within structures and the response of soil were made [6].

Table 2.2. Classification of Liquefaction-Induced Structural Instability [6].

Structures Usually Influenced	**Features of Structural Failure**
Shallow or Deep Structures	Removal of Bearing Capacity in Foundation
Buildings on slope or at the slope bottom	Failure slips on slope
Dam earthworks or understructures	Failure slips on slope
Bridge Columns Rail Lines Roads Utility lines	Mobilization of liquefied soil close to geodetic collapse
Structure on the incline Utility Routes Roads Rail Lines	Spreading laterally on flat surface
Embedded tanks or Utility struts	Extreme floatage of structures due to great pore pressure below ground
Buildings on the certain level	Sink-hole generation by sand boiling
Retaining wall or Coast structures	Rise in horizontal stress in liquefied soil

3. LIQUEFACTION MITIGATION METHODS

3.1. Traditional Methods

The purpose of almost soil improvement methods applied into site is to eliminate the hazards caused by liquefaction. Based on type of soil, economic aspects of treatment, probability of implementation closeness of buildings and service areas, there are several ground improvement techniques appropriate for elimination of ground risks. After improvement, it is expected that engineering parameters like shear strength, drainage properties, compressibility and density are improved. Most of these are listed below [1,5,10].

- Removal and Replacement Techniques
- Densification Techniques
- Reinforcement Techniques
- Grouting and Mixing Techniques
- Drainage Techniques

3.1.1 Densification Methods

Densification is provided by various methods which targets to enhance the density, stiffness, shear strength and durability against liquefaction. Some of extensive densification practices are dynamic compaction, vibro-compaction, compaction grouting and blast densification. Those techniques have strong influence on clean sand and gravels that are compacted by vibrations. By this way, liquefaction during seismic actions may be prevented.

3.1.1.1. Vibro-compaction. Vibro-compaction is carried out by the top of vibration probe during withdrawal. Densification happens solely in a slight distance around probe, so repetition is vitalat constant spacing about 5 to 10 ft depending upon dimensions of probe and the type of soil. Also, this technique is very appropriate for clean, loose cohesionless soils having silt content less than 15% and clay fraction less than 3%. For example, greater and more strong vibrators have been recently promoted with greater spacing and deeper drilling range.

3.1.1.2. Vibroflotation. Vibroflotation technique densifies loose sandy soils by placing a vibroflot vibrating horizontally the soil through spouting out water from the tip of vibroflot in the ground. Therefore, the soil around the vibroflot is compacted with the vibrations led to collapse in soil texture [5]. Mechanical placement process, instruments and soil categories are important parameters for vibro-flotation [10].

3.1.1.3. Vibroreplacement. When vibro-flotation is implemented with a gravel backfill, stone columns are established because of this. By this way, an extent of bracing is supplied except compaction level.

3.1.1.4. Dynamic Compaction. In this improvement practice, cohesionless soils are densified by permitting heavy weight to fall from specified elevations. Liquefaction may locally happen below the drop position. Hence, the sand particles become denser after the dissipation of excesspore water pressure caused by the dynamic loading [5]. Moreover, the amount of heavy weight is varied from 9,000 to 27,000 kg. Several kinds of granular soils can be improved by this method. Also, when working pad of dry porous soil is utilized on the surface, it is remarkable up to 10 m. However, even though dynamic compaction is cost-efficient ground treatment method, it is not advisable at the depth more than 10 m or at the depth under the available buildings. Prior to any construction, it may be beneficial and out of dissipation, main mechanismto make the soil is the reduction of pore spaces as the effective stress upon liquefiable layer increases [1].

3.1.1.5. Blast Densification. It uses explosion occurred within prior bored or jetted excavations to devastate the prior soil system and retrieve grains as being firmer. Intensity of explosion maybe big enough until the level where adjacent buildings are not damaged by intense vibrations. According to Solymar, soils lying at 40 m depth from ground surface may be compacted by this method. Certification of settlements and current blasting events on sites are needed to be checked to achieve efficient densification. On the contrary, since the unacceptable results such as liquefaction occurrence within the local regions may be possibly happened, the level of utilization from this technique is lower than the one from others [1,10].

3.1.1.6. Compaction Grouting. Problematic soils become dense throughout the implementation of this method where multiple grouting units are discharged into soil under specific pressure led to friction between soil and cement grout. By this way, some deformation exists in a soil, but the plasticity of a soil is emerged thanks to the cement shot. Regarding the problematic soil types, there are poorly compacted fill, alluvium, compressible or collapsible soils where liquefaction risk may exist. Under the favor of compaction grouting, a density of soils is increased with reasonable cost and less deterioration to the building than foundation. This method is beneficial to grade the level of building again. On the other hand, it is hard to check the changes over the soil and this method becomes insignificant for the soil close to surface or adjacent slopes due to insufficient confining pressure. In addition to these, the design specifications are not completed but also a heuristic approach is applied over the sites. Nevertheless, some studies towards ideal grout properties, efficient pumping ratios with soil behavior and injection stress are in progress. Development concerning instrument and controlling, and especially for treatment level via seismic tests were lately provided.

3.1.2 Reinforcement Techniques

Soil treated by reinforcement techniques involves the separated involvement as concrete, timber, steel, and densified gravels. These inclusions are established into the soil to enhance its stiffness and strength. Sand compaction piles, vibro-stone and vibro-concrete columns are the examples of implementation of reinforcement techniques [10].

3.1.2.1. Vibro-stone Columns. Since 1970s, vibro-stone columns have been utilized to mitigatethe liquefaction on liquefiable soils. It is such a gravel column which is installed into the soil by vibro-flotation technique. Regarding the process of vibro-stone columns, vibratory probe is inserted into the soil, which dislocates the ground laterally by the vibration and thus, it leads to compaction around the mass of soil. In some cases, steel casing driven into ground and drop hammer may be utilized before the gravel is poured from the top to bottom with a falling hammer. Afterwards, the steel casing is receded. Also, the gap is completed with stone later. Ensuing column and wrapping soil ensure the greater strength and stiffness. Furthermore, liquefaction impacts are decreased, because the stone columns generate a drainage line for the dissipation of excess pore water pressure. However, it is not economical for single deep soil layer [1,5,10].

3.1.2.2. Sand Compaction Pile. For the depths in the range 3-15 m, clean sands become backfill matters. It can be caried out in a wet or dry way. In terms of convenience, wet method is very suitable for clean granular sands, because the presence of water jets supports entrance, reduces strength and provides effective compaction. However, this method is expensive if implementation is within deep sole liquefiable strata. As the reason, all depth of penetration must be advanced. Based on factors such as vibrator kind and dimension, gap of entrance, soil category and compaction conduct, the compaction efficiency of sand compaction piles is determined.

3.1.2.3. Vibro-Concrete Columns. Rather than sand and stone, concrete, in here, is utilized. For the process, the concrete is driven into the soil through the vibrator. In following of extracting the vibrator, concrete is delivered into the gap which forms a concrete column. It aids to conveythe application to deeper underlying appropriate layer. Meanwhile, a concrete column densifiesthe nearby soil strata [1].

3.1.2.4. Compaction Piles. Compaction piles compacts the soil by substituting while the piles are driven into the soil into the layers. For this intention, timber, or prestressed concrete as the type of material are driven in a grillage model and stayed on there. Moreover, compaction piles enhance the vibrative behavior of the soil sediment through three unique ways. Initially, flexural strength of each piles adds the endurance or support to the treated soil. Also, shaking and translations generated by their placement lead to compaction. To sum up, establishment activity expands the lateral stresses in the soil towards the piles. The spacing of piles are carefully determined for the efficiency, since only the small region of the soils around piles are compacted . In addition to these, maximum depth of enhancement is 18 m.

3.1.3 Grouting and Mixing Methods

Grouting and mixing methods enhance the shear strength of the soils by means of injectionof several substances such as chemicals, particulate materials, and resins into the pores. As an example of those methods are jet-grouting, permeation grouting and deep soil mixing [10].

3.1.3.1. Permeation Grouting. Low-viscosity fluid is injected into the voids with small deformation to the soil system, so the soil is densified locally and becomes impermeable. In this way, the danger of liquefaction occurrence and contraction is diminished. A variety of grouts are classified from high-slump cements to several gels with low viscosity, based on basically the void properties of the soil [5,10].

3.1.3.2. Jet Grouting. Jet grouting method has the procedure of replacement or confinement toprevent the liquefaction. A drill rod along with high pressure ports is lowered to highest depth of treatment. Afterwards, water, air and grout under high velocity jets initially deforms the soil and later, mixes the soil during the rotation and withdrawing of drill rod. The consequence is acolumn involving soil and concrete along all designed layers. Moreover, when these columns are combined to each other in such a way that shear in-ground walls is created, the soil is restrained. Therefore, total mass of liquefiable matter inside a designed layer can be strengthen. According to the site characteristics and specific intentions, several jet grouting methods exist as the type of single, double, and triple fluid systems with Super or X-Jet grouting [1].

3.1.3.3. Deep Soil Mixing. It is the technique which enhances the soil properties by blending mechanically the soil with cementitious binder mix by means of a forcible drill which squirts and stirs the binder mix into the soil during process [5]. Deep soil mixing was introduced in Japan during 1970s and there are paddles, rotary mixer mile and jets as equipment which diffuses into the soil during injection of stabilizing substances like Portland Cement. Deep soilmixing is separated into the several types such as soil mix walls, deep cement mixing, deep jet mixing, deep soil mixing, deep mixed method and so on. As a result of implementation of this method, targeted soils or soil layers acquire considerable shear strength and stiffness, lesser hydraulic conductivity, and decreased compressibility. By this way, the ground may have greaterinfluence on supporting the building above in the case of liquefaction happening.

3.1.4. Drainage Methods

It is known that one of the factors' causing liquefaction is the drainage properties of the soil, so liquefaction problem could be diminished by improving drainage characteristics of the soil. The excess pore water pressure under liquefaction condition is lowered in the way that thewater in pores between soil particles could drain easily. Depending on materials, drainage methods have installment of cohesionless soils' drains or synthetic matters' drains. Also, necessary precautions are needed in case of that gravel drains are used. Some settlement may happen after an earthquake take places on the soil improved by drainage methods. In matter of installment procedures, synthetic wick drains are built at several angles, whereas sand or gravel types are established vertically. In addition to these, drainage methods become more influentialon the soil when used with other liquefaction mitigation types [10].

3.1.4.1. Earthquake Drains. For the aim of draining excess pore water together with decrease in overall settlement on liquefiable soil regions, earthquake drains are used which contains 75-100 diameter curled pipe with explicit clefts, coated by strainer texture. The explicit clefts eliminate excess pore pressure as strainer texture blocks the finer particles from plugging the clefts. Moreover, the drains are established by casing, traditional or modified boring methods. Soil conditions and the drain properties has great influence on efficiency of the earthquake drains.

3.1.4.2. Dewatering. This technique is related with the theory of drainage. Therefore, when excess pore pressure is derived by static and seismic actions, liquefaction may occur. By pumping out of water from the soil, liquefaction hazard could be mitigated. On the other hand, this technique is valuable and has the challenges such as long period of time of pumping and some limitation in workplace. Also, even if persistent and constant reinstall demands are supplied, dewatering may not completely mitigate liquefaction [1].

3.1.5. Removal and Replacement Methods

In this method, the soils having liquefaction risk are excavated and recompacted to obtain a suitable fill with a higher density. Hence, liquefaction risk may be degraded. There are some options to have an appropriate fill such as a compacted form of the mixture of natural soils and additives and removal of liquefiable soils with the use of non-liquefiable external materials.

Moreover, removal and replacement could be feasible solution for superficial sediments in terms of economy. Avoiding undesired shallow soils is sometimes the best solution to preventthe damage observed in structures linked to ground without thinking of high costs and hardnessduring construction [10].

However, in case of the close position of groundwater table to surface, this method turns to be inapplicable. In addition to this, out of shallow depths, this method costs too much [5].

3.2. Recently Developed Methods

In general, earlier conventional mitigation techniques are used to remove the liquefaction effect as the primary purpose, but because of some drawbacks regarding their environmental effect, the issue of disrupting to previously constructed buildings as deformation due to shaking effect, volume of the region to be improved. For this reason, some innovations were recently developed as microbial induced treatment, bio cementation and biogas, injection of colloidal silica, bentonite slurry and other chemicals, treatment with geo matters [5].

3.2.1. Partial Saturation Method

This method includes ejection of some quantity of air or gas into the voids of completely saturated liquefiable sands and thus, the soil become resistant against liquefaction either in the way of electrolysis from oxygen and hydrogen gases in the sand as first type or with the occasion of air capturing in voids by entrance and exist of water in saturated sand as second type. Therefore, the saturation level of soil is decreased with increase the pore water compressibility and shear resistance of the soil in order to mitigate the liquefaction. Moreover, it is very economical and useful application for both old and new buildings. Among all laboratory tests, an uniform cyclic simple shear test and a table-shaking test are suitable to show impacts of this type of improvement [1,5].

3.2.2. Short synthetic fibers

The liquefaction mitigation with fibers, natural or synthetic, is one of reinforcement methods. It takes an attention with the usage through randomly distribution into ground and regarding durability, synthetic fibers are better than natural ones. Moreover, the usage of fibers is very economical and environmentally friendly. From one of recent study where impacts of various types, amount, location, and orientation of fibers were analyzed with triaxial compression and direct shear test, the results are such that increment in fiber content provides the soil to be ductile by increasing peak shear strength with rise in axial strain at failure and thus, the loss of shear strength after peak point is limited. The best condition for shear strengthis obtained, when orientation is adjusted to 60^0 angle with respect to shear surface.

In another study about the influence of several types, content, and length of fibers on sand behavior toward liquefaction by conventional triaxial and ring-shear tests, the peak and remolded shear strength is incremented with the increase in fiber percentage, as the natural stiffness and brittleness number decreases.

During undrained shearing, fibers has not significant impact on the loose sand, whereas for medium dense type, development is clear because fibers modify soil’s reaction by turning it from strain softening to strain-hardening. Next, there is an effort related with the mechanical characteristic of sand-fiber geotextile complex under different kind and order of geotextile with various confining pressure and piece of geotextile enhanced bearing capacity, ductility, axial strain at failure and decreased the dilation in the layers of geotextiles. For another research about how randomly distributed fibers alter the strength and shear modulus of loose or medium sand through cyclic triaxial test, it was determined that addition in fiber proportion and length enhances count of cycles for liquefaction, shear modulus and stiffness of sand.

Also, improvement by fibers in medium dense sand is better than the one in loose sand. Inshort, the way to improve the properties of liquefiable soil through fiber is along with filling the void and forming better interlocking arrangement, ensuing enhanced compression characteristics of the sand thanks to fibers. Therefore, advancing the toughness and ductility of soils made by fibers is very important for mitigation of liquefaction.

Furthermore, the strength gaining due to fibers was approved in the way of increase in number of cycles to liquefaction. Fibers also is not influenced by weather. On the other hand, several problems regarding transportation and storage such as gathering and sintering of fibers,adhesion between soil and fiber are needed to be considered.

Lastly, the rupture process, strength in more critical environments and the impact of pre-handling requires advanced effort.

3.2.3. Chemical Grouting

Recent researchs primarily concentrated on the investigation of shear strain amplitude, grouting kind, percentage, strength and treatment ambiance. According to consequences of one of the investigations, it was obtained that solidifying activity and microstructural bonding ability of cement provides a resistance against not only first earthquake, but also following shakings.

In addition to cement stabilization, lime stabilization was proposed later. Moreover, some additives as fly ash, phospho-gypsum develops the features of soil stabilized with either cementor lime.

Next, there is another the investigation concerning the impact of various chemicals, as sodium silicate, acrylate polymer and polyurethane, on dynamic liquefied sand by using cyclic triaxial and resonant column test. From results, it was realized that shear modulus of soil with sodium silicate becomes higher and chemical had minimal impact on damping ratio.

Also, acrylate polymer did not influence the shear modulus, but it raised considerably the damping ratio. In conclusion, shear strain amplitude is incremented as much as varying from 10^{-4} to 1% as well as damping ratio and shear modulus was declined. In addition to this effort, the study consisting indoor trials to obtain the long-term resistance of chemical grouting gave that grouted sand's resistance is preserved under steady groundwater flow without modifying the unconfined compressive strength throughout the treatment of 360 days.

Besides, one of scholars after a series of cyclic triaxial tests, stated that two-cycle phases of partially sand with chemical grouts retards the development of pore water pressure more than one with pure sand [11].

Additionally, there is another example related with chemical grouting as passive site stabilization. In this non disruptive way, soil is treated by stabilizing chemical such as colloidal silica by means of slow injection into the inclined region. Ejection wells are utilized to distributethe substances into the liquefiable soils because of prior groundwater flow. The chemical is not toxic, and it generates the indissoluble gel connecting the soil grains, promoting the strength and distortion durability of the soil [5].

In grand flex mole technique as a recent development among chemical grouting techniques, it supplies an essential support to the soil more effectively and readily under the former building without the necessity of extra vertical shafts such as usual chemical grouting method. Therefore, it may be used on an already advanced area with some obstacles. Besides, this new technique is demonstrated to be feasible for the level deeper than 3 m than traditional permeation grouting [1].

3.2.4. Microfine Cement

It is known that Portland Cement cannot enter to micro-pores of soil due to large grain size. Therefore, microfine or superfine cement is beneficial to be grouted into only fine sand having a permeability constant less than 10^{-2} cm/s, but it provides great resistance and environmentally friendly system. Fluidity and bleed strength are enormous because of small grain size and corruption of environment may be diminished by microfine cement. There were several investigations and experiments concerning the impact of micro-fine cement on the features of sand such as permeability, unconfined compressive strength, and grouting characteristics of micro-fine cement as setting time, stability, and viscosity. Among all studies, one of them used torsional resonant column and bender element tests to observe the variation between ordinary and micro-fine cement for consideration on the dynamic features of soil in terms of confining pressure, shear strain, ratio of water to cement, kind of cement and grain size. This study presented that the proportions between water to micro-fine cement had significant impact on the dynamic behavior of soil, whereas for ordinary cement, the effect of this ratio was nearly insignificant. As a result of this, many scholars focused on this ratio and it was observed that reduction in the percentages of water to cement enhanced viscosity of mix, storage constant,unconfined compressive strength, but decreased the permeability and setting period.

Regarding enduringness of soil with micro-fine cement, there was an investigation using creep test, exhibiting that micro-fine cement addition considerably diminished the axial strain and made creep strain steady over 200 days of treatment. To sum up, the factors as the proportion of water to cement, grain size of soil, cement kind and pore size have significant impact on themicrofine cement grouting.

Despite this, microfine cement supplement enhances the strength and grouting distance and there is no destruction to the ambiance. However, it is expensive, but in terms of powerful reinforcement and advanced permeability made by micro-fine cement, cumulative advantages are economically greater than that of ordinary cement penetration [11].

Another effort was related with liquefiable clean sand and silty sand. After the soils were saturated, microfine cement whose water/cement content is %25 was injected into saturated soils. Hence, at various consolidation pressure from 100 to 300 kPa, samples were sheared in away of monotonic loading with a strain-controlled way. As a result of these, the effect of undrained monotonic loading and grouting on flow failure from liquefaction were examined. The peak and residual strength of grouted soils considerably enhanced with stronger deformation capability.

While silt content increases, residual strength decreases. Thus, highest undrained shear strength was obtained on the grouted clean sand. As a result of these, soil characteristics were improved by this cement grouting method. It was less complex and feasible to limit liquefactionhazards. Moreover, the soils under prior structures were improved with this way. This study showed that microfine cement increased the peak shear strength of liquefiable soils by reducingthe strain at failure [4].

3.2.5. Colloidal Silica

It is an aqueous solution having silica nanoparticles derived from saturated silica acid liquid. Its size is usually varying between 2 and 100 nm. To prevent gelation, CS blends are mixed with alkali solution, which makes the nanoparticles ionized for repelling each other. Gelation, depending on the interplay between molecules, softens these repulsive forces, so consistent structure of siloxane (Si-O-Si) connection which cement the soil grains together andinhibits the pore water is generated.

As primary aspect affecting the conduction of colloidal silica in liquefiable sand, viscosity of the colloidal silica, hydraulic gradient and conductivity ofliquefied sand have significant role. Also, the period of gel production is principally influencedby the content of silica in the mixture, dimension of silica molecules, pH, ionic resistance, and temperature condition.

Based on observational efforts, it was obtained that CS has several benefits sorted as a little initial viscosity, containable duration of gelling, well long-term mechanical durability, andminimum damage to structures because of lower particle size and outsize charge of the silica molecules. Actions and structural pattern of particles is directed by the electrical force betweenparticles, so chemical bonding proceeds in the following that gel phase is attained. Therefore, progressing bonding leads an increment in the strength of gel by the time.

Further, CS is non-toxic, biologically inactive, and the strength properties are remarkable. Thus, CS molecular gelin a little content may mitigate the liquefaction of sand by linking the particles to each other andstabilizing the pore fluid. According to previous testing, it was indicated that the resistance of improved sand raised with the increment in CS content. If the period of gel is necessarily prolonged, rise in shear resistance can be kept. Even though CS additives, depending on CS content, let unconfined compressive resistance of improved sand enhanced, equal to 400 kPa, permeability of cured sand diminished throughout growth in CS content in an approximate log-linear approach lowered to $2x10^{-9}$ at least. This means that CS cannot be feasible in a wide area site with homogenous arrangement.

For sufficient and efficient convey of CS in realistic operations, it was founded that with holding CS content at 5% under pH value between 7.5 and 8.7, the duration of gelling may be retarded, so CS can be efficiently and homogenously injected into liquefiable sand. Therefore, resistance to liquefaction was reached to demanded level.

On the contrary, it is hard to control sintering behavior of CS, due to greater density than water, before delivered into desired position. Researchers utilized a pilot-scale pattern with 3D fluid models to estimate the percentage of penetration needed for sufficient constant transfer. As a result of tests, it was realized that CS, in the percentage of 1% by weight, can decrease the liquefaction possibility in loose sands by connecting particles into each other without diminishing the percentage over the lower boundary of 1% in the pore water. At the same time,incrementing the penetration speed may create required horizontal compression to sufficiently transfer the suitable content of dilute rarified CS for liquefaction.

In any case, to accomplish desired vertical injection distance, the speed of penetration should be low. As an event, scholars investigated the influence of various CS portions by using resonant column and centrifuge pattern test. Thus, it was stated that ground deformations of lateral spreading and settlement were gradually diminished with rise in CS portion and the soil treated with CS showed larger cyclic strength values and smaller cyclic shear strains. The modulus of shearing was raised throughout an incrementing CS percentage and damping ratio was not considerably affected. Additionally, from the output of another study, improved sand with CS at the different contents, sorted as 5%, 10%, 15%, 20% respectively, can be resisted to more cycles with even small strain during shearing.

Increase in relative density raised cycle count to start liquefaction and loose sand with 20% CS showed the behavior like dense sand in terms of cyclic resistance. The advantages of CS improvement on loose sand are obtained from the lines of pore pressure generation and the degree of increment in cyclic resistance [11].

Next study about improvement of liquefiable sand with various D_r values sorted as 40%, 60% and 80% by gel method involved colloidal silica gel in the ratio of water to cement as 40%. Soils are saturated and colloidal silica gel is injected into soils. Then, improved soils were waited for 7 and 28 days at 20 ºC under 95% humidity. Solidified soils inserted triaxial machine7 or 28 days later were not saturated due to the structural formation, so these were consolidated at 100 or 300 kPa immediately before shearing phase under undrained condition. As a result, when relative density of a soil became greater, the possibility of static liquefaction occurrence declined, and this study showed that the sands having 60% or more relative density did not encounter the liquefaction event due to their dilative behavior against compaction. However, limited liquefaction, where dilation involved the process, happened for the sand with 40% as relative density percentage.

As mentioned earlier, increase on confining pressure and relative density prevented the liquefaction probability. The samples cured at 7-days gave the more than 4 times of the strength of clean sand, because colloidal silica gels filled almost completely the voids between the soil particles. Additionally, the samples waited for 28-days demonstrated more resistance into compaction, but the difference between 7-days and 28-days was not considerable.

Moreover, the colloidal silica gel affected to the strength increment of the 40%-D_r sand more than others. If an excess pore water pressure decreased, deviatoric stress increased. Then, deviatoric stress reached to the point where saturated soil under undrained condition moved to opposite direction of contraction. Lastly, curing times, concentration of colloidal silica, time of gelation, injection types as extraction wells and low-head injection, injection pressure, pH level, acid ratio in the mixture influenced the deformation and strength characteristics of the soil by generating the cohesion in grains [9].

3.2.6. Bentonite

It is an aluminum phyllosilicate, and a clay including mass of montmorillonite. Bentonite, because of thixotropic features, can improve the loose sand properties. Additionally, it is easily handled, cheap and non-toxic. However, in terms of rheological behavior, it is readily impressed by clay content, pH, water ionic durability, kind of anion and cation. If these are altered, distribution of bentonite may be performed in distinctive form such as sol-form, repellent gel, attracting gel and precipitate. Therefore, depending on the state, bentonite interacts with pore fluid and soil pores is filled with bentonite gel, based on the rheological behavior of pore fluid. Hence, during seismic shaking, soil particles move partially since development of bentonite gel limit this movement. By this way, bentonite improves the sand properties efficiently.

Also, the permeability characteristic of sand modifies the diffusion of bentonite having initial yield stress and viscosity, because the period of gelling in bentonite water mix is very short. As a result of non-homogenous movement of bentonite gel within soil, there is reduction in mechanical properties of improved sand. In the aim of advancing the rheological properties of bentonite, scholars added sodium pyrophosphate varying from 0% to 2% of the weight of bentonite, so it was realized that the rheological behavior of bentonite mixture can be considerably enhanced.

When bentonite with sodium pyrophosphate more than 5% was added to sand, injection of additives into sand was made more effectively, resulting the improvement of sand into liquefaction by the time, except enhancement within cohesion and friction angle. From another study concerning the impacts of various percentages of bentonite and treatment period on liquefiable sand by using static, dynamic triaxial and resonant column tests, it was obtained that sand with bentonite can be 10 times more durable than pure sand to liquefaction in terms of cycle count, as bentonite gel restricts the action of soil particles, enhances the elastic characteristics of sand, and retards the excess pore water pressure development during cycles of shearing.

To evaluate the effectiveness of bentonite, some parameters are specified such as storage modulus, real modulus, loss modulus, but there are some difficulties to calculate these due to complex, non-linear, and anisotropic properties of used substances.

3.2.7. Other Additives

Several studies were made to find the other additives, out of cement or lime, which make soil denser and more compact. The purpose is to support sustainability through restrict environmental contamination during the application of huge quantity of cement or lime. As an initial, fly ash may be option which contains silica and alumina in the porous honeycomb structure. Specific surface, water absorption of fly ash is high which has changing particle sizebetween 0.5 to 300 nanometers.

The liquefaction durability of sand would be developed through incrementing Fly ash, asa filler. In the light of corresponding research, the utilization of fly ash provides considerable strength gaining against liquefaction for sand and the development in durability was raised by the extension of treatment period. However, rise in confining pressure decreased the liquefactionresistance of sand-fly ash complex in terms of cycle count, as seen in Table 3.1 and 3.2.

Table 3.1. Influence of relative density on liquefaction resistance of clean sand and sandwith %2 Fly-Ash under 50 kPa confining pressure and 0.2 CSR and 1 Hz frequency [11].

No.	**Specimen**	**Confining Pressure (kPa)**	**Relative Density, D_r (%)**	**Number of cycles to liquefaction, N_L**
1	Untreated soil	50	20	175
2			40	275
3			60	300
4			80	325
5	Sand + 2% FA	50	20	290
6			40	350
7			60	375
8			80	450

Table 3.2. A brief of the outputs related with the impact of the 4% and 6% Fly Ash content on liquefaction resistance of sand with 20% Dr and under 50,70 and 90 kPa confining stress [11].

No.	**Specimen**	**Confining Pressure,CP (kPa)**	**Relative Density, D_r (%)**	**Number of cycles to liquefaction, N_L**
1	Sand + 6% FA	20	50	350
2			70	325
3			90	250
4	Sand + 6% FA	20	50	375
5			70	350
6			90	275

In the following study, the significance of zeolite-cement binders on liquefaction behavior of sand through unconfined compressive strength test was examined. As the composition, zeolite includes silica and alumina, as identical to pozzolanic matters. The replacement of zeolite with a portion of cement can raise the unconfined compressive strength of the specimen. In case of limited percentage of cement, the increment in zeolite may greaten the unconfined compressive strength of the mixture.

Additionally, one of scholar made an experiment to observe the effectiveness of graphene oxide nano-sheet, having high specific surface area and surface ions with fine pores) on the silty soil's resisting ability by mechanical testing. Thus, outcomes were such that cohesion and angleof friction of mixture progressively augmented as the graphene content and treatment period were extended. In addition, the space between particles of sample were filled by the structure arised from connection between C-S-H gel and graphene oxide nano-sheets [11].

In one of studies enhancing the liquefaction resistance of liquefiable soils, consolidated undrained compression triaxial tests were statically carried out on both very loose clean sand and the sand mixed with slag and bentonite. There were three confining pressures sorting from 100 to 200 kPa used in testing and samples were prepared by the moist-tamping or dry mixing method. The performance of the soil against static liquefaction was examined. Therefore, the sample containing 4% Slag as percentage of soil dry weight has shown the highest deviatoric stress with lowest excess pore water pressure and thus, complete static liquefaction on the clean sand turned to non-flow response. Hence, slag improved sand soil against liquefaction by covering pores between grains and diminishing the pore water pressure. On the other hand, low content of bentonite increased the pore water pressure with reference to the clean sand, so soil structures were impaired with the loss of link between particles. Regarding both usage of them, non-flow liquefaction results existed and among this group, sample containing 3% bentonite and 4% Slag showed the highest deviatoric stress and the lowest excess pore water pressure. Additionally, either increase in confining pressure or relative density of the soil enhanced soil's liquefaction resistance because the contact of grains with more stable arrangement wereadvanced and excess pore water pressure were reduced [12].

Despite these chemicals, further agent, known as laponite, was used to improve the mechanical properties of the soil. Laponite agent was mixed with silty sand in the dry form andthen, the amount of water as much as natural water content of the soil is added to mix. Next, mix was poured to split mold for moderate compaction. From the rheological test and constant rate ramp test, samples containing various concentration of laponite starting from %2 to 3.5% as the ratio of laponite dry weight to water weight were treated for 2,4,6 days. Again, cured samples were put back into triaxial machine and subjected to saturation, consolidation and shearing. The type of triaxial compression test in this study was dynamic or cyclic, so additional components such as frequency, sinusoidal waves, cyclic shear resistance and number of cycles existed. As a consequence, in spite of thinning behavior where great strain in laponite gel was observed, laponite continued its elastic properties when turning into gel.

Even though laponite gel delayed excess porewater pressure initiation and limited the liquefaction development, it did not totally occupy the voids of the soil. Therefore, uniform distribution of laponite over the pores was prevented and thus, free water imparting to measured pore water pressure may have been generated. In addition to this, viscosity of gel increased by the time, so shear strength of complex rised thanks to cementation of soil particles. Also, extending the curing time was beneficial for the shear resistance of improved soil and it was advised to have laponite injectedinto soil in the suspension form with nano-particular size. For this purpose, radial injection well and central extraction well may have been useful for the context of the permeation of the laponite suspension into the soil. Lastly, chemical properties and composition of pore water affected chemical stability and rheological action of laponite. Therefore, liquefaction risk may have been prevented by the laponite as the chemical in terms of providing solidification of pore water, soilgrain cementation and retardation of pore water pressure development. As the note, grouting permeation method was very sensitive and difficult to control and dry mixing cause cavities and free water. That’s why mixing under specific water content and cured in outside condition could have given better result. Extra, the laponite gel mixed with soil did not turn to solid state instantly, so it stayed at gel formation 6 days later and by this way, soil with laponite can have been saturated. The rheological property of the laponite-silty sand mixture did not give stiff cement grout columns. However, it could have seen in the study about colloidal silica gel whichwas cemented after 7 days. The suspension form of laponite transformed into gel, so it gained elastic tendency improving silty-sand resistance [2].

One of research aimed to observe the action of nano soil, pulverized from sandy silt by high energy machine, with the percentages of 2%, 3%, 4% of dry weight of sandy silt on the mechanical properties under 1 days treatment at 25 ^{0}C. For testing, respectively liquid-plastic limit test, unconfined compressive strength test was done based on the specification of British Standard and CD triaxial compression test was conducted under confining pressure at 50,100 and 150 kPa separately. As a result, the addition of nano-soil slightly raised to LL and PL value,but Plasticity Index was lowered, which defines that the plasticity in soil was diminished.

For compressive strength, nano-soil content increment enhanced the Unconfined CompressiveStrength, changing between 3% to 22% with varying nano-soil percentages. Thus, it was provedthat small amount of nano-soil augmented the compressive properties of soil because of an effective interaction between soil and nanoparticles, ensuing strong connections between soil particles. According to CD Triaxial Test, nano-soil gave an increment in the range of 7%-17% depending on nano-soil percentage. As nano-soil content was incremented, cohesion of soil, in contrast to angle of friction, was decreased. At the end, sandy silt soil gained important reinforcement through its nano-size form in terms of geotechnical parameters. Like sandy silt, soft sandy clay soil with intermediate plasticity could have been improved with nano-size of sandy silt soil based on the outputs mentioned above [13].

3.2.8. Biological Matters

In general, for all biological grouting techniques there is utilization of the micro-organisms to improve soil in the way of biogas producing small gas bubbles improving the sand against liquefaction with decreasing permeability, bio-cementation as a way where shear resistance of soil is increased thanks to the materials binding the grains and lastly, bio-clogging in which pores of a soil are filled with substance to lower porosity and hydraulic conductivity. Biological grouting in opposite to chemical grouting is very feasible and anenvironmentally friendly. Despite, it is practiced on specific type of soils and environments [5].

As being environmentally friendly method, biological based improvement of soil takes significant place. Sandy soil boosted with biological matters have recently become very trending topic and presented a new option as a novel discovery for liquefaction mitigation. The primary process of this method includes the microorganisms, nourishments, biologically induced calcite precipitation through urea and calcium.

It is aimed to connect the grains of sand, known as bio-cementation. Major elements disturbing calcium carbonate precipitation are the quantity of microorganisms, nourishment amount, the feature of porous ambiance and pH number. Microorganisms spontaneously operate several biological reactions where urease reacted with water to increment pH number to precipitate the calcium carbonate. Moreover, the negative charged microorganism surface created to nucleation on the cell surface. It was checked that MICP may be efficient, non-pollutive and non-disrupt to ambiance, but during conveyance of the microorganisms, the permeability of soils was influenced. Hence, undesired delivery of microorganism would be resulted. According to relevance effort, increment in microbial amountgenerated the enduring precipitation of calcite and lowered the permeability of soil.

In the next one, under dynamic triaxial testing and its cyclic condition, the dynamic effect from microorganisms greatened liquefaction strength, stiffness of sand. Even, through the microorganisms, pore water pressure development and the foundation settlement were decreased.

In addition to MICP, a recent exploration, enzymatically induced calcite precipitation (EICP), was made to improve liquefiable sand. Here, urease is utilized as the activator for urea hydrolysis, rather than bacteria. By this way, both calcium and carbonates ions generate calcium carbonate precipitation. Last studies resulted that EICP implementation can produce more calcite precipitates under lower saturation level than the one under higher saturation level. Horizontal or vertical deformation of soil caused by liquefaction were efficiently decreased, even though there were an amplification from quickening of the earthquake actions when biological matters were used in the soil. Nevertheless, out of being nontoxic, some adjustments about both strength augmentation and decline in acceleration effect of earthquake at surface caused by the utilization of biological materials require further analysis [11].

3.3. Control

3.3.1. Feasibility Study for Traditional Methods

Soil improvement techniques generals aims to decrease or remove the liquefaction impactsby enhancing the low strength, stiffness and drainage features thanks to trial and error. Majority of these methods were generated more and more by the time. The most efficient types used to mitigate liquefaction are removal and replacement, densification, reinforcement, grouting and mixing and drainage methods. In despite of extensively utilization, under certain conditions the conventional mitigation techniques have restrictions such as environmental effect, disruption toformer buildings under shaking and deformations, dimension of the region to be improved etc. Therefore, practicability and compatibility over real site cases depend on the specific conditions [5,10].

For reinforcement method, it presents additions in certain spacing to brace and rigidify a soil sediment. On the other hand, great stiffness and resistance of the additions lead to stress reduction in the soil mass between the additions, so the soil gets weaker. In removal and replacement method, as an earliest, easiest and safest way for eliminationof liquefaction in ground, cost may be high when the mass needed to be improved becomes larger. Moreover, it is not applicable for the depth below ground water table level.

Concerning densification of ground, as the most generally utilized method against liquefaction, it benefits from the powerful vibrations to compact the ground. Densification is very efficient especially for sandy soils. However, powerful shaking may cause disruption to nearby buildings, pipelines and other facilities, so it is not appropriate to be implemented near to constructed buildings. In one of densification methods involving all vibratory techniques except vibro-stone columns and vibro-concrete columns, expense would increase when a deep single layer requires to be improved. Also, for deep compaction, the application under previously constructed structures is very difficult. Moreover, blast densification requires environmental space.

Regarding grouting methods which involves the shot of specific liquid or solutions to enhance the soil, but in some case, Portland Cement may be used or consumed. Even though it makes soil resistance greater, it restricts the deformation characteristics of soil with minimizing the hydraulic conductivity and improves the deep insulated liquefiable layers, the presence of underground construction as pipelines, subway lines or well make this method difficult to be applied. In all examples of grouting methods such as jet grouting, permeation grouting, deep soil mixing and even compaction grouting, the underground facilities may be obstacle to operate this method. In soil treat with drainage method, development of porewater pressure is lowered well during seismic shaking through making the drainage distance smaller within a soil sediment. In contrast, the installment of drains usually requires some densification. Also, the drains increase the bracing degree of soil. For instance, in earthquake drains method, conduction on specific liquefiable soil strata is not practically valid. Also, for dewatering, the cost would be higher if continual delivery of ground water is needed [1,10].

3.3.2. Feasibility Study for Recently Developed Methods

The primary question for all mitigation method is to monitor the sudden development of porewater pressure in the soil. Recent techniques may be separated into three kinds to dominate the pore water pressure as listed in below.

- Decrease the volume of pore content,
- Advance the attachment impact between soil grains,
- Lower the soil saturation.

Recent methods are also divided into two groups to reach these purposes as listed in below.

- Natural and passive drainage,
- Blending of binders with soil.

Best practice depends on the size of soil pores. Meanwhile, these techniques influence the homogenous spreading of binders and cause the disruption to adjacent structures, while pore water pressure decreases along with these procedures. Moreover, the techniques of penetration, curing of nanomaterials, biological advancement, chemical injection, partial saturation about implementation on site are appropriate for available soil, whereas recycled matters and short synthetic fibers are applicable for new earth-structures due to the reason that environmental problems may arise in the following of destruction of former earth structures including these materials.

3.3.2.1. Soil’s Pore Size. The injection of chemicals, microfine cements and other materials with biological based improvement are diffused into soil efficiently. On the contrary, the implementation for fine-grained soils is very hard due to small pore size. In turn, chemical additives and microfine cement are appropriate for the soils with grain size higher than 16 μ. Regarding biological improvement, it is difficult to make penetration into soil having grain size smaller than 0.4 μ due to the existence of biological matters greater than 0.6 μ. In partial saturation method through air diffusion, less permeable strata may reduce the performance. As the limitations of this method, soil needs to be sandy soils having hydraulic conductivity ranging from 10^{-4} to 10^{-3} and operation depth must be between 10 m to 20 m. For short-synthetic fibers,except being appropriate for different pore sizes, based on desired functioning, changing specifications are needed.

3.3.2.2. Influence on Surrounding Buildings. Recently developed liquefaction mitigation techniques may cause to disruption to nearby structures. Primary reason is due to high passive drainage stress. For chemical grouting of chemical or microfine cement, natural drainage is not safe and rather than this, high pressure is needed when the injection occurs within desired depth.In other saying, infiltration is done by the penetration under high pressure. On the other hand, it disturbs the soil and surrounding buildings because of powerful penetration pressure. Even, under high pressure way, homogenous injection of additives is not provided as desired. Among the mitigation methods in above, nanoparticles-water mixture, opposite to conventional techniques, may be injected more uniformly within soil and considerably decrease the disruption to nearby ambiance, because high pressure during diffusion of matter is not needed. For chemical additives, penetration into fine sands or coarse silts under high pressure has great cost and it includes environmental danger as in the example of silicate grout generating large percentages of soda and like-compounds, carrying toxicity. Therefore, in chemical grouting, several restrictions exist because of its expensiveness, permeation, and environmental dangerousness. According to real cases regarding implementation of chemical penetration, thereare very few which is ready under the former structures and analyzing the ground distortion is very hard in the buildings. Although the penetration of chemicals, as expansive polyurethane and resin, below the structures was approved based on previous records from real site projects,some disturbances such as ground heave and raising of the building base. Nevertheless, the disturbing from chemical grout to nearby buildings is lower than that of traditional methods such as stone columns or driven plies. Compressive grouting was selected as the mostapplicable one to densify the foundation base.

3.3.2.3. Cost. In the light of recent studies concerning the techniques to mitigate the liquefaction, the most important aspect for utilization is related with the feasibility level based on cost, dangers to ambiance and durability. On the contrary, it is difficult to predict the cost precisely since all cost is linked with a plenty of considerations, as unit price, concentration, installment process, equipment, efficiency and environmental effect. Thus, evaluation in terms of numbers is very hard to handle. Primary factors are explained in the list as follows.

- Majority of techniques are currently at laboratory phase and not applied into the sites aspractically, so cost can not be simply calculated.
- For the applied techniques in reality, the cost is related with improvement degree of soil, the sedimentation level, Project cost which changes with country or region.
- Other than these, construction obstacles, the duration of consturction, delivery, extracted contaminants makes cost calcuation difficult.

Furthermore, the situation of environment and adjacency of existing building and constructed edifices affect to cost evaluation. As a basic characterization for cost calculation, there is a report about whether the methods are cost-efficient or not based on prior investigations.

- In nanomaterials, the unit price is great but total cost about its functioning or functioning/price rate is greater than that of cement or chemical inejction. Moreover, nano-materials provide great resistance and desired permeability for improved soil. In constrast, disturbance to ambiance and environmental effect are existed, out of being very low.
- For short-synthetic fibers, the cost is competitory with respect to lime,cement and other chemical. The construction procedure is simple and fibers gives feasible supporting and strong endurance capability to the soil. Lastly, fibers are not influenced by weather.
- Concerning micro-fine cements, even if manufacturing cost is more, the fine

cement suspension provides desired permeability and well supportive impact. By this way, total cost is less than that of ordinary grouting cement.

- Regarding biological methods such as biogas or biosolidifcation, both of them are cheap and have less environmental danger with reference to chemical injection, which is generally expensive and hazardous for ambiance. On the other hand, microbial mechanism are complex and only practical for specific soil kinds and the condition offsite.
- For partial saturation by air infusion, greater flow speed could enhance the dispersion of air, but raise the compressor necessity. As mentioned before, the air infusion technique may be advantageous for buildings with very restricted entrance and it would be very economical due to the usage of air instead of chemical grouts.

3.3.2.4. Time of Durability. The previous assessment about the measurement of endurance formany liquefaction mitigation methods are very few, because nearly all of mitigation methods are currently at laboratory phase. In addition to this, monitoring of treated soil under long-term condition for all periods of durability is very hard to apply. Nevertheless, there are several cases giving an information about endurance period for soil. According to summary of these,saturation degree of sand column raised by 82.9-83.9% within two days due to the primary de-saturation after the installment of deep sand column into ground. The partial saturation in sandysoil could be protected for 26 years. Also, collodial silica injection, as one of chemical groutingmethod, is durable for long-term in the ground with progressive groundwater inflow. Also, unconfined compressive strength of chemically treated sand would not reduce for 360 days of treatment. There is another analysis regarding microfine cement through creep test. Eventually, for nanomaterials, bio-matters, chemical additives and micro-fine cement, durability to liquefaction enhances with the extension of treatment period [11].

4. THE FEATURES OF MAGNESIUM OXIDE

4.1. General Properties

4.1.1. Physical and Chemical Properties

Magnesium oxide or magnesia is hygroscopic white solid mineral known as periclase. Empirical formula is specified as MgO whose lattice includes $Mg^{+2}$and O^{-2} ions connected through ionic bonding. In the history, magnesium oxide was accepted as magnesia alba which is magnesium carbonate $MgCO_3$ having literally white mineral and it is different from magnesia negra involving a black mineral known as manganese.

For physical characteristics, color is changing from colorless to black or brown. It also hasclear oxide structure, known as Rock-Salt arrangement. The density of MgO is approximately 3.579 g/cm^3 and hardness is about 5 for Mohs test. Meanwhile, the number of thermal conductivity for rigid formation under 100 ^oC is 36 W/(mK) as being very high. The melting and boiling points are respectively 2800 ^oC and 3600 ^oC. As the level of electrical resistance depending on the chemical purity, it may attain 10^{16} Ω·m for high pure MgO which is very low, whereas magnesium oxide calcined over 2000 ^{o} C has the purity grade whichis not affected by electrical resistivity. Regarding the dielectric constant of MgO is between 3.2 and 9.8 under 25 ^oC temperature with 1 MHz frequency and similarly, dielectric loss under this condition is nearly 10^{-4}.

Furthermore, chemical characteristics and surface arrangement are affected by the calcination condition such as temperature level, ambience as air or vacuum, initiator type. As an example about this, magnesium oxide having defects at surface or numerous pores absorbs the water easily compared to others. In Table 4.1, main indicators of the properties of magnesium oxide are given [14,15].

Table 4.1. The details of general indexes for Magnesium Oxide [15].

Properties	**Values and Features**
Other Names	Periclase or Magnesia
Density	3.58 g/cm^3
Melting Point	2852 ^{0} C
Boiling Point	3600 ^{0} C
Thermal Conductivity	-10.2 x 10^{-6} K^{-1}
Flash Point	Non-ustorious
Crystal Texture	Cubic Halite
Appearance	White Powder
Fragrance	Odorless
Chemical Formula	MgO
Solubility	In Acid and Ammonia
Insolubility	In Alcohol
Geometrical Order	Octachedral (Mg^{+2}) Octahedral (O^{-2})
Heat Capacity	37.2 J/mol K
Lattice Parameter	4.212Å
Molecular Weight	40.305

4.1.2 Production

Magnesium oxide is mainly generated by the calcination of magnesium carbonate $MgCO_3$or magnesium hydroxide $Mg(OH)_2$. Meanwhile, through the curing of magnesium chloride solutions including limewater, magnesium oxide is produced. During calcination process, thereis a heat curing condition and it has influence on the either grain size or specific surface area of product and thus, reactivity is determined at the end.

Hence, based on temperature degree, thetype of magnesium oxide is produced. Classification of those types is listed below.

- Light-burned or caustic calcined MgO is formed at the temperature ranging from 700 ^{0}C to 1000 ^{0}C.
- Hard-Burned MgO having limited reactivity is formed at the temperature between 1000 ^{0}C and 1500 ^{0}C.
- Dead-Burned MgO as unreactive type utilized as refractory is produced at the temperature in the range of 1500 ^{0}C- 2000 ^{0}C.

4.1.3. Implementations

Firstly, magnesium oxide has widespread usage for several industrial sectors. It is refractory substance which is very consistent at great temperatures as a physically and chemically. In construction area, it is effective fire-proofing material used in magnesium oxide wallboards for such aims as resistance against fire, termite, moisture, mold and mildew and strength. Also, chemical, nuclear or superalloy sectors, it is used as corrosion preventer. In medical industry, magnesium oxide as in the drugs aids hearth burn and sour stomach, as an antacid, short-term laxative and supplement. Moreover, several another practice of magnesium oxide such as in fertilizers, water treatment, insulators and coating exist. Statistically, magnesiais mostly used in refractory industry, spent nearly 56% of magnesia in United States in 2004 and rest of that is utilized in chemical, agricultural, construction, environmental, medical and other operations.

Moreover, magnesium oxide is the constituent of Portland Cement in dry operational facilities. As environmentally, MgO is applied broadly in treatment of wastewaster, air emission, drinking water and waste disposals together with groundwater reclamation due to acid protection ability and similar impact for the dissolved heavy metal types.

Those soluble heavy metals influence the existence of species and balance of the soil-groundwater complex, so these are toxic to the livings and the reduction of heavy metal's concentration is needed. At this point, MgO stabilizes the pH level of metals-contaminated soils and wastes by forcing into the 8-10 interval in which majority of metals are insoluble and precipitated out of solution. Thus, MgO is appreciated as the most powerful stabilizator of metals compared to Portland Cement, lime, dust kiln and other special materials because of dominant buffering ability, cost performance, safety of delivery. Nowadays, nano-magnesium oxide having effective fabric between 0.1 and 100 nm through chemical or physical methods are used as effective fillers in the large voltage isolation.Other specific utilization is listed below [14,15].

- As a food contribution as an anticaking substance presented in cacao product, frozen dessert and peas,
- As a white color reflector in colorimetry due to its efficient reflectivity and diffusionability,
- As a reagent in the fixture of carboxbenzly (Cbz) group through benzyl chloroformatein EtOAc for the N-preventition against amines and amides,
- As an inhibitor against enlargement of particles in ceramics and advances their toughness of fracture by converting fissure expansion process at nano-size,
- As a preservative coating in plasma monitors,

4.2. Hydration of MgO

4.2.1. Phases of Hydration Reaction

Hydration of MgO including several dissolution phases and precipitation phase under 90 ^{o}C temperature is presented step by step in below.

- $MgO(s) + H_20(l) \Rightarrow MgOH^+$ (surface) + $OH^-(aq)$
- $MgOH^+$ (surface)+ $OH^-(aq) \Rightarrow MgOH^+.OH^-$(surface)
- $MgOH^+.OH^-$(surface) $\Rightarrow Mg^{+2}(aq) + 2OH^-(aq)$
- $Mg^{+2}(aq) + 60H^- (aq) \Rightarrow Mg(OH)_6^{-4} (aq)$
- $Mg(OH)_6^{-4} (aq) + Mg^{+2}(aq) \Rightarrow (Mg(OH)_2)_2(OH)_2^{-2}(aq)$
- $(Mg(OH)_2)_2)(OH)_2^{-2}(aq) + Mg^{+2}(aq) \Rightarrow (Mg(OH)_2)_3(s)$

4.2.2. Mechanism

As the explanations about the procedures, these may be stated as follows.

- Water transforms into OH^- and H^+ ions through the electrolysis.
- H^+ moves on the MgO surface.
- Mg^{+2} ion is produced after H^+ and OH^- attaching into MgO and it is known as the dissolution of MgO.
- Mg^{+2} ion is absorbed by ionized OH^-.
- $Mg(OH)_6^{-4}$ growth particles are created in the following that Mg^{+2} amd OH^- ions specific degree of saturation.
- As reaction lasts, growth particles unite with Mg^{+2}
- Eventually, Mg(OH)2 is produced and precipitated.

After water transformation by electrolysis, H^+ attacks and attaches on the MgO surface. Then, Mg^{+2} and OH^- ions are produced. Mg^{+2} ions stay as being attached into the MgO surface, whereas OH^- ions are seperated from MgO particles. Next, Mg^{+2} and OH^- concentrationsreach to certain saturation degree and by this way, $Mg(OH)_6^{-4}$ particles are grown on the part ofMgO surfaces. After Mg^{+2} ions are bonded with $Mg(OH)_6^{-4}$ and following products, $Mg(OH)_2$ crystals are generated on the internal MgO. By this way, the concept of dissolution and precipitation is carried out during hydration. Initially, single $Mg(OH)_2$ crystal is formed far awayfrom each other, but by the time, $Mg(OH)_2$ crystals growing on the both MgO surface and poresare combined to generate one layered sheet like arrangement. After certain period, MgO internals are completely wraps due to enveloping effect of $Mg(OH)_2$ sheets. Therefore, the waterparticles could not encounter with MgO solid particles and conversion of MgO into $Mg(OH)_2$ is stopped.

4.2.3. Reaction Kinetics

The hydration of MgO is generally defined in the scope of second order regarding chemical kinetics. However, the concentration of MgO depending on the amount of water in the system is at very low level as stated from 10% to 20%. Due to this reason, hydration reaction turns to first-order reaction and the rate of reaction is calculated by considering this. Despite this, there are two models about reaction kinetics for MgO hydration according to previous studies. These are first order kinetics model and multi-rate order kinetics model.

According to first-order reactions, , the continuity of the dissolution of Mg^{+2} ions is provided, so the difference in rate of hydration between MgO units is removed and the homogeneity in the reaction is created. Even though the production rate of $Mg(OH)_2$ is affected by wraps of $Mg(OH)_2$ on MgO surface, all MgO particles undergo into the hydration at same grades. In other words, hydration rate of particular MgO grains is kept constant up to the end of the reaction. In addition to these, the order and model of hydration kinetics is dependent to surface area of MgO, the stirring type on mix, time of hydration, reaction phases and concentration of reactants.

Regarding the reactivity of MgO, low activity MgO based on chloride ion absorption without any external force has first-order kinetics model because the size of MgO particles is high which provides slow hydration. By this way, every MgO enters to the hydration under same rate, so there is no difference in the hydration rates of MgO on the system. Nevertheless, rate of hydration for low activity MgO particles is lowered by the time, since the period of time for the hydration of low active MgO is not long enough. Therefore, there is hindering effect after a certain time, but the main relevance of low reactive MgO with first-order kinetics under unforced condition is basically similar hydration rates of every MgO particles due to larger grain size.

In addition to this, the stirring type has great importance on the hydration. The hydration under ultrasonic mixing enhances the dissolution and diffusion of MgO and avoids the $Mg(OH)_6^{-4}$ units growth on the surface of MgO. Therefore, $Mg(OH)_2$ particles as the products are firstly precipitated and then, $Mg(OH)_2$ units are peeled off from MgO surface due to the breakage of bonding between MgO and $Mg(OH)_2$ by ultrasonic energy. By this way, a new MgO particles are dissolved into Mg^{+2} ions by contacting with surrounding water in the mixture. The grow unit generation is supported through new nucleation sites in the solution, so MgO particles continuously decomposes from outside to inside. By means of ultrasonic irradiation, wraps of $Mg(OH)_2$ enclosing MgO surface are blocked, so either constant dissolution of MgO or sustained $Mg(OH)_2$ production is provided. At this point, the model of reaction kinetics in terms of order is first-order kinetics model, because hydration rate for MgO is similar for everysoil grains. Moreover, the degree of MgO activity is not important under ultrasonic mixing condition. Low and high reactive MgO particles enter into hydration under first-order kinetics model [16].

Concerning the period of hydration, based on past study about the hydration of MgO it is inferred that the extension of hydration period causes to the breakage in the bonds between $Mg(OH)_2$ sheets and new MgO surface is available for further hydration. By this way, the size of $Mg(OH)_2$ gradually gets smaller and similarly, MgO particles continues to dissolve in the solution. Therefore, increase in period turns the kinetics model of hydration into the first-order kinetics model, because the mechanism in the hydration under this condition is same with what is stated in above. Moreover, the degree of reactivity or surface area value of MgO is not considered. The main relevance of longer duration with the reaction kinetics model is such that it leads to open new dissolution points for the MgO on the system with full of $Mg(OH)_2$ wraps at MgO surfaces. Thus, every MgO particles again start to chance for decomposition in the mixture, so similar hydration rate for each MgO grain is provided and first-order kinetics model is again available under this condition [17].

Moreover, the kinetics model of hydration is influenced by the reaction phases which involve the type of phases of the reactants such as solid, liquid, gas or etc. According to recent effort towards this, it may be understood that $Mg(OH)_2$ molecules enveloping the all MgO poresis peeled off from MgO surface through the strong impact forces causing breakage between $Mg(OH)_2$ crystals. The impact forces come from the micro explosion happened in the solution. Under three phase conditions, this effect is brought by the gas phases in the mixture. New MgOsurfaces is continuously available for the diffusion and dissolution, so hydration rate for singularMgO becomes constant and the reaction starts to have first-order kinetics model [18].

In addition to first-order model, multi-rate kinetics model is such that the rate of hydration is affected by the $Mg(OH)_2$ wraps and there is the difference in the hydration rates between MgO groups. This type of model is observed for the hydration of high activity MgO under 2- phases as reaction system and zero or specific number of RPM-based mechanical stirring methods, because some MgO particles reacts faster than what others do. Therefore, the formation of precipitated $Mg(OH)_2$ particles is variable from the area to area, so in the locations where high activity MgO is available, solid $Mg(OH)_2$ crystals wrap to specific MgO surfaces on the system. By this way, due to this enveloping effect, diffusion of water into MgO surfaces is prevented and MgO units are not dissolved into the water. Hence, under the determined period, high activity MgO particles have different hydration rates, so multi-rate kinetics model is valid for this type of hydration. As an important notice, to make decision about the degree of MgO reactivity, Chloride Ion Absorption test results are vital. Extra, the hydration under ultrasonic mixing has the potential to cause the reaction whose kinetics model is other than multi-rate kinetics model [16].

From the former studies, it makes sense that the reaction system, the type of stirrer and duration of hydration affects to the reaction kinetics of high activity MgO reactants. Especially, multi-rate order kinetics model is observed for the hydration having 2-phase reaction system, short period of time and mechanical or magnetic mixing conditions [17-20].

4.2.4. Reaction System

There are two types of reaction systems in the MgO hydration from the previous studies. Depending on the phase type of component within the hydration, these are namely 2-phase and 3-phase reaction systems. In 2-phase reaction system, general dissolution and precipitation procedures exist and hydration reaction occurs between MgO as solid and water as the liquid. Therefore, in 2-phase reaction system, there should be the liquid and solid phases in the hydration. In addition to this, 3-phase reaction system consist of the gas, liquid and solid as the phasesof reactants. As an example of gas used in hydration according to recent studies, the water vaporis utilized for this purpose. Also, it is observed that the type of reaction system has significant effect on the hydration of MgO such as changing the reaction model, the model of reaction kinetics, rate of hydration, morphology of products and maximum conversion degree of the hydration [18].

4.2.5. Reaction Models

Among several actual reaction models describing the mechanism of reaction, there are two sorts of hydration reaction model based on foregoing investigations which are semi-empirical reaction model and shrinking core reaction model.

4.2.5.1. Semi-Empirical Reaction Model. In semi-empirical model, after water transformation by electrolysis, H+ attacks and attaches on the MgO surface. Then, Mg^{+2} and OH^- ions are produced. Mg^{+2} ions stay as being attached into the MgO surface,whereas

OH^- ions are seperated from MgO particles. Next, Mg^{+2} and OH^- concentrations reach to certain saturation degree and by this way, $Mg(OH)_6^{-4}$ particles are grown on the part of MgO surfaces. After Mg^{+2} ions are bonded with $Mg(OH)_6^{-4}$ and following products, $Mg(OH)_2$ crystals are formed on the internal structure of MgO. At the beginning, ,there a few amount of crystal products, so the total surface area of $Mg(OH)_2$ solids is high but by the time, these crystals combines together to turn the larger crystals having thin leaf plate like formation.Then,the formation of these plates gets greater and becomes more firmer by the time,so it turns to single layered the sheets formations covering the internal MgO. In this way, total surface area of $Mg(OH)_2$ solids decrease due to the change in morphology. Also, the shape of $Mg(OH)_2$ is uniform and hexagonal in general. Therefore, the contact between internal structure of MgO and water is prevented by packaging of $Mg(OH)_2$ and by this way, the dissolution of MgO into Mg^{+2} is limited. It is known as the enveloping or the sintering effect of $Mg(OH)_2$ tabular crystal. Also, the conversion degree from MgO to $Mg(OH)_2$ starts to be constant after short period of time. In other saying, under specificduration, formation of $Mg(OH)_2$ crystals does not exist anymore after a specific period of time. At this point, based on semi-empirical model, the internal structure of MgO emphasizes both ofthe external surface and inside the pores of MgO particles. Also,this model assumes that microstructural formation of MgO is porous and because of this assumption, there is no actual surface of MgO, but also it is the extension of the pores. The location of $Mg(OH)_2$ formation starts inside the pores and it continues into the external surface of MgO or extension of pores. Hence, the porosity of MgO during the hydration plays an important role and even, hydration rate is depending on how the porosity level is changed with the period of time. Again, the semi-empirical reaction system is basically based on the dissolution of MgO, diffussion of water, precipitation and sintering effect of $Mg(OH)_2$ crystals [16,18,19,20]. From background, semi-empirical model is applicable for the hydration having 2-phase system,the mechanical mixing types based on rate per minutes system,short period of time. Under these conditions, reactivity of MgO or the reaction temperature does not influence the reaction modelof hydration. Moreover, the temperature has the potential to change the duration of hydration by affecting to based upon the sintering effect of $Mg(OH)_2$. Lastly, some external effects such as longer duration, the usage of ultrasonic mixer and changing the phase system may impact on the reaction model [16-20].

4.2.5.2. Shrinking-Core Model. In shrinking core model, it is assumed that MgO particles are spherical, so hydration growth mechanism occurs on the surface of MgO. By this way, the diameters of MgO spherical particles gradually diminishes due to the reason that $Mg(OH)_2$ products are peeled off from the growth locations on the MgO surface.In other saying, MgO particles are continously dissolved from outside to inside,so the size of spherical grains gets smaller. Actually, depending on the hydration condition, MgO pores may be covered with $Mg(OH)_2$ crystals at the beginning of the hydration, so hydration is hindered by $Mg(OH)_2$ crystals forming the single layered sheets in the MgO internals, but through the external and internal agents, the bonds between MgO and $Mg(OH)_2$ particles are broken together with the decomposition of plates of $Mg(OH)_2$ particles. As a result of this, hexagonal $Mg(OH)_2$ crystals are peeled off and dispersed into the different locations in the complex. Therefore, instead of constituting the wraps around MgO surface, $Mg(OH)_2$ particles form the supernatant layer(s) above on the underlying products such the complexes of the MgO and $Mg(OH)_2$ particles. These dispersed products may exist in unlayered formation on the system. Also, precipitated crystals which are peelled off from the MgO surface shows the amorphous characteristics, so these are loosely gathered in the supernatant layer whose relative density is low. For this reason, supernatant layers float on the surface of subjacent pieces. In spite of this, depending on the condition, $Mg(OH)_2$ products are pulverized by different impact and thus, these are not packaged or layered as mentioned above. As a result of these, the wraps of $Mg(OH)_2$ products do not prevent the dissolution of MgO particles in the water. In the microparticular perspective regarding mechanism, after MgO is seperated into ions as Mg^{+2} and OH^-, there is repetitive peelling off $Mg(OH)_2$ solids within the internal MgO. The number of $Mg(OH)_2$ molecules increases while more Mg^{+2} ions are present in the solution as a result of decrease in the size of MgO spherical particles. However, if MgO particles becomes very smaller, the peeling off mechanism is no longer available and the hydration reaction starts to be limited. For the externalor internal agents, there are several notices from recent sutides. As the first agent is the changing reaction system.Likewise, in case of the usage of vapour pressure for the gas component in the3-phase reaction system, this high pressure steam either leads to overheat the water or providesthe decrease in completion of hydration. Thus, powerful effects from microexplosion due to overheating in the solution causes the wraps of $Mg(OH)_2$ sheets enveloping MgO internal surfaces to be peelled off from there,so fresh MgO surfaces are available to be attacked by surrounding water. By this way, continous formation of $Mg(OH)_2$ crystals is gained into the system,so the efficieny in the hydration is provided. For another agent, the extension of time has important role to change the reaction model into shrinking core model. As the mechanism, the stress generated at the layer of $Mg(OH)_2$ wraps within the MgO internal leads to breakage in bonds between the $Mg(OH)_2$ crystals.By this way, there is a peeling off-like mechanism in the system, so $Mg(OH)_2$ crystals are seperated from the mass and scattered in the solution. As thesebreakages continues, fresh MgO surface becomes ready for another hydration. As a result of this, the diameter of MgO and $Mg(OH)_2$ particles gradually reduces and thus, total surface areaof $Mg(OH)_2$ solids rises. Regarding the reaction model of the hydration under this condition, shrinking core model is more prominent. Extra, the type of mixing is able to change the reaction model in the hydration. One of the example is the utilization of ultrasonic mixer. It supplies the ultrasonic irradiation into the system, so it creates the cavitation in the solution. This cavitation enhances the diffussion of water and dissolution of MgO into ions. Hence, $Mg(OH)_4^{-6}$, the evolution entities are precluded on the MgO pores. Also, irradiational energy breaks the bonds between MgO and $Mg(OH)_2$ particles, so $Mg(OH)_2$ precipitates are peeled off and pulverized. By this way, the more surface of MgO and further nucleation placement is available for water contact by supporting the growth unit formation. Next, MgO spherical particles decompose from outside to inside with the release of Mg^{+2} ions by the time. Therefore, precipitated $Mg(OH)_2$ crystals are instantly peeled off and those are scattered in the form of dispersive platelets. As a consequence, ultrasound energy prohibits the agglomeration of $Mg(OH)_2$ growth on the MgO surface during the hydration. Thus, there is an increase in the number of $Mg(OH)_2$ particles with controlled hydration rate. In morphological point of view, the hydration under 3-phase reaction system gives the amorphous and weakly combination of $Mg(OH)_2$ crystals which are peeled off from MgO surface. This combination creaates the supernatant layer floating above on the surface of hydration products due to havingthe less relative density. Moreover, the shape of $Mg(OH)_2$ within supernatant layer is not uniform with fuzzy layout, whereas $Mg(OH)_2$ crystals on the underlying sheets have hexagonal shape On the other hand, in this type of hydration, there are also hexagonal $Mg(OH)_2$ sheets as single layer between MgO spherical particles. Therefore, some pits in MgO surface with two $Mg(OH)_2$ layers exist in the hydration under 3-phase reaction system according to the relevant effort. For the hydration with ultrasonic mixing, the growth unit includes the formation of $Mg(OH)_2$ particular crystal on the surface of MgO and pulverization of this single crystal into the surronding solution. Therefore, those seperated precipitates form the tiny platelets having hexagonal or non-uniform shape of $Mg(OH)_2$ crystal. The reason for difference in the shape of $Mg(OH)_2$ crystals may be arised from the ultrasonic irradition which breaks the gravity between MgO and $Mg(OH)_2$ particles. Also, there is no any packaging between dispersive $Mg(OH)_2$ platelets, so supernatant layer-like outputs are not valid for ultrasonic mixing [16-18].

4.2.6 Reaction Conditions

4.2.6.1. Temperature. The effects of temperature on the hydration are examined in the previous studies. As a result of these, temperature inversely influences the conversion constant which is the parameter related with the transformation level from magnesium oxide into magnesium hydroxide. Therefore, an excessive increment in the temperature decreases maximum conversion level of hydration within the certain time. However,rise in the temperature shorten the time to reach maximum level of conversion between MgO and $Mg(OH)_2$ because of early arrival of the activation energy. Hence, it seems that the maximum conversion level for the hydration having short duration under high temperature is greater than those of hydration underlow temperature, but it is completely matter of the duration of hydration. In the light of this information, it is important to provide both low temperature and longer duration as much as possible regarding the condition for the efficient hydration [17-20].

4.2.6.2. Specific Surface Area and Activity. Based on recent studies, the activity is determined by the specific surface area of MgO particles which is influenced by the calcination conditionssuch as temperature and the period of time. While the specific surface area gets smaller, activity of MgO particles increases as an quantitatively. Similarly, activation energy for the hydration rises as the activity of MgO particles increases. Even though denser arrangement between $Mg(OH)_2$ wraps is obtained for more active MgO particles, there is a defect in lattice structure in case of high activity. Moreover, the level of sintering during hydration increases with the development in the activity of MgO particles because of high reactive tendency towards the hydration. Also, the change in activity may be responsible for the effect on the type of reaction kinetics. For instance, the short period of 2-phase hydrations under mechanical mixing or non-mixing condition has the multi-rate kinetics model for high active MgO reactants or first order steady-state kinetics model for low active MgO particles. Also, conversion level of hydration from reactants into product under short period of time is directly influenced by the degree of particular activity. However, if the duration of hydration is extended, the influence of activity on the hydration is removed. Therefore, under longer period of time, the model of reaction kinetics for either low or high active MgO particles becomes steady-state first order kinetics model. Likewise, the hydrations under ultrasonic mixing give first-order steady state kinetics model for both situation concerning the degree of activity. Lastly, the reaction model of hydration is not affected by the degree of acitivity but also influenced by type of mixing ,the reaction-phase and the extension of time.

4.2.6.3. Time. It is realized that extension of duration advances the conversion quantity of hydration without depending on the activity and temperature. As an example for temperature effect, the hydration done under higher temperature has the potential to reach greater conversion value compared to the hydration at lower temperature the end of same hydration duration. However, in case of the extension of time for only the hydration at lower temperature, the conversion value exceeds to the one coming from the hydration under higher temperature and shorter period of time. Also, from prior studies, the hydrations of various active MgO particles under same temperature presents that the conversion value of lower active MgO particles may catch up to the those of higher active MgO particles thanks to extension of duration. Similarly, the hydrations made in the previous efforts have generally limited period of time, various or constant temperature and it may be inferred that the maximum conversion levels under longer duration reach to the point much greater than the conversion level under shorter duration. Moreover, the actual impact of ultrasonic mixing condition may be more observable if necessary time is given to the reaction. Extra, time of hydration is more important than the temperature and specific surface area or activity because it affects to reaction kinetics and reaction model directly. When the duration is prolonged, the model of reaction kinetics turns to first order kinetics model without considering the activity level. Similarly, reaction models become the Shrinking Core Model instead of Semi-Empirical reaction model in the case of longer period of time. As a result of these, extension of time provides the more productive and efficient hydration [16-20].

4.2.6.4. Mixing Type. The hydration carried out under rate per minutes (RPM) based mechanical mixers show that the level of RPM as the number is insignificant regarding the conversion degree according to previous outcomes. The range of RPM does not have any impacts on the reaction models and reaction kinetics. In other saying, the hydration done with mechanical mixers under zero RPM gives the nearly similar conversion values to the one undersimilar mechanical mixers having more RPM than zero. On the other hand, ultrasonic mixers remove the variation of the hydration rates between MgO particles. Therefore, all MgO particlesare hydrated at the same grades. By this way, the model of reaction kinetics turns to first-order kinetics model from multi-rate kinetics model without depending on the activity level of MgO particles. Likewise, the reaction model of hydration under this condition becomes Shrinking Core reaction model. On the other hand, from the previous studies, it is observed that the ultrasonic mixing condition does not impress the conversion degree of hydration within the shortrange of time except small increase. Nonetheless, the prolongation in the time may show the actual impacts of ultrasonic mixing on the hydration [16-19].

4.3 Application in Civil Engineering

In one of investigation about the effect of MgO on the cement, MgO powder was added into mix as much as seperately %1, %2, %3, %4 of dry weight of cement. The mixes consisted of cement, sand, gravel and water in the certain proportions. Also, as an accelerator, some amount of plasticizers were included. Samples were formed in the cubic mould and treated for determined periods such as 7, 14, 28 and 60 days. Regarding the condition, samples were inserted into autoclave throughout 18 days and immersed by water up to test day. As a result of this investigation, the samples having %2 MgO gave the highest compressive strength, lowest percentage of water absorption and maximum pulse velocity by ultrasonic pulse velocity test after 28 days. The main consideration was caused by the great fienness of MgO at %2 level. Therefore, high level of fienness and autoclave usage accelerated the chemical interaction of MgO, so larger spaces for MgO products are provided. By this way, cracks in capillary pores were reduced. The result of water absorption and Ultrasonic Pulse Velocity Test concluded thatthe samples with %2 MgO had very low cavities and firm texture [21].

Another study was related with the influence of hydration characteristics of MgO replacedand used together with Portland cement on pulverized Fly ash. In this effort, the mixture aiming to be used as construction material contains minimum 50% as the dry weight Pulverized Fly Ash and rest was changed between Portland Cement and Reactive or Dead Burnt MgO. Also, each mixture had specific ratio of water to solid content. Then, all samples were prepared by being casted in cylindirical mould and these are waited for 2 weeks under 98% humidity. Then,there was another 3 weeks treatment under the 30-45% as the level of humidity on the purpose of drying. Lastly, samples extracted from moulds were crushed into fragments in order to be dried in oven for 24 hour at 105 ^{0}C. Therefore, the evaporation at the oven stated Total Porosity, so how much water content within small pores was observed. Likewise, vaporization under 30-45% as level of humidity defined the Capillary Pores concerning the volume of water within large pores. Additionally, under low level of humidity the water content within the small pores was not evaporated due to capillary water suction of large pores. As a consequence of this study,it was observed that reactive MgO consumed more water than Portland Cement. Moreover, dead-burnt MgO needed the water less than the one for Portland Cement, because hydration reaction for dead-burnt MgO was very slow. Also, MgO hydration was not affected by Portland Cement's pozollanic reactions between silica-oxide from Fly Ash and $Ca(OH)_2$ from Portland Cement, but also depended on activity of MgO. Nevertheless, when only MgO, especially reactive type, existed in mixture, formation of $Mg(OH)_2$ became more efficient. Moreover, with the addition of other components into $Mg(OH)_2$, some products were formed like hydrotalcite containing MgO-Al_2O_3 bonding. Furthermore, hydration product of MgO was crystalline unlike Portland Cement product which was in the form of gel. Extra, reactive MgO broke the packaging system of the mixture, but in case of the usage of dead-burnt MgO, homogenous, densely packed paste mix was easily observable. Portland Cement forming microporous gel filled the large pores within the Fly Pulverized Ash similar to sand about the content, whereas MgO was less effective than Portland Cement regarding filling large pores because of the solid-crystalline hydration product named as brucite. Even, MgO content led to very porous microstructre since it generated the volume expansion within the mixture and thus, the presence of MgO caused to the reduction in compressive strength depending on the existence of large pores in the blend, even though hydration products were more solid than the one by Portland Cement [22].

Next study was about the performance of reactive MgO with the high percentages on the cementitous mortars properties. Portland Cement, in this research, was substituted by light-burnt MgO in the percentages varying from 10% to 20% of dry weight of Portland Cement. The mixture was made by using sand, Portland Cement, MgO and water with the determined proportions. Samples were mixed and cured for several days under room conditions. In matter of results from tests conducted on samples, there were considerable content of amorphous silicon dioxide in sand which reacts with $Mg(OH)_2$ from MgO hydration, so the complex,known as Magnesia Silica Hydrates was the responsible for the strength development caused by MgO hydration. From flexural and compressive strength tests, the resistance of samples were decreased as MgO presence was increased due to the rise in ratio of water to binder in mixturesand the $Mg(OH)_2$ formation diminishing the hydration product of Portland Cement. Moreover, additional water caused to more porous microstructures for specimens including 15% and 20% MgO content,so strength reduction emerged. Extra, at early curing age, only $Mg(OH)_2$ contributed to strength development, but Magnesium Silica Hydration as cementitous product was essential for strength gain coming from MgO. In spite of this, aluminosillicate mineral provided the reinforcement to the mixture, since it reacted with $Mg(OH)_2$ to form hydrotalcite as an output decreasing the porosity in complex except the strengthenning. Also, if the reactivityof MgO was increased, water requirement became higher and thus, less resistant porous microstructre was formed. In addition to strength experiments, specimens were posed tocarbonabiton for 28 and 91 days and carbonation was efficient at initial stage of hydrations, because generation of further hydration products after a period of time caused to reduce the volume of porosity within microstructre. Therefore, the porous mass in specimens gradually decreased by the time. Regarding water absorption by capillary action, water absorptioncapacity slighlty increased along with additional MgO content because of increment in total pore volume. Shortly, MgO caused dilution and reduces Calcium Silica Hydrates, so strength was reduced. MgO needed greater water demand and it caused to generates more porous structure, but with this way, carbonation and water absorption by capillary action became easier. Specimens with MgO presented lower shrinkage depending on quantitiy of hydrated compoundsand MgO reactivity due to the volume expansion due to the $Mg(OH)_2$ hydration. Lastly, in the view of microstructure, MgO in the mortars reduced the cracking caused by shrinkage,so therewas a point of which MgO improves durability of material [23].

Following work was related with Magnesium Oxychloride Cement comprising of MgO and $MgCl_2$ within the specific proportions. In this study, it was observed that the reactivity of MgO influences the reaction rate and characteristics of the products. In the point of strength gaining mechanism during hydration, Phase-5 products had great importance. For the sample preparation, MgO and $MgCl_2$ in the form of hygroscopic hexahydrate crystal as $MgCl_2.6H_20$ asformulation were mixed with water and poured into cubic mould under vibrational compaction.There were also several molar ratios between MgO, $MgCl_2$ and water. Moulded samples were waited for 3, 7, 14 days under room condition for Compressive Strength, X-Ray Diffraction and Ternary System Phase Diagram Test. Eventually, in the case of increase in water content, compressive strength enhanced and it had same trend as MgO amount became greater. The generation frequency of Phase 5 crystallines along with the rate of reaction arrived the fastest at the point where $MgO/MgCl_2$ ratio reached 13 together with that $H_2O/MgCl_2$ is 12 as all in moles. However, previous findings remarked that $MgO/MgCl_2$ proportion was theorethically 5. Consequently, the reactivity of MgO was considered as principal factor and the reactivity was lowered as rate as 52% in percentage. Hence, the highest compressive strength more than priorwas founded at the point where mole fraction between MgO and $MgCl_2$ was converted into 6.8 instead of 13 as numerically. Moreover, the resistance of MOC cement was not subjected to the production of needle-like Phase interlaced crystals but also pore structure of the products and water content within the mixture were very effective. Likewise, the amount of required water depended on MgO reactivity for efficient MOC workability. Additionally, excess MgO powder behaved as a filler and thus, denser microparticular system aganist compressive actions wasprovided [24].

Another work was made by using polypropylene fibers and micro-silica for the purpose of improving the MOC cement properties. The content of specimens comprised of sand, gravel, MgO, $MgCl_2$ and Micro-Silica or Polypropylene Fibers if needed. All mixtures were poured into cubic mould to be cured for 28 days under standart room conditions. Then, all samples weredried for compressive strength, water tightness and corrosion test. After all, microsilica showedbetter results compared to polypropylene fibers such as enhancing the compressive strength as 17.5% increase as percentage, filling the gap in concrete, providing greater stability and well- formed crystalline skeleton by reacting with particles. In addition to this, it was known that MOC cement had poor water resistance. Hence, when both of them was utilized, ,these enteredinto the pores of concrete and thus, water penetration into samples and acidic corrosion in the specimens were reduced. Although Polyproplyne decreased the cracks from shrinkage, excess usage gave negative results [25].

What's more concerning the advancement of MOC cement was the application of sawdust and perlite consisting mainly Silicon Dioxide as the aggregates. Those additives was used to enhance the non-hydraulic and low alkaline properties of Magnesium Oxychloride Cement. The samples were prepared with changing content of perlite and sawdust, but the quantity of MgO and $MgCl_2$ was kept constant. The curing period was 1 day under 20 ^oC and 60% humidity andthen, it lasted 3 days after the samples were put into steam condition under 60 ^oC and 60% humidity. Next, samples were hardened and dried at laboratory for 1 day. As a results of these,the perilte instead of sawdust decreased swelling characteristics and enhances volume stability of MOC without causing bulk density higher. In addition to this, it was observed that the increasing amount of perlite within the sawdust provided better strength and temperature resistance of the MOC. Therefore, it was concluded that SiO_2-based materials had strong consistence with the substances involving MgO and $MgCl_2$ [26].

Similar to previous investigation, the influence of sand with different types of water on Magnesium Oxychloride Cement performance was examined in the following effort. The samples were composed of MgO, $MgCl_2$ in the form of hygroscopic hexahydrate,sand, water asseperately drinking and sewater and small amount of addivites such as glass fiber and ammonioum dihydrogen phopshate. All substances were mixed with the various water and sandcontent,but the amount of MgO, $MgCl_2$ and additives was the same. Then,the samples were formed by addition of all matters and cured for 3, 7 and 28 days under 26 ^oC and 40% humidity. As a conclusion of several tests, the bulk denisty of the treated MOC got greater if the sand proportion increased. Moreover, the increase in bulk density reduced the total porosity and water adsorption. Additionally, soluble phosphate reacted with $Mg(OH)_2$ and generated the hydrated products whose solubility was very low, because it filled capillary pores to make the MOC paste denser. By this way, all surface of Phase-5 grains were protected from decomposition by water, so it was observed that small quantity of soluble phosphates enhanced the water resistance of MOC in the hardened form. Likewise, sand behaved as water repellant filler due to the very fine texture. Afterwards, the strength of MOC developed with the time and the usage of seawater, since several precipitation reactions on seawater consisting MgO accelerated the $Mg(OH)_2$ production with following hydration phases of MOC. On the other hand, if the sand amount rised, the MOC strength was lowered due to the reason that sand did not have hydration properties except water repellant characteristics. The effect of soluble phosphate on strength made it constant. Lastly, the psyco-mechanical behavior of MOC samples was more advanced than concrete and limestone after 28 days [27].

There was further output about the impact of the moles of MgO, $MgCl_2$ and water on the mechanical tendency of MOC. According to this, the specimens moulded in cubic cast included constant percentage of MgO and water, but the moles of $MgCl_2$ was varying. It was provided that samples were waited at 25 ^oC for 20 days. Later, XRD, SEM, cold crushing strength, the period of settling, water dissolution test were made on all. Ultimately, it was seen that crystallization needed the time changing from 1 days to weeks and prior to crystallization, amorphous gel phases were formed by means of gelation.

Therefore, during the conversion from amorhous to crystallized products, the presence of necessary Cl^- ions with water had great importance. Nonetheless, optimum Phase-5 needle shaped crystals created during fifth step were responsible for the strength growth. In this point, excess $MgCl_2$ moles in the system generated too much structural water filling the pores in cement and thus, the decrease in the strength was observable if the amount of $MgCl_2$ with the structural water was not care. In spite of this, at optimum $MgCl_2$ level, excess MgO monitored in the system behaved as a filler and had the potential to reduce Phase-5 products in case of sufficient $MgCl_2$ available in complex. Also, the extension of curing period gave the raise in the quantity of Phase 5 within the composition. Extra, the setting time increased depending on the additional $MgCl_2$ moles, since the time to generate Phase-5 crystals got longer. Similarly, more porous texture on the complex arised due to the factor that increment of $MgCl_2.6H_20$ moles leads to gather the free water and structural water generating large pores in the system. By this way, diffussion of water into the sample anddissolution of the sample in water became easier. Another output was that the size of Phase-5 crystals got coarser as $MgCl_2$ content rises. Also, it was revealed that the phases different than Phase-5 were created in the event that the amount of $MgCl_2$ compared to MgO was lesser, because the interaction between $Mg(OH)_2$ and Phase-5 crystals happened. Lastly, the usage of nano-silica carbide which was silicon based substance was proven to be as beneficial for the results such as strength gaining in the MOC [28]**.**

In additional practice, sewage sludge with chemicals was used to enhace the physical and mechanical efficiency of MOC including MgO, $MgCl_2$ bounded with H_2O and water. The specimens were ready after stirring MgO, $MgCl_2.6H_20$, water and chemical sludge up. Afterwards, these were moulded for 1 day and extracted to be cured for 10 days at 20 ^0C temperature. The various ratios between MgO and $MgCl_2$ in moles were on the contrary with the changing proportions between MOC and chemical sludge in mass. Following the several tests conducted on samples, at the level where MgO was three times of $MgCl_2$ in the moles, maximum strength was attained. Similarly, strong strength development existed at the mass ratio of MOC to chemical sludge as 3:100. Moreover, on this couple as proportions, shortest settling time was obtained. The components of MOC consumed the water presence within the chemical sludge, but lack of MOC in the complex caused uncompleted gelling and crystallization reactions during the hydration of MOC. Even though some of $Mg(OH)_2$ remained as an unconsumed situation, some interacted with SiO_2, Al_2O_3 from chemical sludge, so the pozollanic reaction happened during solidification of the samples. Nevertheless, the products from Phase 3 and Phase 5 had main responsibility for the strength. Despite these,water adsorption capacity of $MgCl_2$ was high due to being powerful electrolyte and the effect of MgO on the environment was at minimal level and it had high alkalinity which helped to precipitate the metals by neutralizing the acids. Also, as time gets longer, phase 3 and 5 products of MOC with Chemical sludge may have generated crystallize network which limited the heavy metal ions precipitation on the system [29].

4.4. Application in Geotechnical Engineering

In another performance, fine sand with powder nano materials such as nano-SiO_2, nano-Al_2O_3 and nano-MgO in terms of stifness and compressibility under different confining cases was examined. After the preparation of the sample with various cement and nanomaterials content, it was founded that all nano-matters provided the great interlocking of the grains by enhancing effectively the intensity on the surface roughness of them and filling the intergranular pores. By this way, the friction between grains was improved which was important for the deformation reduction in the soil. As an example, %1 addition of any type of nanomaterials raised the Young Modulus to the level of 36 %, 55% and 24 % under the case of unconfined, partly confined and confined, successively. Extra, the pozzolanic products decreased the weak fragment of hydration mechanism and advances the powerful pieces of it through consuming $Ca(OH)_2$ which generated C-S-H. Hence, volumetric and axial strains were diminished. Depending on the types, nanomaterial diminished the porosity and enhanced the stiffness. Lastly, the pozzolanic capacity of nanomaterials had great influence on the strength of the cemented sand [30].

There was further examination associated with the effect of $Ca(OH)_2$ and $Mg(OH)_2$ on the tropical residual soil. These hydroxides were produced by a quick precipitation method and mixed with the soil. According to the results of some chemical visualization tests such as XRD,SEM, EDX and several geotechnical tests involving Atterberg limits, unconfined compressive strength, falling head permeability and compaction, calcium and magnesium hydroxides decreased the hydraulic conductivity and enhanced the UCS value of the soil. As an amount, thereduction of hydraulic conductivity after 98 days was %85 by the magnesium hydroxide and 98% by the calcium hydroxide compared to original soil. Regarding UCS values, the strength growth after 14 days with reference to virgin soil was 148% from magnesium hydroxide and 180% by calcium hydroxide. Also, both were environmental-friendly and the effect on the soilis remarkable [31].

Next, the highly plastic clay was improved with the use of nano-MgO with various dose from 0.2% to 2.0% as of soil's dry weight. After obtaining optimum water content of soil, chemical was added into soil for mixing and molding. Then, samples were waited inside the tight bags for 28 days and these were submerged in water for 4 days in order to determine swelling properties and durability against volumetric change under saturated ambiance. Several tests such as EDXR-Spectroscopy Test, CBR, water tightness test was made to obtain the effect of nano-MgO on the soil. Eventually, at 0.80% dose, CBR value was increased as 1200% compared to pure soil in percentage. Also, at the same dose of MgO, soil swelling ratio was increased into 10% in percentage with respect to those of original soil, so the durability of soil against volumetric change under saturated environment was enhanced because of the both electrical and chemical bonds between nano-MgO and soil particles. According to EDXR's results, in dry condition, there was no reaction between nano-MgO and chemical components of soil. However, thanks to the addition of water, MgO particles were hydrated and turned into $Mg(OH)_2$ solids which reacted with aluminum and Silicon ions dissolved in the solution. As a result of this, Magnesium Silicate Hydrate compound was formed which creates strong bonds within the sample. Lastly, the size of nano-MgO developed the reaction intensity which was responsible for greater bonding ability inside the components in the complex [32].

In another study, the effect of two MgO grades, as being regular or nano-size, on the mechanical properties of silty clayey sand was studied. Before the mixing, several parameters of soil such as optimum water content, liquid limit and UCSC classification was determined. Samples were formed by mixing the soil having four different remolded water content from 16% to 22% and MgO chemicals in the doses sorting as 0, 0.3%, 0.5%, 0.8%, 1.0% of dry soil weight. For next, samples were compacted within the cylindrical mold and covered by plastic wrapping for the purpose of curing. Several tests were carried out in all samples, and these are Liquid-Plastic Limit Test done after 1 and 28 days on the samples, unconfined compression Test made on the samples following 1,3,7,28 days and stiffness calculation obtained at the end of 28 days. As a result of these, the extension of curing time increased to the reduction in Liquid Limitand Plastic Limit of samples meaning that soil resistance against moisture, workability and shrinking/swelling stabilization was enhanced. Also, the type and the optimum dose of MgO for the best results concerning Liquid and Plastic Limit was successively nano-type and 0.3% as ofdry weight of soil. Regarding UCS test result, the curing period increased directly to the strength of treated soil. Similarly, the addition of MgO enhanced the UCS results. However, increment in water content led to the decrease in strength.

Also, nano-MgO showed better results in the UCS test with respect to regular type MgO, because the ability to fill the pores within the soil was greater for nano-type of MgO. The ultimate UCS value which was 500 kPa as in the numberwas obtained by the samples having 1.0% dose of nano-MgO under the water content as 16% as percentage after 28 days. In addition to UCS test, it was observed that Young’s Modulus increase was in proportion with the doses of MgO, type of MgO and curing period of time. Moreover, the water content had negative impacts on the Young’s Modulus. As a result, MgO as treatment chemical changed the mechanical behavior of soil from ductile to brittle by increasing Young’s Modulus after analyzing the failure strain and stress. At the point about stiffness, the soil with 1% dose of regular-MgO having 20% and 22% water content presented stiff behavior, whereas the samples having 16% and 18% water content with same dose of regular-MgO showed very stiff tendency. On the other hand, all samples mixed with nano-MgOacted as very stiff soil without depending on MgO doses and water content level. For the reference, original soil showed the behavior between soft and medium stiff. Thus, it was proved that nano-MgO improves the geotechnical characteristics of residual soil [33].

Like to previous efforts, there were outcomes towards the influence of nano-MgO on the mechanical performance of clay. Under different contents of nano-MgO going from 0% to 6% as dry weight of soil in percentage, the samples were prepared within the cylindrical mold by adding of three variable water contents such as 10%,16% and 22% successively. Afterwards, samples were cured for 28 days under room condition and for 1 day under water before UCS test. As a consequence, the increase in water content diminished the strength gaining in the soil,but additional nano-MgO provided the rise in UCS values. Similarly, the soil with 6% nano- MgO content under 16% water content gave the maximum UCS result among all. Regarding the improvement process, nano-MgO bonded all clay particles to each other, so the quantity and volume of intergranular pores of soil was reduced. By this way, thanks to mixing soil with nano-MgO, large soil aggregates were formed through the pore filling, cementation and surfacewater adsorption effect of nano-MgO and thus, stiffness, brittleness and the morphology of claywere improved. Additionally, according to the path of strength increment from related graphs, high surface energy, tension of nano-MgO and the increase in nano-MgO content caused to dispersion effect, so strength gaining was negatively affected. The reason for this was that some of MgO particles did not enter to the hydration [34].

Despite this, one of recent studies was about the importance of hydrated MgO cured with CO_2 on the clayey silt sand soil compared to Portland Cement within 5% and 10% contents of dry weight of soil as in the percentage. For this purpose, all specimens were formed under mentioned contents of chemical with various water content starting from 2.5% to 15% as percentage. After molding phase of specimen inside the cylindrical cast, samples were waited for 1 hour and classified into two groups such as ambient and carbonated types. Ambient groupwere cured for 28 days at 20 ^{o}C temperature and 95% humidity, whereas carbonated series wereinserted into triaxial apparatus and subjected to carbonation process having different grades of permeation pressure of CO_2 and various periods ranging between 0.5 to 7 days. Later, all specimen from both groups were taken for UCS testing with constant displacement rate.

In addition to this, the standard proctor test for only ambient group was also caried out to see the relationship between optimum water content and maximum UCS value. Hence, it was observed that the UCS values of samples for both binders, as MgO chemical or Portland Cement, gave the maximum level among all, so the maximum dry density of soil-binder complex was important parameter to show the strength gaining within the treated soil. For the comparison between the effects of PC and MgO chemical in the soil, ambient cured MgO-stabilized soil had much lower UCS value than those of PC-based soil in the ambient group. Likewise, at the same content, MgO-based soil from carbonation group had 20 times higher than those of ambient group regarding UCS results. Also, UCS value of carbonated soil having 5% MgO content had 2 times compared to those of the ambient cured soil having 5% PC content. However, it was not valid for the comparison between the carbonated soil having 10% MgO content and ambient soil having 10% PC content because of the high MgO content, greater water consumption and excessive expansion of MgO with the carbonation process. The best condition for carbonation was lower water content and higher porosity within the complex.

However, at very low water content level, the hydration was not efficiently completed due to insufficient amount of water, whereas CO_2 gas started to not able to move properly into the soil as water content increases. Furthermore, the rapid strength gaining was provided through the carbonation and as an example, soils with 5% MgO and 10% MgO content carbonated just after 3 hours gave the UCSvalues equal to those of soils having same PC contents cured under ambient condition.

Despite the UCS result, according to SEM micrographs outputs, ambient cured sample having MgO showed the porous $Mg(OH)_2$ matrix which was limited binding characteristics due to its poorly interconnected structure, whereas carbonated samples presented stiff microstructure including the several products come from the carbonation of $Mg(OH)_2$. Lastly, from XRD diffractograms results, ambient cured samples had the two strong $Mg(OH)_2$ peaks and a weak MgO peak whichdefined that the hydration continued within the mixture and some MgO was left as being unconsumed [35].

Besides, following examination was linked with the significance of $Mg(OH)_2$, seawater and olive mill wastewater for the purpose of reduction in the swelling potential and plasticity ofbentonite clay. The different dosages of these substances were given into the bentonite soil andseveral tests examining the mineralogical or geotechnical aspects of mixture were made. In other saying, consistency, bearing capacity, compaction and swelling pressure tests with XRD analysis were caried out to realize their impacts on the bentonite. Therefore, it was founded thatthe swelling potential and plasticity of bentonite was affected by mineralogical arrangement. Montmorillonite quantity was decreased by the adding of those substances because of the cation exchange ability from cation introduction.

Moreover, three additives diminish the soil swellingpressure and the plasticity index because several conditions of complex such as pH, presence ofvarious cations were changed. The amount of change was between 60-87% as percentage. Regarding mechanical properties of complex, these ingredients caused an increase in the maximum dry density and California Bearing Ratio ofbentonite soil, but optimum water content after addition was less than the one of original soil [36].

5. MATERIALS AND METHOD

5.1 Materials

5.1.1. General

In this study, properties of materials had crucial role to analyse liquefaction occurence in the site. Regarding sand, parameters are the primary indicator of its susceptibility. For this reason, based on previous considerations mentioned in liquefaction susceptibility of soil part, the sand was chosen carefully and the necessary checks were done. In respect to magnesium oxide, only one type of chemical was selected which will be specified in below, because the type of magnesium oxide used in this study had many advantages compared to other types generally utilized within several sectors. Furthermore, in terms of availability and convenience for production, selected type of magnesium oxide was more feasible.

5.1.2. Sand

Sand was taken from the region located on the shore of Black Sea at Karaburun which shows Karaburun-Ceylan formation . Then, it was sieved into the size between 0.420 mm (No.40) to 0.074 mm (No.200) so that uniform fine sand can be obtained according to Wentworth size class and ASTM Mesh size. Afterwards, necessary control about whether the gradation left in the interval of the gradation limits for liquefable soils or not was checked.

General and state properties of sand were determined through Specific Gravity by Pycnometer Test by ASTM D854 B, Determination of Dry Density and Void Ratio by ASTM D7481-18, Maximum and Minimum Void Ratio with Relative Density Calculation by ASTM D4254-16M, Grain Size Distribution by Sieve Test by ASTM D422. Corresponding results were presented as seen in Table 5.1 and Figure 5.1.

Table 5.1. General and state properties of pure sand.

Soil Properties	**Results**
Dry Density (g/cm^3)	1,382
Specific Gravity (G_s)	2,621
Void Ratio of Soil (e)*	0,895
Maximum Void Ratio (e_{max})	0,941
Minimum Void Ratio (e_{min})	0,773
Relative Density (D_r) (%)	27
Saturated Water Content (%)	34.18
Coefficient of Uniformity (C_u)	1,95
Coefficient of Conformity (C_c)	0,97
USCS Classification of Soil	SP
* : These are state properties, which are influenced by storage, compaction and transportation	

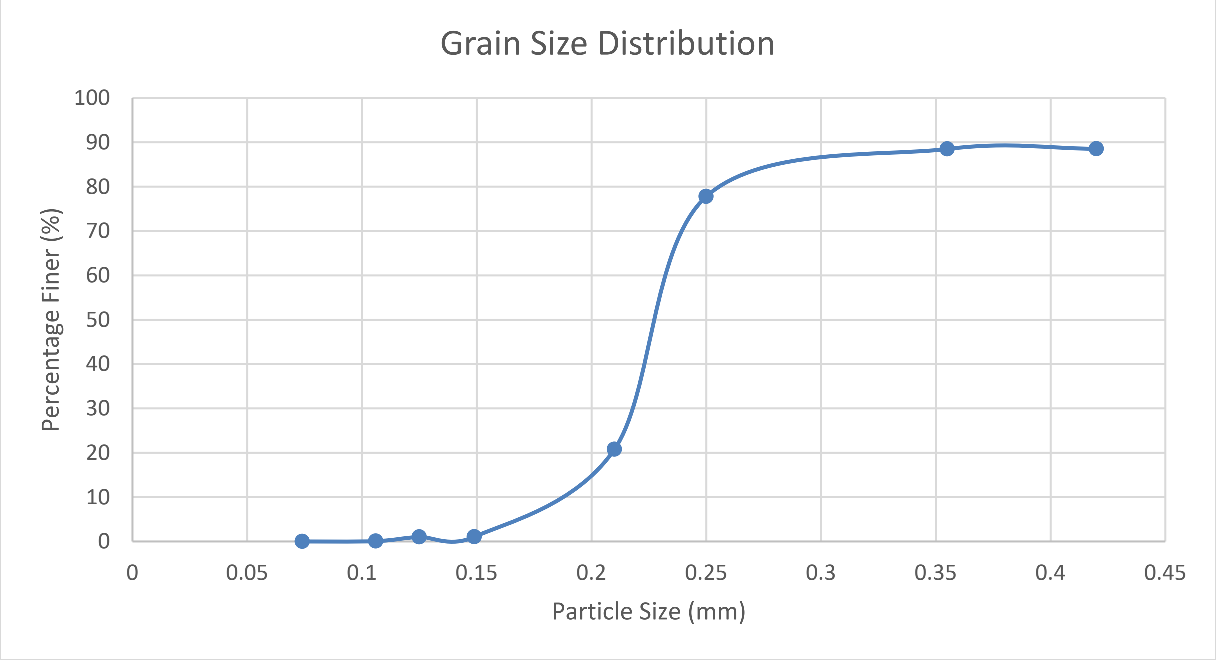


Figure 5.1. Grain size distribution of clean sand.

In the following of determination of basic properties of clean sand, Unconfined Compressive Strength test based on the procedures of ASTM D2166M-16 was caried out for treated sand samples.

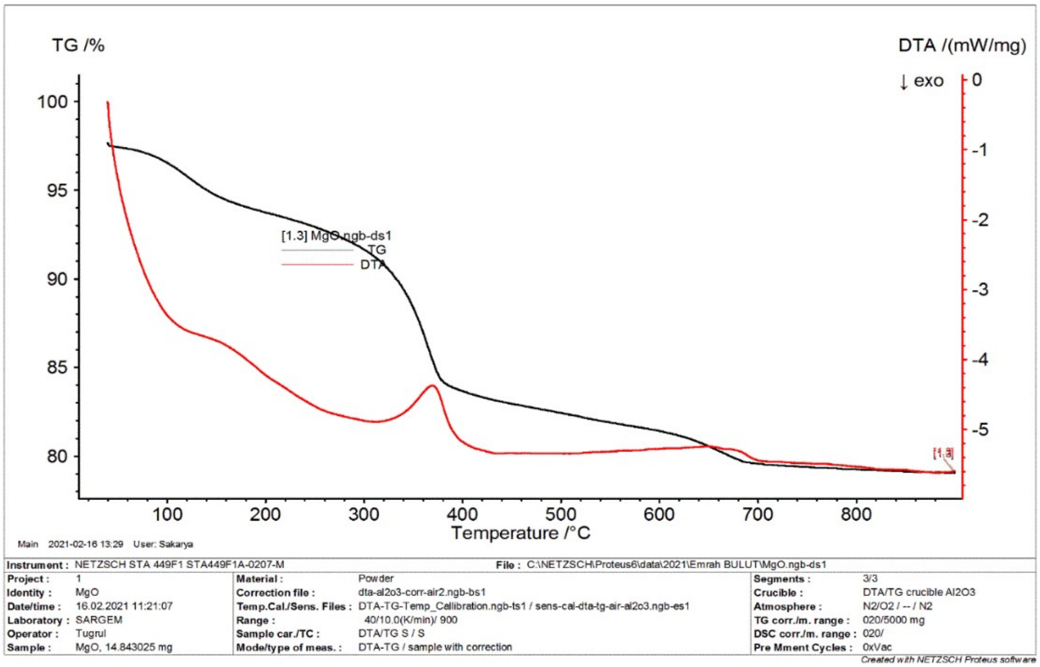
5.1.3. Magnesium Oxide

Magnesium oxide, was supplied in white-powder form by OSA Dış Ticaret Company,Istanbul. There were also experiments applied into the magnesium oxide for the determination of its chemical and physical test. Among these, XRF Test for Chemical Composition, BET Surface Area Analysis and Thermogravimetric Analysis were implemented and corresponding outcomes were stated in Table 5.2. Accordingly, chemical composition, apperance, average particle size, type and calcination temperature of MgO were provided by suppliers BET Surface Area Analysis and Thermogravimetric Analysis were performed by SARGEM at Sakarya University.

Table 5.2. Chemical composition of MgO material by XRF Analysis obtained from OSA Dış Ticaret Ltd. Şti.

Chemical Composition	**Percentage (% by weight)**
MgO	88.19
SiO_2	5.28
CaO	2.61
Fe_2O_3	0.68
Loss on Ignition	3.04
Physical Properties	**Value**
Specific Surface Area (m^2/g)	18.33
Density (g/cm^3)	3.58
Appereance	White Powder
Average Particle Size (μ)	90/100
Melting Point (^oC)	2800
Boiling Point (^oC)	3600
Type	Hard-Burnt
Calcination Temperature Range (^oC)	1000-1500

Regarding TG Analysis, NETZSCH STA 449F1 instrument was used to conduct the thermogravimetric analysis for 15 mg MgO sample under the temperature increment, 3 ºC per minutes, based on the regulations of ASTM E1131–20. Through increment in temperature and loss in weight (%), chemical and physical features of magnesium oxide could be altered because of occurrence of several reactions under changing temperature condition. From the graph in below, it should be stated that MgO powder has impurities as much as 18% after calcination. During temperature increment in MgO, there are evaporation of water adsorbed from surrounding atmosphere, transformation of possible $Mg(OH)_2$ into MgO and calcination of small content of magnesium carbonate, $MgCO_3$, which are happened in range of 50 ⁰ C to 700 ⁰ C. Beyond this, there is a steady position on the curve, meaning that crystallization and homogeneity within MgO starts. For this reason, majority of calcined material is MgO , even though some impurities due to incomplete calcination or storage at wet condition may exist. In Figure 5.2, the results of TG Analysis were presented. The necessary one was exhibited in black curve.



Instrument : NETZSCH STA 449F1 STA449F1A-0207-M
File : C:\NETZSCH\Proteus6\data\2021\Emrah BULUT\MgO.ngb-ds1

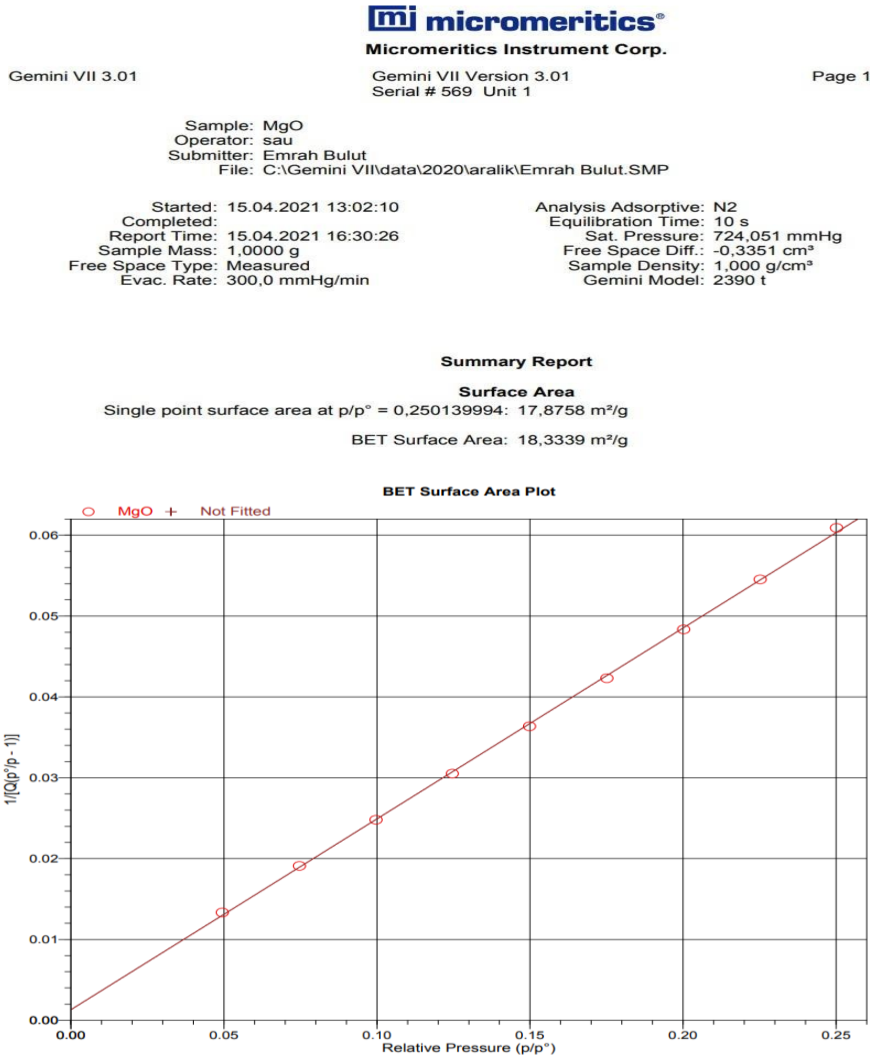
Project : 1
Identity : MgO
Date/time : 16.02.2021 11:21:07
Laboratory : SARGEM
Operator : Tugrul
Sample : MgO, 14.843025 mg

Material : Powder
Correction file : dta-al2o3-corr-air2.ngb-bs1
Temp.Cal./Sens. Files : DTA-TG-Temp_Callibration.ngb-ts1 / sens-cal-dta-tg-air-al2o3.ngb-es1
Range : 40/10.0(K/min)/ 900
Sample car./TC : DTA/TG S / S
Mode/type of meas. : DTA-TG / sample with correction

Segments : 3/3
Crucible : DTA/TG crucible Al2O3
Atmosphere : N2/O2 / -- / N2
TG corr./m. range : 020/5000 mg
DSC corr./m. range : 020/
Pre Mment Cycles : 0xVac

Figure 5.2. The result of Thermogravimetric Analysis on MgO product

For specific surface area determination by Brunauer-Emmet-Teller (BET) Analysis, the model and properties of instrument was Micromeritics Corp. Gemini VII 2390 t. This method involves the utilization of nitrogen gas at boiling point for adsorption purpose on the solid surface due to weak chemical bonds and determination of the amount of gas adsorption on the surface of powder sample through a continual or volumetric flow mechanism. The procedure is done according to the standard of BS ISO 9277: 2010. In this study, mass of sample for BET analysis was determined as 1000 mg and corresponding result was presented in Figure 5.3.



micromeritics®
Micromeritics Instrument Corp.

Gemini VII 3.01
Gemini VII Version 3.01
Serial # 569 Unit 1
Page 1

Sample: MgO
Operator: sau
Submitter: Emrah Bulut
File: C:\Gemini VII\data\2020\aralik\Emrah Bulut.SMP

Started: 15.04.2021 13:02:10
Completed:
Report Time: 15.04.2021 16:30:26
Sample Mass: 1,0000 g
Free Space Type: Measured
Evac. Rate: 300,0 mmHg/min

Analysis Adsorptive: N2
Equilibration Time: 10 s
Sat. Pressure: 724,051 mmHg
Free Space Diff.: -0,3351 cm³
Sample Density: 1,000 g/cm³
Gemini Model: 2390 t

Summary Report

Surface Area

Single point surface area at p/p° = 0,250139994: 17,8758 m²/g

BET Surface Area: 18,3339 m²/g

BET Surface Area Plot

Figure 5.3. The result of BET Analysis for MgO.

In the light of information expressed in previous parts, the range of calcination temperature for magnesium oxide having such specific surface area resulted in above is between 1000 ⁰C to 1500 ⁰C. Therefore, it could be concluded that the type of MgO is hard burned.

5.2 Methods

5.2.1. Unconfined Compressive Strength Test

5.2.1.1. General. Unconfined Compression Test is a laboratory experiment to evaluate the mechanical features of rocks and cohesive soils. It determines the undrained shear strength and the stress-strain behaviour of the rock or soil. The unconfined compression test is generally involved in the laboratory experiment software of geotechnical explorations, actually when conducting on the rocks.

5.2.1.2. Aims and Procedure. The main aim of the UCT is to rapidly find out the unconfined compressive strength of cohesive soil or rocks which has adequate cohesion to allow experiment in the unconfined state. The finding is later utilized to determine the unconsolidated undrained shear strength of the clay under unconfined cases. The UCT mostly could be implemented on rock samples or on undisturbed or compressed cohesive soils. In this experiment, the specimen in the surcharge machine between the bottom and top plates. Before beginning the surcharging, the top plate is fixed to be close to the sample as much as possible and the deflection is arranged as zero. The experiment begins by exerting a same axial strain of nearly 0.5 to 2 % per minute. The load and deflection values are noted as required for determining a full of load-deflection curve. The loading is proceeded until load measure reduces or it is a steady with rising strain, or the axial strain becomes 20 %. At this condition, the samples are thought to be at failure. Next, specimen is taken for the water content determination. In the results, the uniaxial stress versus strain is generally drawn and the maximum point on the stress axis over the curve or the stress point at 20% strain value if the curve is like straight-line is obtained as unconfined compressive strength, qu.

By this way, undrained shear strength for cohesive soil could be calculated in Eq. 5.1 as follows [38]

$$S_u = q_u/2 \tag{5.1}$$

where

S_u = Undrained Shear Strength,

q_u = Unconfined Compressive Strength.

5.2.2. Friction Angle of Soil (φ)

Friction angle of soil is the one of parameter to determine shear strength of soils. It is obtained from Mohr- Coulomb failure criterion and it exhibits the frictional resistance of soil along with normal effective stress against shearing. In shear stress- normal effective stress plane, the friction angle of soil is the angle of slope of Mohr-Coulomb envelope [39].

5.2.2.1. Critical Friction Angle. The shear strength of sand is the total of the critical and dilatancy case parts. The critical condition part is described by the critical state friction angle. It is not dependent to the deformation behaviour and stress degree. For clean sands, critical state friction angle changes between 30^0 to 36^0 based on gradation and particular shape. When loose sand is contracted during shearing, it has maximum strength at critical state. On the other hand, dense sand shows maximum strength at failure together with great dilatancy and lowest strength at the ultimate state [40].

5.2.3. General Purposes

In this work, the liquefaction susceptibility and the mechanical response of clean fine liquefiable uniform sand improved with magnesium oxide aganist uniaxial static loading was investigated.

For sample preparation, the dry mixing of soil and binder was performed along with the addition into water through wet-pluviation method instead of injection of binder in slurry formation. The reason is due to some drawbacks for monitoring of slurry concentration,which is very affectable from various conditions. Accordingly, wet pluviation method was used to prepare loose soil sample to prevent segregation. To check degree of improvement aganist liquefaction, some physical properties of treated samples like shape, water content, dimensions were investigated and then, resistance of treated samples having different MgO dosages under curing periods to anisotropic stress was determined together with deformation outputs over Unconfined Compression Test. Likewise, for UCT machinery, ELE-Digital Tritest 50 model was used along with related equipments like load dial gauge, base plate and top cap placed in Karl Terzaghi Soil Mechaincs Laboratory in Bogazici University in Istanbul. The reason for the use of the Unconfined Compression Test was to determine compressive strength of loose saturated liquefiable soil without any confining condition. By the way, such a loose fully saturated soil had cyindirical block like-shape after improvement depending on MgO level and curing condition. During application of loading, the degree of deformation of treated samples was measured.

Since effect of liquefaction was increased by the depth, such as 1 m, closer to ground surface, confining pressure was very small and it was important to conserve soil's loose state and saturation. Therefore, confining effects was removed and it was necessary to pour soil-MgO mixture through pluviation method to keep the loose state into the certain amount of water, where saturation of soil was completely provided.

Plus, from preliminary test, it was known that magnesium oxide took water molecules to generate solid hydration precipitates. At the end, cylindirical block shape samples were possibly obtained with low or zero water content along with negligible permeability after soil, MgO and water elements were mixed. Depending on this, UCT rather than UU Triaxial Testing should be selected to conceive actual response of treated soil against unixial loading. Even though UU Triaxial test did not contain consolidation, cell pressure, nevertheless, affected to stress-condition of soil.

Thanks to removal of confining pressure, the influence of MgO on the soil prepared very loose under fully saturated condition could have been observed more realistically. Moreover, Unconfined Compression Test was more suitable to start Static Liquefaction due to type of application of loading, eccentricity and existence of confining pressure. As the reason, Static Liquefaction was triggered just a change in stress condition of loose soil under fully saturation.

In spite of MgO content, effect of curing periods was also included. At the end, the water content test, unconfined compressive strength, known as maximum compressive stress from dial gauge after calibration, and axial failure and end strains for all treated samples were obtained through balance, oven and UCT apparatus such as load dial gauge. Then, further conclusion and review could have been made regarding Young's Modulus, stiffness and durability for improved specimens. By this way, the influence of chemical and curing periods on the pyshical, hydrological and mechanical performance of intially liquefiable loose saturated sand were clearly learnt under condition without compaction but mixing and comment regarding how much MgO was appropriate to remove triggering factors of Static liquefaction was made.

5.2.4 Procedures of Sample Preparation and Testing

The steps of sample preparation for testing were listed below.

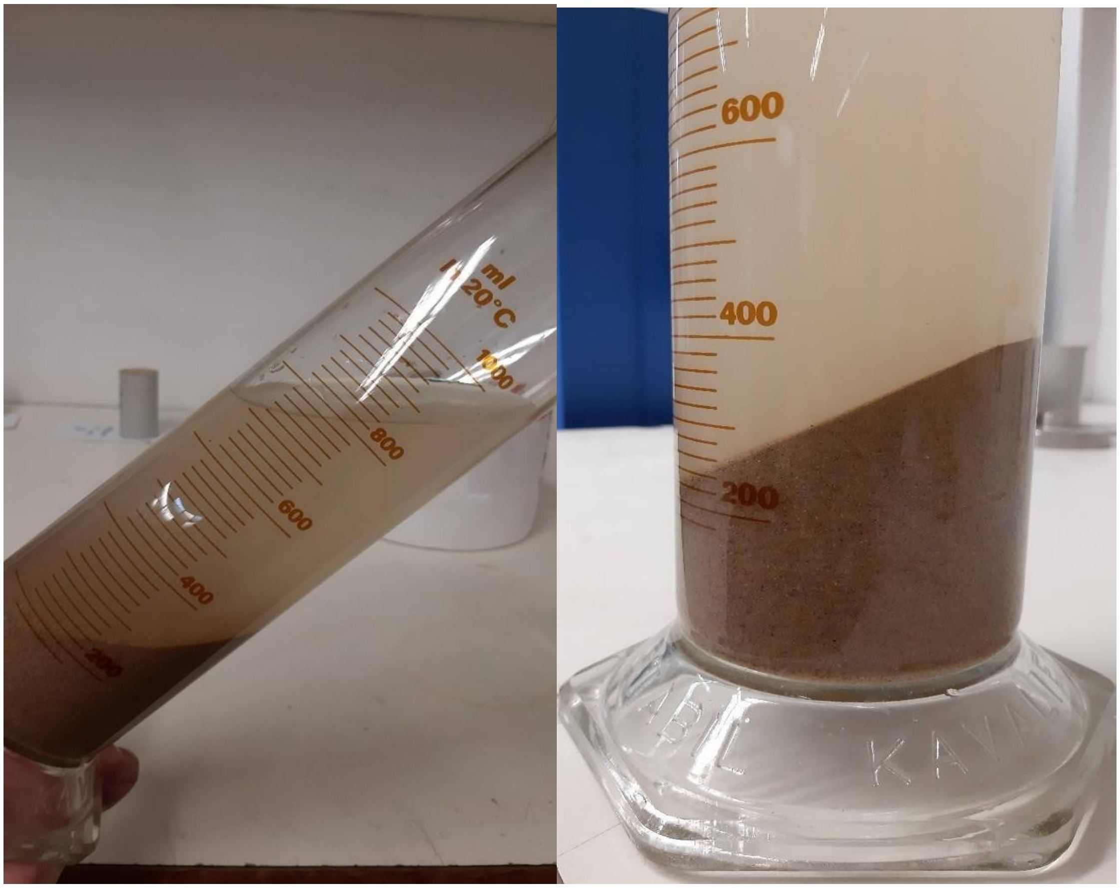
- Sand was sieved by 0.420 mm to 0.074 mm by corresponding mesh types based on ASTM D422 standart shown in Figure 5.4. According to this, coefficient of conformity and uniformity along with UCSC classification and the domestic group of size in the soil were determined.

a)

b)

Figure 5.4. Grain Size Distribution Test for pure sand : a) Sieving , b) Weighing.

- The critical friction angle was measured based on Simplified Method [41]. In his method, soil was loosely poured by using wet-pluviation method into 1 L water at graduated cylinder. Then, cylinder was rotated about 60^{0} and gradually reversed into previous status. The slope was formed as diagonally. When the angle from the midpoint of the slope was determined, the critical friction angle of sand was obtained. By this way, critical friction angle of sand was determined as 31^{0} as degree after 7 trials as shown in Figure 5.5.



a) b)

Figure 5.5. Determination of critical friction angle for pure sand by Simplified Method : a)Rotating 60^{0} angle, b) Friction Angle Determination [41].

- Specific gravity (Gs) test by pcynometer was done for the 50 g clean sand sample according to procedures from ASTM D854 B. After 3 trials, specific gravity was determined as 2.62 as value as presented in Figure 5.6.


a)
b)

Figure 5.6. The Specific Gravity Test for pure Sand by using pycnometer : a) Weighing, b)Vacuuming.

- Certain amount of clean sand were taken and dried in the oven at 105 ^{0}C for 5 hour. By this way, water was removed completely.
- Next, dry sand was pluviated into plastic mould by using the funnel at certain height with spiral motion from sides to center so that segregation and agglomeration of soil was prevented and loose state was preserved at best.

- As similar to the process of Maximum Minimum Void Ratio Determination Test during finding e,max, sand was trimmed followed by obtaining soil weight as 400.84 g after several pouring, trimming and weighing trials as seen in Figure 5.7.


a) b)

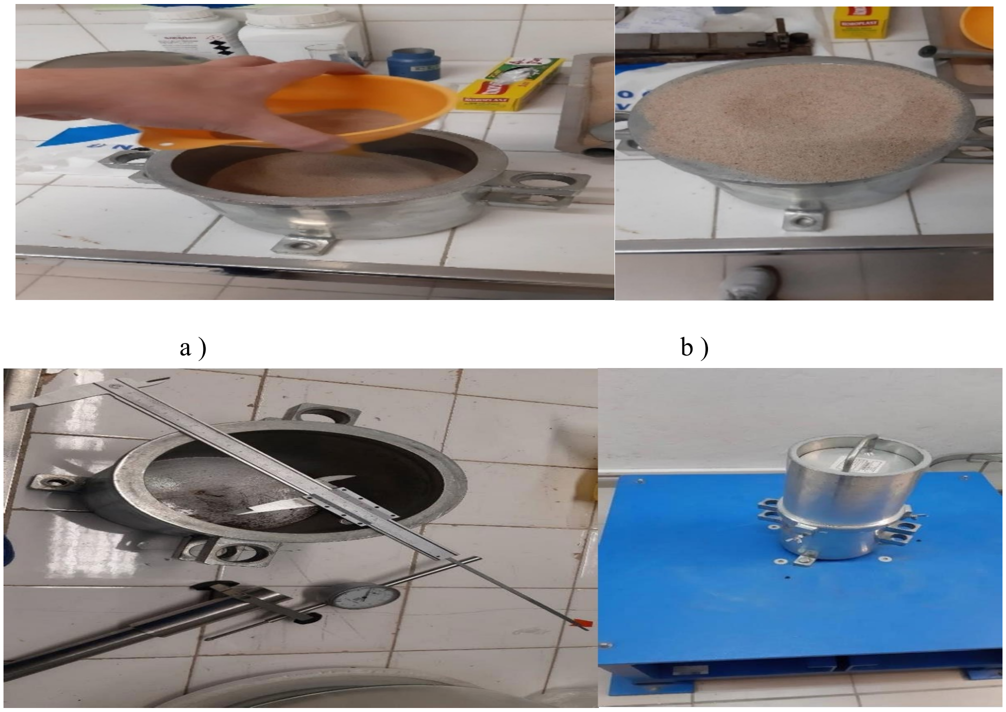
Figure 5.7. The determination of bulk density and void ratio under laboratory conditions : a) Filling the plastic mould with dry sand completely, b) Trimming of excess dry soil.

- Mould volume was measured by being filled with distilled water whose density was calculated as 1. At the end, mould's volume was founded as 290 mL.
- By dividing soil weight into mould volume, Dry Bulk density (ρ_{dry}) sand was calculated.

- After obtaining Dry Bulk density (ρ_{dry}) and Specific Gravity (G_S), void ratio at laboratory condition for 400.84 g sand (e_0) was determined as 0.8955, expressed in the Eq. 5.2 as follows

$$\rho_{dry} = \frac{Gs \times \rho,w}{1 + e,o} . \qquad (5.2)$$

- Then, maximum and minimum void ratio as denoted by e_{max} and e_{min}, respectively, were determined by Relative Density Calculation Test having procedures according to ASTM D4254 as shown in Figure 5.8.
- During conducting the test, funnel and surcharge apparatus were utilized to obtain the soil at loosest or densest state.
- Through the maximum and minimum density calculation, those indexes were founded.



a) b)

c) d)

Figure 5.8. Relative Density Test : a) Pouring soil by funnel, b) Filling the mould, c) Test apparatus, d) Vibrating the loose sand.

- As a consequence of these, relative density (Dr) of clean sand in this study was determined as 27 % , as given by

$$D_r\,\% = \left(\frac{e_{,max}-e_{,o}}{(e_{,max}-e_{,min})}\right) \times 100. \tag{5.3}$$

- As a result of these, it was checked whether clean sand used in this study was susceptible to liquefaction or not.
- To do that, grain size distribution, UCSC Classification, coefficient of uniformity and conformity, relative density (D_r), some physical properties mentioned in previous parts and critical friction angle were used to categorize the soil.
- Afterwards, based on proposed typical limits, clean sand in this study was loose, uniform, fine, poorly graded, non-cohesive, marine deposited and rounded type.
- By this way, it was observed that the soil in this study was liquefaction susceptible one having the gradation curve left between the gradation limits of most-liquefiable soils.
- In the following of Specific Gravity (G_s) and void ratio (e) under laboratory condition, by giving degree of saturation (S_r) to “1” , water content (W) for saturation was obtained as

$$e \times Sr = Gs \times w\,. \tag{5.4}$$

- After finding water content as 34.18%, it was controlled whether or not soil was saturated under these conditions. Thus, it was used 400.84 g soil and 137 mL water during sample preparation.

- As seen in Figure 5.9, 400.84 g soil was pluviated into the mould containing 137 mL water by funnel at certain height where rotation techqniue as void ratio determination part was utilized similar to in previous part.

Figure 5.9. Saturation check for 400.84 g dry sand pluviated into 137 mL water within plastic cylindirical mould.

- MgO powder was blended and sieved into the particle size ranging between 0.09-0.10 mm as seen in Figure 5.10.

a)　　b)

Figure 5.10. The preparing of MgO to the experiment : a) Blending of MgO by mixer, b) Sieving of MgO with the size between 0.125 to 0.074 mm.

- Next, MgO powder was placed into oven and dried for 5 hours at 105 ºC to prevent the content of water adsorption from environment during transportation and storage of MgO powders.
- Based on soil mass (400.84 g), MgO chemicals was added in the proportions from 0.5% to 6% as percentage as seen in Figure 5.11. As specifically, contents of MgO were sorted as 0.5%, 1%, 2%, 4% ,6% of dry mass of soil. The amount of magnesium oxide were 2, 4, 8, 16, 24 g, as respectively.

Figure 5.11. Dry Mixture of Sand-MgO before blending.

- After mixing of certain soil mass with MgO portions under dry condition through mechanical mixer Kitchenart Planetary mixer with Artisan Kitchenaid 5KSM7580XEER Model under maximum rate for 10 minutes, homogenous distribution of magnesium oxide fractions within sand mass and homogenous dry blends were obtained as presented in Figure 5.12.

Figure 5.12. Dry mixture of sand and magnesium oxide after 10 minutes blending.

- Afterwards, 137 mL of water was poured into the plastic mould and certain amount of dry sand-magnesium oxide mixtures was pluviated into the water by using funnel by using wet-pluviation method as shown in Figure 5.13. During pouring dry mixture into water, funnel was rotated with sprial motion from sides to center and by this way, segregation and agglomeration within dry components was prevented as conducted previous parts(void ratio determination part).

Figure 5.13. Preparation of compontents and pluviation of sand-MgO blends into 137 mL water by funnel at certain height with spiral motion.

- After pouring of dry blends into water, it was checked whether or not soil was at loose state under completely saturation condition and it was realized that the size of plastic mould took all compontents of samples without overbrimming as seen in Figure 5.14. Therefore, remoulded water contents was equal to 34.18% for all samples.

Figure 5.14. The moulding of samples after pluviation of dry blends into water and check for fully- saturation, excess components and remoulded water content.

- Next, samples within cylindirical plastic mould having 50 mm diameter and 110 mm height under fully saturated condition were prepared by using wet-pluviation method to keep loose state of soil as much possible without compaction which causes deterioriation and unsedired breakage within microparticular structures of specimens. Therefore, keeping soil at loose state was main aim in this study.
- By this way, the influence of MgO on the mechanical properties of loose soils would be observed more clearly without changing state of soil through compaction or mixing with stirrer.
- As in Figure 5.15, all samples were waited for 2 days to extract from the mould and cured for 3, 7 and 14 days under 23 ºC temperature and 41% humidity. It had an aim to investigate the effect of time on the improvement due to the reason that hydration reactions between MgO and sand needs certain time for completion.

Figure 5.15. Samples cured for 3,7 and 14 days under laboratory condition after extracted from mould after 2 day.

- After certain curing period, top of all samples which did not have planar and smooth surface was cutted in order to make cylindirical samples appropriate for Unconfined Compression Testing. By this way, cylindirical block-shaped samples with planar and smooth surface were obtained.
- Then, diameter and length of samples were gauged with straightedge to check length to diameter ratio which should be between 2 and 2.5 based on ASTM Standarts as seen in Figure 5.16. Also, all samples were weighed before testing.

Figure 5.16. Measurement of dimension of improved samples after curing periods.

- Under all curing conditions, all treated samples taken from mould and trimmed from top had 55 mm diameter and 120 mm length which was between 2 and 2.5 regarding length to diameter ratio.
- Before implementation of unconfined compression testing, calibration for dial gauge was needed. Accordingly, dial gauge was extracted from the proving ring and loads in N were put upon the dial gauge sensor so as to be incremented step by step as seen in Figure 5.17.

- Meanwhile, the change in dial gauge was noted according to degree of loadings, so logical correlation between numbers in dial gauge and level of surcharging in N as unit was made in order to make an essential transformation from dial gauge numbers to loading for following steps.

Figure 5.17. Calibration of Load Gauge by putting surcharges with increasing rate.

- Treated soil samples were inserted upon the bottom plane seating over moving plate. After the cylindirical sample block was positioned with respect to loading cell sensor fixed with conical seating.
- Then, top cap with concave space at the center of upper surface and planar shape at bottom was placed above sample.

- Through the upward movement of bottom moving plate with the use of button, top cap and loading cell sensor were approached to each other. After very small gap between them was left, they were adjusted such that fixed loading sensor with conical seating filled to concave space completely based on the middle of top cap.
- Similarly, cylindirical samples were positioned such that centers of bottom plane seating, sample block and concave space at top cap were at the same line as shown in Figure 5.18. The reason for positionning was to provide centricity during axial loading which would reflect maximum response of specimen aganist one-directional compression.

a) b) c)

Figure 5.18. Placement of specimen into LDVT Frame : a) 0.5% MgO Sample for 7 days, b) 2% MgO Sample for 3-days, c) 4% MgO Sample for 14-days.

- Then, “Run” button to start an experiment was pushed and base plate was moved upward to compress to treated specimens at the rate of 0.7 mm per minutes under strain-controlled condition and Unconfined Compression Test was conducted on all treated specimens based on procedures of ASTM D2166M-16.
- During mobilization of bottom plate, uniaxial compression upon the sample was carried out. Testing was proceeded untill a reduction within load dial gauge was observed as.
- During failure proceeding, due to Poison’s effect, the samples were failed as with 45^0 slip surface, vertical slip surface or no slip surface which were spreaded from the top to bottom of the specimen as shown in Figure 5.19.
- At the end, some samples were just cut, whereas some samples with more solid structure were broken off a large particles of the specimen.

a) b) c)

Figure 5.19. The shape of slip surface within the fractured samples after failure : a) 3-days waited 2%, b) 7-days waited 0.5%, c) 14-days waited 4% Sample.

- Afterwards, small pieces from specimens after failure were taken as shown in Figure 5.20. By this way, water content were calculated.

Figure 5.20. Determination of water content of samples after experiment.

- Since the tesing was implemented under strain-controlled condition, deformation of the specimen during loading was calculated based on time by using strain rate per every minute. Also, corrected area was calculated for each strain data.
- Loadings applied on the specimens were obtained through loading dial gauge and measurements were noted in order to convert force (in N as unit) by using calibration correlations determined in previous step.
- Then, unconfined compressive strength in kPA as unit was obtained through dividing corrected maximum force in N as unit after conversion for maximum point taken at dial gauge into the cross-sectional area of the specimen followed by considerable reduction.
- At this point, time lasted was noted in minutes and deformation was determined by multiplying of strain rate per minute with noted time at failure point. Then, through the length of specimen, strain at failure point was figured out.
- At the end, stress unit was converted to stress in kPA as unit and strain values were turned to percentage according to measures and the mechanical response of treated specimens against uniaxial loadings was handled along with the stress-strain curves and stiffness determination.

5.2.5 Classification of Samples and Outline of Testing

As the repetition, Unconfined Compression tests were performed as 15 times for improved sand treated under three curing conditions. For the sake of reliability, samples were triplicated and stress-strain outputs were averaged for each sample cured for different periods.

Additionally, the liquefiable pure sand layer was designed till its depth became 1 m from ground surface and shear strength for fully saturated pure sand was calculated through the critical friction angle, saturated and dry bulk density of pure sand and effective vertical stress at the average depth of this layer as 0.5 m. By this way, the comparison was made and improvement percentage could be seen. As in Table 5.3, the classification was done for the label of improved sand samples.

Table 5.3. The labelling of sample for Unconfined Compression Test.

Name Of Sample	**MgO Content (%)**	**Curing Periods (days)**	**Remoulded Water Content (%)**
MGO 0.5% /A-B-C	0.5 %	3-7-14	34.18
MGO 1% /A-B-C	1 %	3-7-14	34.01
MGO 2% /A-B-C	2%	3-7-14	33.78
MGO 4% /A-B-C	4 %	3-7-14	33.24
MGO 6% /A-B-C	6 %	3-7-14	32.88

6. DISCUSSION AND RESULTS

As mentioned before, liquefaction occurred in loose saturated liquefiable clean sand has the mechanism where the effective soil stress was totally or partially lost after the generation of excess porewater pressure caused by undrained loading within the soil. Depending on the equality between porewater pressure and surrounding stress, shear resistance of soil is reduced partially or completely based on the level of effective soil stress. In case of completely loss of strength, soil behaves as liquid, and it is not able to carry any external load anymore. In case of partial loss of resistance, liquefied soil has very low shear resistance, but a soil is poses to huge axial strains, led to significant damages.

To sum up, when steady-state shear resistance becomes smaller than external shear stress, liquefaction may be triggered. Therefore, to improve liquefiable soil mechanically, magnesium oxide chemical was mixed with fully saturated loose sand under several doses of 0.5%, 1% ,2%, 4%, 6% of dry weight of soil and mixture were molded up to have block shape.

To evaluate the role of magnesium oxide on the behavior of liquefiable uniform fine sand, unconfined compression tests (UCTs) for improved samples treated under 3, 7 and 14 days of curing were uniaxially conducted as 45 times for treated soil. During uniaxial loading, stress condition of fully saturated loose sand may be changed, resulted with upward movement of porewater within complex known which triggers static liquefaction. Even though fully saturated sand under Unconfined Compression Test had drained condition, slow changes within stress condition under strain controlled cases would be sudden enough to raise porewater pressure, which was a key factor for static liquefaction occurrence.

Main reason for utilizing UCT instead of Triaxial Tesing was removal of confining effect and thus, the influence of cementation of MgO on the loose sand,which is risky for liquefaction, would be observed more clearly.

As the mentioned, uniaxial compression on saturated loose sand could be sufficient to start the static liquefaction even if soil was under drained condition due to sudden stress variance during loading.

Likewise, UCT was an important indicator to evaluate mechanical properties of cemented-like soil blocks without confining effect. Through UCT, unconfined compressive strength was determined by finding maximum compressive stress on the loading dial gauge and at this point, failure point was reached followed by development of vertical and diagonal fractures in cylindirical specimens. After failure point, compressive stress was reduced.

On the other hand, there was no any possibility for conversion of UCS values to undrained shear strength in this study, because the material used in this study was sand and the water content of samples was much more under the saturation. Therefore, drained condition existed due to the UCT system like in case of the usage of sandy soil.

Furthermore, undrained condition would be not possible in the absence of water. Under these circumstances, Unconfined Compressive Strength was only element to give mechanical response of treated soil in this study.

In this study, UCT was caried out under 0.7 mm/min controlled strain-rate condition. The datas on loading dial gauge were transformed into loading through calibration algorithm and through strain value, cross-sectional area of samples was adjusted into corrected area. Thus, compressive stress (as unit of kPA) at certain strain was calculated by dividing the loading to corrected area.

Regarding strain calculation, time in minutes was used and based on change in loading gauge, specific moments were defined in order to multiply with constant deformation rate in minutes. At the end, the strain values were determined by dividing the deformation into the length of specimens. Accordingly, stress (kpA)- strain (%) curves were obtained after accumulation of all datas for samples having different MgO contents cured under different periods.

From stress datas, Unconfined Compressive Strength was identified where strain was defined as “strain at failure”. By dividing UCS into strain values at failure, Young’s Modulus was handled in order to make a conclusion about the stiffness of cemented sand samples.

Based on this, durability against continous uniaxial loadings along with the level of liquefaction occurrence under sudden stress change would be clearly seen by looking outputs such as UCS (q_u), strain at failure, strain at the end and water content after curing periods. In addition to this, water content after 7 and 14 days of curing became nearly zero except the ones cured for 3 days. Nevertheless, the shape of saturated uniform sand after treatment became cemented blocked-like cylindirical solid.

Corresponding results were presented by the determination of frictional angle of pure sand at critical condition and checks for gradation and relative density of pure sand along with unconfined compressive strength were presented in the followings.

Firstly, liquefaction susceptibility of pure sand was made. According to datas mentioned in Liquefaction Susceptibility part previously stated, pure sand in this study was labeled as “loose” type as classification of relative density due to the properties given in corresponding outputs.

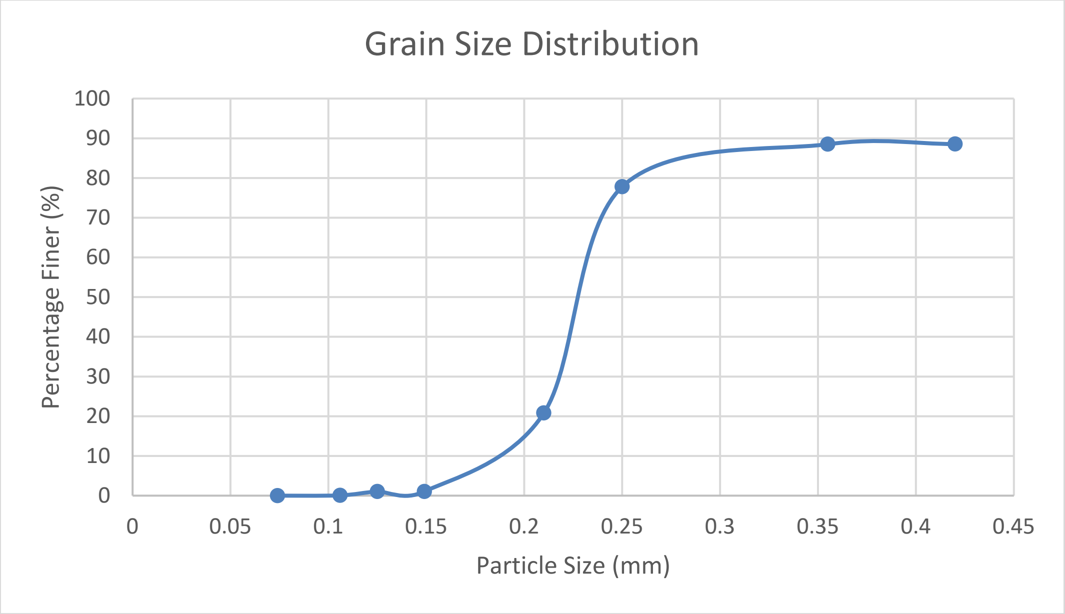


Figure 6.1. Grain Size Distribution of Pure Sand in This Study.

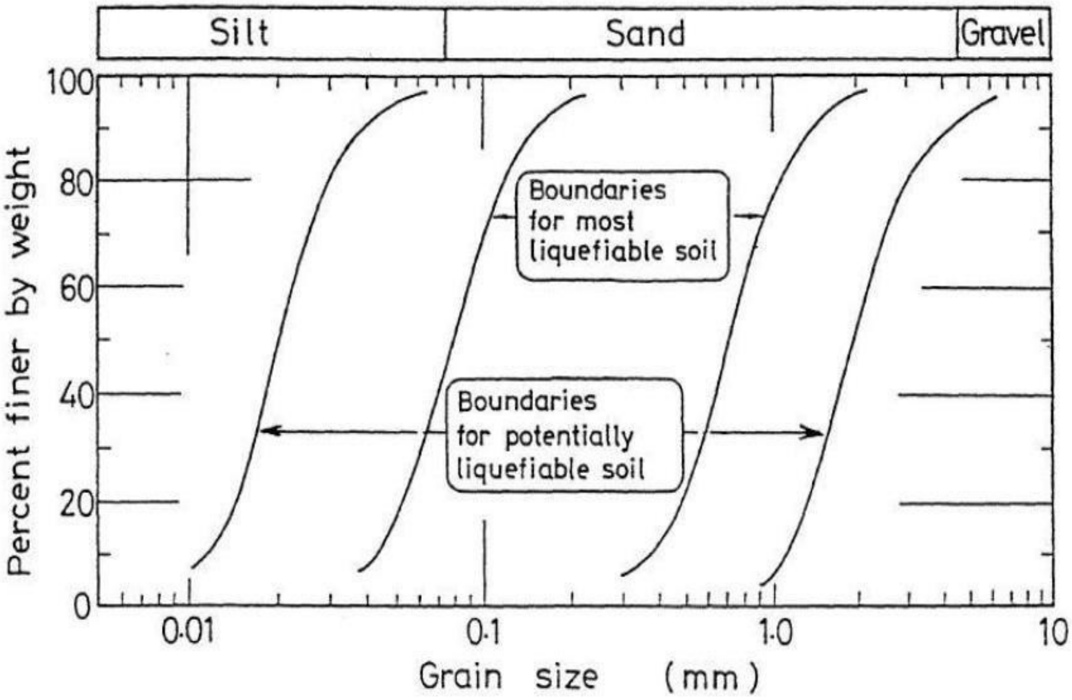


Figure 6.2. Limits in the gradation curves separating liquefiable and non-liquefiable soils [6].

According to the gradation values from Sieve Analysis shown in Figure 6.1, pure sand, in this study, left between gradation limit curves for most liquefiable soils given in Figure 6.2.

Pure sand gradation was Poorly Graded and Uniform type including generally fine sand particles with rounded and same size. Therefore, corresponding indexes taken from previous parts and friction angle of soil at critical state founded by Simplified Method mentioned in previous parts were determined as presented in Table 6.1.

Table 6.1. General values for pure sand parameters in this study.

Index of Pure Sand	**Value**
Relative Density (Dr %)	27
Deposition	Marine Deposit
Critical Friction Angle (^{0})	31
UCSC Classification	Uniformly Graded (SP)
Type of Soil	Clean Sand
Saturation (%)	100%
Shape	Rounded Particles
Pre-Condition	No Prior Loading and Shaking

As seen in Table 6.2 and Table 6.3, typical values of relative density and friction angle for sand classification were categorized for liquefaction susceptibility check of pure sand in this study.

Table 6.2. Classification of soil in terms of typical values of relative density and friction angle [42].

Classification	**Relative Density (%)**	**Angle of Friction (^{0})**
Very Loose	<20	<30
Loose	20-40	30-35
Compact	40-60	35-40
Dense	60-80	40-45
Very Dense	>80	>45

Table 6.3. Friction angle intervals for loose soil types with specific properties [39].

Properties	**Classification**	**Angle of Friction (^{0})**
Uniform Rounded Grains Clean Sand (SP)	Loose	>27
Well and Poorly Graded Clean Sand	Loose	29-30
Poorly Graded Sand with No Fines	Loose	30-39

Based on all information, pure sand, in this study, had relative density value as 27% and critical friction angle as 31^o. Also, it was stated in previous parts that critical friction angle for loose sand is changed from 30^o to 36^o. It is resulted by contraction during shearing and soil gives maximum shear strength at this state. Due to contraction behaviour, it is only observed in sands with loose formation [40]. In the light of this information and data, sand was categorized as loose type. For rest of them, pure sand conformed to all conditions for liquefaction susceptibility regarding gradation, size, shape, deposition and type of soil by comparing the datas from pure sand in this study with the features of liquefaction susceptible soils from previous part.

In the light of this information, it was known that loose type of sand had tendency for collapse. Therefore, vacuumed membrane, split-spoon mould and cell pressure should be needed to hold loose soil at cylindirical shape. This became harder when soil was saturated. Accordingly, saturated loose sample could be moulded out of Triaxial frame, but during demoulding, it was not possible to have cylindirical sample, which was suitable for Unconfined Compression specifications.

In this respect, it was decided to estimate shear strength of sand through critical friction angle and predetermined effective vertical stress after assumed circumstances. Firstly, it was known that critical friction angle of soil was 31^o and saturated unit weight was 18,18 kN/m^3 along with the unit weight of water, assumed as 10 kN/m^3.

Based on these, bouyant unit weight was founded as 8,18 kN/m^3 by extracting water unit weight from saturated unit weight. Since the liquefaction was very risky for fully saturated soil very near to ground surface, the maximum depth was taken 0.2 m. Therefore, to reflect whole layer, average depth was taken into account for shear strength calculation. From this, when average depth was multiplied with bouyant unit weight, effective vertical stress was found as 0,818 kPA.

Also, soil was clean fine sand with no cohesion, so shear strength was only dependent to critical friction angle and effective stress. Thus, shear strength of sand was multiplication between effective vertical stress and tangent of critical friction angle.

Based on this, shear strength of clean sand was determined as 0,49 kPA. Although saturated sand showed drained properties during Unconfined Compression Testing , transformation from undrained shear strength into UCS done for clay could have been used. Therefore, it was assumed that UCS for pure saturated sand was equal to 0,98 kPA as maximum at corresponding depth. Nevertheless, such smaller UCS value was under confining condition and real UCS value, if applicable, would have been much more smaller than that. At the end, pure sand's UCS was equal to 0.98 kpA.

Secondly, Unconfined Compressive Strength (UCS) values and correspoding strain values(%) for all treated samples under 3 days, 7 days and 14 days curing were gathered in Table 6.5 and Table 6.6 . As seen in Figure 6.3, Figure 6.4 and Figure 6.5, comparison between them was made.

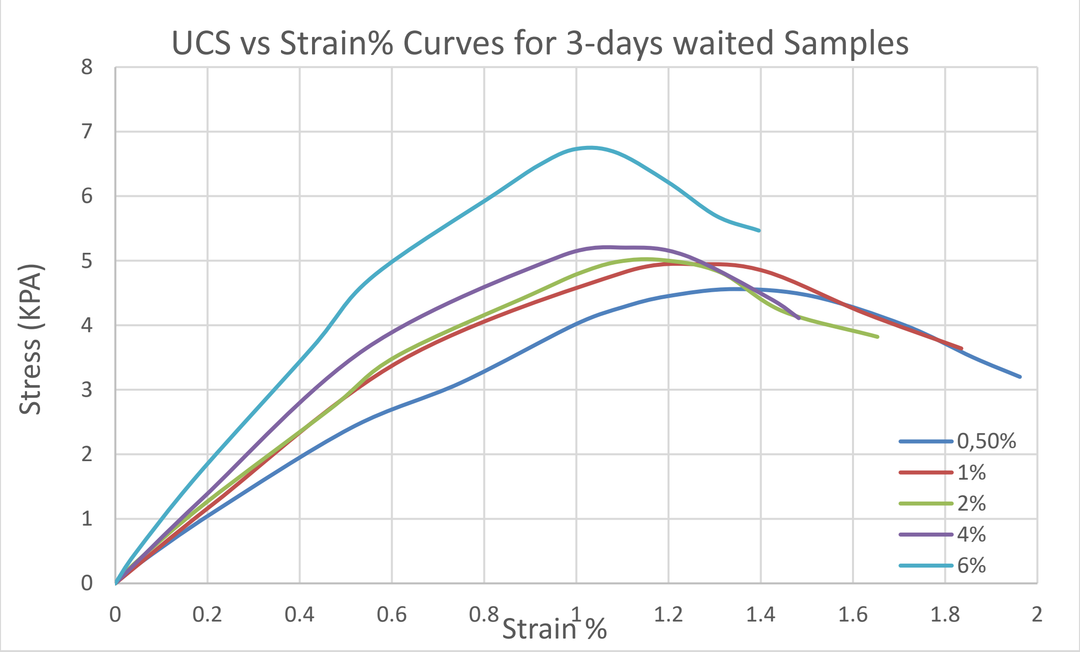


Figure 6.3. The stress-strain curves for 3-days waited samples.

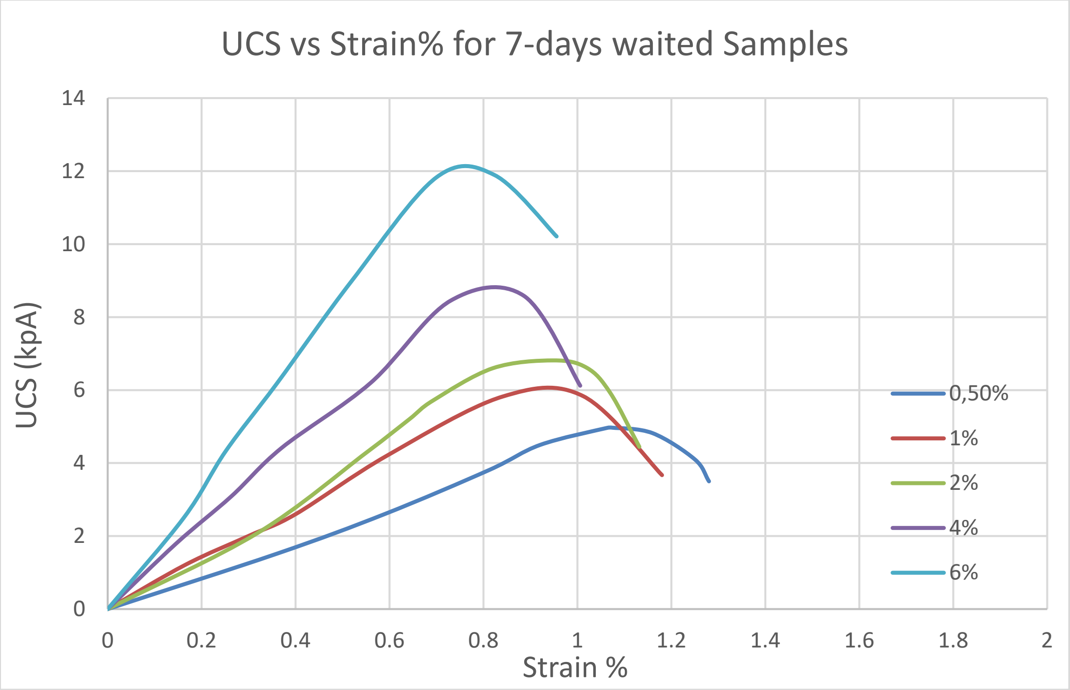


Figure 6.4. The stress-strain curves for 7-days waited samples.

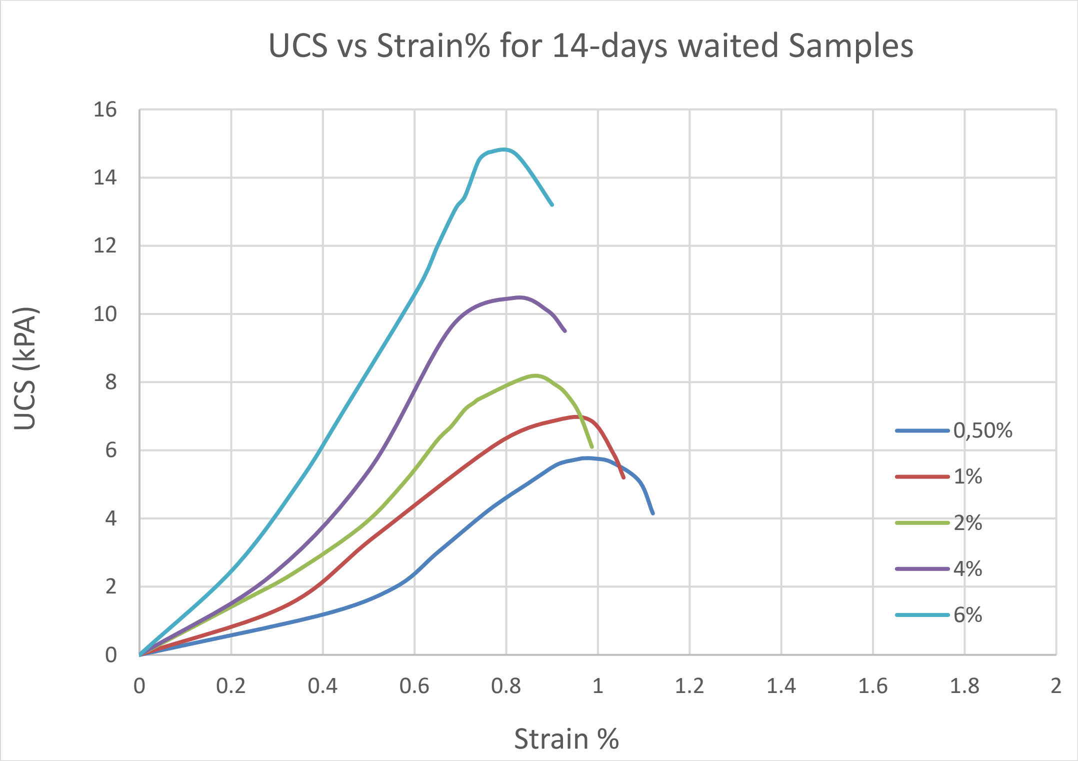


Figure 6.5. The stress-strain curves for 14-days waited samples.

Table 6.4. Water Content of Samples after for 3,7 and 14 days.

MgO Content (%)	**Water Content (%)**		
	3-DAYS	**7-DAYS**	**14-DAYS**
0,5	7,14	0,09	0
1	5,48	0,065	0
2	3,09	0,04	0
4	1,235	0,023	0
6	0,72	0,012	0

In Table 6.4 , addition of MgO and longer period of curing may have reduced water content into nearly zero level where treated samples had stable blocked-like shape except that of samples with MgO contents from 0.5% to 4% under 3-days of curing condition. From this point, it was realized that hydration reaction between MgO and water along with further ones between $Mg(OH)_2$ and adjacent chemicals were not ended properly. Thus, suitable cementation and demanded interconnection within system started to completed after 3 days, unless MgO content was excessive. Nonetheless, 6% MgO addition achieved to remove water content in the complex, but it was not enough to establish uniform,denser and integrated formation.

The extension of curing time supported the system for decreasing water content. This was apparent for samples cured under 7 and 14 days, which provided to nearly zero water content because further reactions and products between $Mg(OH)_2$ and surronding chemical in sand were created. Moreover, water content would have been reduced at even lower MgO contents from 0.5% to 2% by means of longer treatment. In this respect, liquefaction was prevented by reduction in water content.

Table 6.5. Unconfined Compressive Strength of Samples treated for 3,7 and 14 days.

MgO Content(%)	**UCS (kPA)**		
	3-DAYS	**7-DAYS**	**14-DAYS**
0,5	4,565	4,970	5,766
1	4,945	5,836	6,886
2	5,010	6,813	8,176
4	5,202	8,584	10,472
6	6,679	11,889	14,743

Table 6.6. Failure Strain Values of Samples treated for 3,7 and 14 days.

MgO Content (%)	**Strain at failure (%)**		
	3-DAYS	**7-DAYS**	**14-DAYS**
0,5	1,342	1,074	0,966
1	1,265	1,012	0,911
2	1,186	0,949	0,854
4	1,107	0,886	0,818
6	1,018	0,824	0,761

Under same curing periods, rise in MgO contents increased mechanical resistance of samples as shown in Figure 6.6. In the range of MgO level from 0.5% to 4% under 3-days curing, raise in MgO content did not significantly affect on compressive response of samples and the failure strain values. Beyond 4% level of MgO under 3-days curing, enhancement in UCS and reduction in strain at failure were clearly observed.

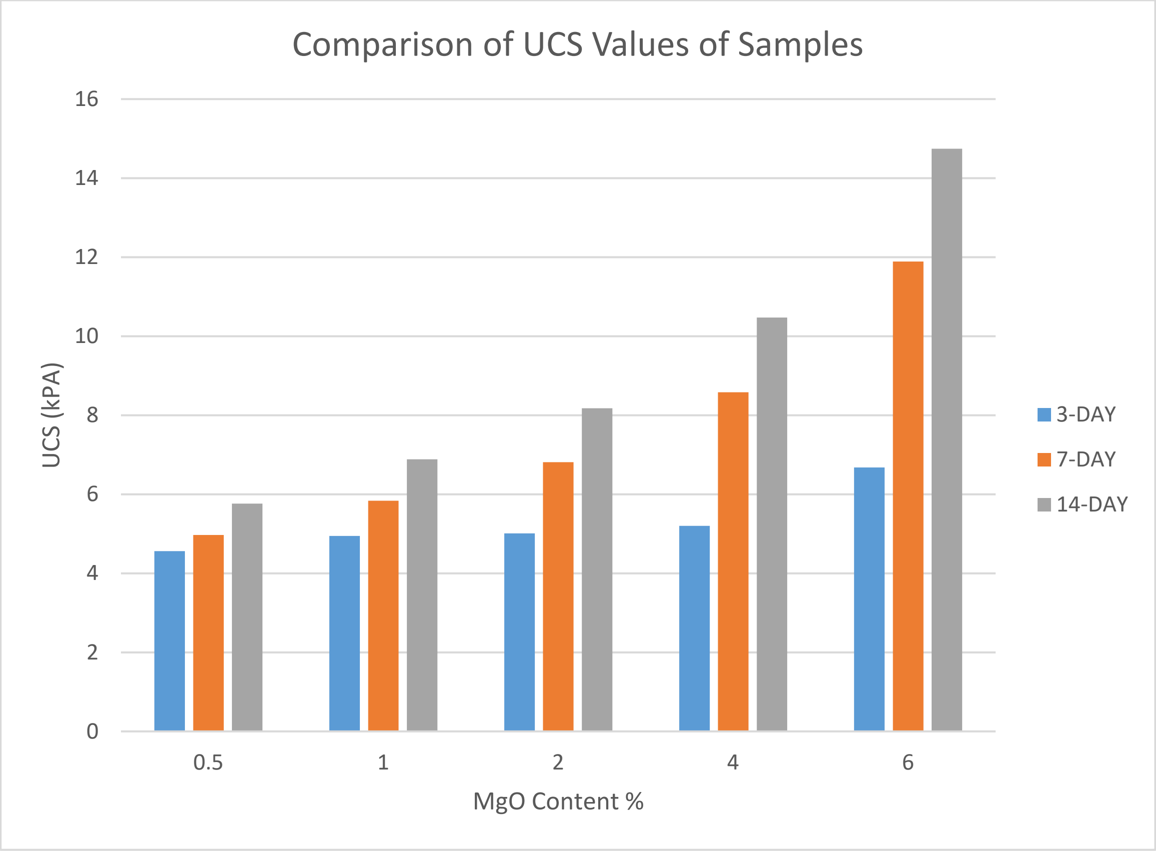


Figure 6.6. UCS Comparison for Sample with varying MgO content.

On the other hand, the reduction rate of the strain at failure MgO was increased after 4% MgO-sample under 3-days of curing condition. Likewise, UCS values for samples under 7 and 14-days of curing were considerably increased after 1% MgO level and strain values at failure were gradually decreased with rising rates.

In addition to this, increase in MgO contents under 14-days of treatment had maximum influence on UCS and strain values at failure compared to other curing period. Rise in MgO content generated more $Mg(OH)_2$ particles within pores between sand grains, resulted with more powerful cementation of sand particles, greater uniaxial resistance and lower deformation capability.

Limited effects of rise in MgO contents on strength development for lower MgO contents, 0.5% to 4%, under 3-days of treatment came from existence of water in the system, which was the indication of uncompleted hydration reactions and unadequate cementation. Moreover, it was appeared that water broke the activity of $Mg(OH)_2$ based-cementation between soil grains and thus, inner support system come from $Mg(OH)_2$ precipitates at earlier period of hydration was not properly established. By this way, existence of water caused the behaviour of sample with MgO to be turned into the one of fully saturated loose sand. That's why period of strainining of samples with lower MgO contents under 3-days was prolonged.

Even though 6% MgO-samples waited for 3-days had no water at all and it seemed that hydration reaction was completed, the capability of $Mg(OH)_2$ precipitates to provide interconnected and integrated network was not enough at those level of curing. However, the removal of water after addition of 6% MgO into soil gave lower strain at failure compared to other degrees in the samples cured for 3-days, because it was predicted that loadings in the absence of water would be trasmitted into specimens more effectively along with easy response of soil through defromation.

On the contrary, samples cured for 7 and 14 days did not have any water content, which was enough time for completion of hydration reactions in the system, so $Mg(OH)_2$ precipitates linked to soil grains more properly, where uniform, denser and homogenous arrangement within microparticular size was provided. Therefore, the response of samples aganist loadings occured as soon as possible. By this way, strain points at failure were decreased with inclined rate. Higher impact of MgO increase on UCS values and strain values at failure under 14 days could be explained with such that more interactions between $Mg(OH)_2$ particles and other chemicals such as sulfate,aliminute,phosphate and silicate within sand would become possible, which supported to strength-gaining much more and reduced to deformation capacity.

The main reasons for MgO content affecting UCS values were the higher diffusion from greater concentration into the water and wider distribution of MgO over sand at greater MgO contents. Thanks to hydration between water and MgO particles, $Mg(OH)_2$ solid particles were precipitated into soil pores. Therefore, gap between soil particles was filled with these precipitates and soil grains were connected to each other, which would provide to increase in cohesion and friction angle inside treated samples. By this way, loose saturated uniform fine sand became denser and well-connected.

When amount of MgO was incremented, more surface between MgO and water molecules existed, causing to increase in number of $Mg(OH)_2$. By this way, more pores of uniform clean sand having nearly uniform void spaces were filled with precipiates, so contact points between solid grains within samples were increased. Also, since the size of MgO particles (95 μ) is closer to the average size of uniform fine clean sand (252 μ as D_{50}), the arrangement of soil grains started to have more homogenous morphology and well- connected network inside the complex. By this way, pure sand was cemented along with increase in friction angle due to change in formation.

Under same MgO content, the longer curing provided UCS values to be increased. At higher MgO content as 4% and 6% compared to lower one from 0.5% to 2%, the effect of extension of treatment on compressive resistance on the soil was greater. Even, at 0.5% MgO level, the effect of time on UCS was very limited except strain values at failure compared to the one with higher MgO content. But when proportions of MgO was incremented, treatment period supported to enhancement.

Nevertheless, mechanical resistance of sample with lower MgO under longer curing period exceeded to the one of sample with higher MgO under short curing period as can be stated in general. Based on these, at lower MgO contents from 0.5% to 2% as percentage, the curing period was more significant than the amount of MgO chemical regarding strength development.

On the other hand, at greater MgO levels as 4% and 6%, especially after 7 days, the effect of time was behind the impact of MgO content on UCS values. This was proved by looking into water content level after 7-days of curing, where completely dry cylindirical blocked-shape samples were easily observed as that of samples cured for 14-days.

Even, it was inferred from comparison between 7-days cured and 14-days cured samples having similar water content level such that the influence of extended time on UCS started to be decline after 7 days, In other saying, the increase of MgO content begun to be more effective on UCS and strain values at failure after hydration of MgO was completed and water content of samples became zero.

In the light of this information, hydration reactions between sand-MgO-water complex needed at least 7-days of curing to give reasonable results.. However, under 3-days of curing, hydration reaction was not completed for samples with lower MgO contents from 0.5% to 4%. It was proved by existence of water on the samples cured for 3-days. Therefore, soil grains under loose state were not connected properly due to water existence even if MgO reached to considerable content such as 4%. Although 6% MgO-samples waited for 3 days had more denser formation and nearly zero water content level compared to others, UCS on this level was still below that of 2% MgO-samples cured for 7 days.

Similarly, 6% MgO-sample under 3 days curing had just strength due to early formation of $Mg(OH)_2$ precipitates, so it needed still time to connect soil grains more properly and more integrated,denser ,uniform microstructures would be occurred.

Regarding strain at failure, prolonged curing decreased the strain at failure for all samples with all degrees of MgO. The hydration reaction of MgO showed its influence on the sand-water complex beyond 3 days. Therefore, except 3 days curing condition, strain points of samples at failure were affected directly from extesion of time at all MgO content levels. Even, the longer curing influenced on strain points at failure much more than the addition of MgO under all conditions.

However, samples waited 3 days with MgO contents from 0.5% to 4% had strain values at failure more than that of samples waited 7 days with just 0.5% MgO content. Similarly, under 3-days curing and 6% MgO level, strain value at failure reached to the level less than that of the one with 0.5% MgO cured for 7-days. As main reason, hydration reactions were not finished or existence of water diminished the actual effect of MgO.

As mentioned before, samples started to have a texture which can transmit loading over the sample in the absence of water. By this way, sample showed the reaction readily into uniaxial loading. That's why sample with 0.5% MgO under 7-days curing was failed at lower strain compared to ones under 3 days curing condition.

After 7 days, improvement in mechanical resistance was raised up along with reduction in strain values at failure considerably. Even, extension of curing period beyond 7 days of curing let samples to be failed at strain points much more lower than that under other conditons.

To sum up, prolonger time also supported to strength-gaining by giving greater UCS and strains values at failure point and thus, at 14 days, a maximum compressive resistance and minimum strain values were obtained at failure among all. Maximum UCS value among all was attained at the sample with 6% MgO cured for 14-days.

According to previous parts, it was known that at initial stage of hydration, only reason responsible for strength-gaining was precipitation of $Mg(OH)_2$. However, after certain time, it would be possible to see that the reaction between $Mg(OH)_2$ and reactive chemicals from sand like aliminum, phosphate and sulfate occurred. In addition to this, some findings from literature has shown that there was an interaction between $Mg(OH)_2$ solid and silica from sand particles to form Magnesium Silicate Hydrates (M-S-H) , which enhances the strength development more than that by precipitation of $Mg(OH)_2$ at later period [37].

Nevertheless, when amount of MgO reached to 4% level or more, effect of curing period was insufficient, since number of hydration products binding soil became greater than that of soil having less MgO content under longer curing duration. As statistically, unconfined compressive strength reached to 14,74 kPA as maximum among all after 14 days and UCS values were risen more than 1.24 times compared 7-days and 2.21 times compared to 3-days under 6% MgO levels.

Table 6.7. Young's Modulus of Samples after for 3,7 and 14 days.

MgO Content (%)	**E_{50} (kPA)**		
	3-DAYS	**7-DAYS**	**14-DAYS**
0,5	339,791	462,962	596,717
1	390,934	576,627	756,021
2	422,335	717,962	957,295
4	469,806	968,975	1313,493
6	615,487	1369,461	1886,812

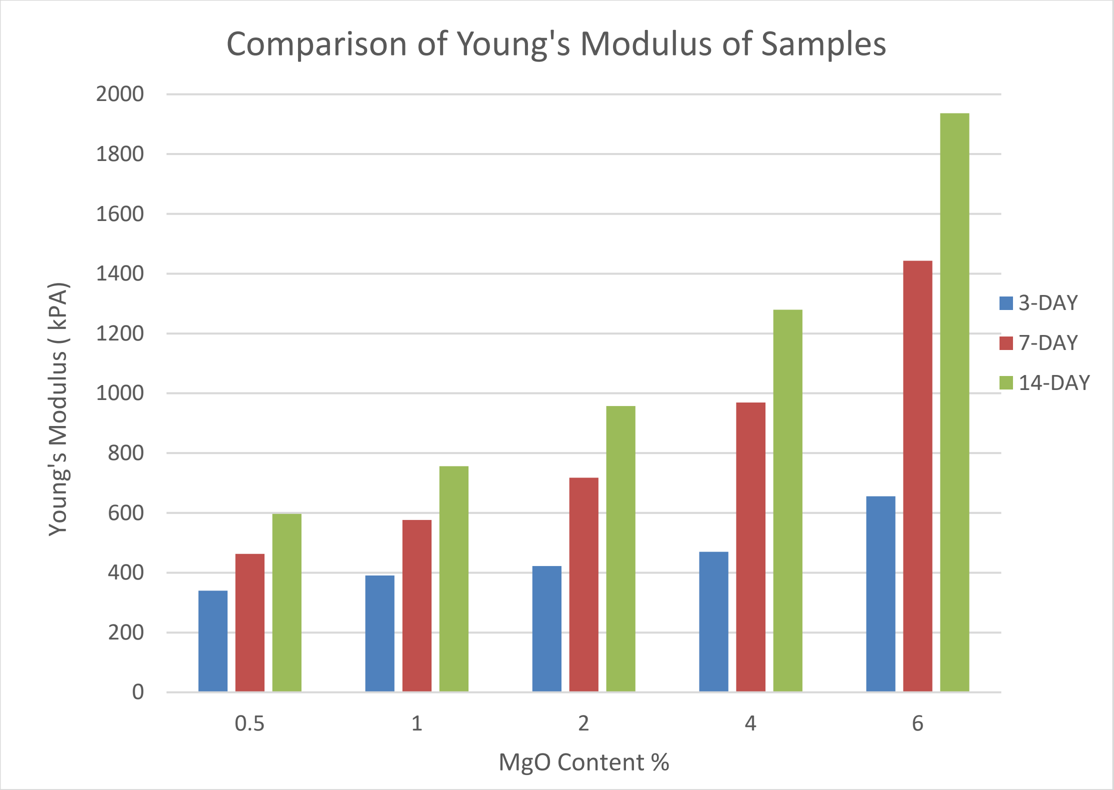


Figure 6.7. Comparison for Young's Modulus of Sample with varying MgO content under 3,7 and 14 days of curing.

In the relationship between MgO contents and mechanical behaviour of samples, it would have stated that Young's modulus of samples became greater, when MgO content was increased at constant curing days as presented in both Table 6.7 and Figure 6.7. As the reason, MgO particles turned into $Mg(OH)_2$ precipitates solid within soil pores, which were produced to connect soil grains into each other. Increase in number of MgO particles enhanced the its distribution over sand and eased to its diffusion into water along with growth of $Mg(OH)_2$ particles, which supported to cementation of the soil, integrated networks and denser formation inside the treated samples. By this way, compressive strength was increased.

In addition to this, water content level was also decreased during hydration, which made specimen drier, so solid components (hydration products and sand particles) integrated and contacted with each other more conveniently. Therefore, with the addition of MgO after adequate time required for the completion of hydration , transmission of loading and easier response of sample became easier. By this way, deformation capacity of sample was decreased and specimens experienced less axia deformation at failure. Thus, Young’s Modulus was inclined by addition of MgO into sand.

Due to increase in Young’s Modulus, mechanical behaviour of samples was changed from ductile to brittle. Regarding more details for the effect of MgO contents, its impact on Young's Modulus was more clear for the samples under 7 days and 14-days of curing. For samples waited for 3-days, Young's Modulus was increased much more at 6% MgO level, whereas significant development in Young's Modulus started from 1% level of MgO for samples cured for 7 and 14-days.

As mentioned before, both increase in MgO and extension of period provided the completion of hydration reactions, better early and post hydrations products, provided to better interconnection between soil grains, which supported to UCS development and earlier failure. By this way, Young’s Modulus became greater, which made samples brittle gradually. Moreover, since rate of UCS development and reduction of deformation had inclined trend, Young’s Modulus was raised many times more. On the other hand, samples waited for 3-days containing low MgO contents from 0.5% to 4% showed more ductile behaviour with respect to 6% MgO level, because the transmission of unixial loading was not done by integrated solid components completely within complex due to the presence of water as a result of uncompleted hydration reactions. Thus, lower mechanical resistance with higher axial deformation at failure was possible under this condition, which lessened Young’s Modulus. As a result of this, appropriate response of treated 3-days waited samples with lower MgO contents showed more ductile behaviour compared to 3-days waited sample with 6% MgO content.

Regarding the influence of curing periods on the stiffness, extension of time enhanced to Young's Modulus of samples. The significance of time could be seen clearly after 3-days of curing, because time of curing was not enough to complete hydration reaction along with existence of water. Thus, weaker samples, which was failed at longer time, were generated. By this way, more ductile behaviour was presented compared to other curing periods. In the following of 7-days, water presence became zero, which provided to more developed conduction of loading over denser, more uniform and connected structures that also improved to mechanical resistance. In this way, samples begun to be failed earlier along with higher UCS values, so Young's Modulus was incremented for each MgO content level between 3-days and 7-days waited samples. At the end, increase rate of Young's Modulus by MgO addition reached at maximum level at 14-days, as possible reactions between $Mg(OH)_2$ solid with surronding chemicals in the system such as phosphate,alliminum,silicate,sulphate occured after 7 and 14-days, but more efficiently in 14 days. Thus, Young's Modulus became greater allied with more brittle behaviour.

The effect of time on Young's Modulus compared to the effect of MgO contents rised between samples under 7-days and 14-days curing started to be clear at lower MgO contents from 0.5% to 2%, because strength growth under lower MgO levels was affected by post-hydration progression and samples in both conditions did not have water content. However, when number of MgO particles was raised excessively and sufficient time was given into complex as 7 days, $Mg(OH)_2$ particles were increased in number, resulted with rise in connection points and more enstructured formation.

The resistance of samples with higher MgO from 4% to 6% could easily exceeded to that of the one with lower MgO contents (from 0.5% to 2%) under longer treatment as 14 days. This also caused the samples to be failed much more earlier. In this way, Young's Modulus was varied according to rise and drop rate of both UCS and strain values at failure.

On the other hand, the effect of time on Young's Modulus compared to the effect of MgO contents was valid for every step of MgO levels between samples waited 3-days and others due to the uncompleted status of hydration reaction under this limited time because hydration reactions were influence by temperature, mixing type and reactivity of MgO but mostly by time, since hydration degree was changed by mainly time. That's why Young's Modulus of 6% MgO-sample waited 3-days was close to that of samples with 1% MgO cured for 7-days and 0.5% MgO cured for 14 days.

Table 6.8. Maximum strain capacity of samples with varying MgO contents after for 3,7 and 14 days.

MgO Content (%)	**Strain at End %**		
	3-DAYS	**7-DAYS**	**14-DAYS**
0,5	1,962	1,283	1,125
1	1,831	1,187	1,064
2	1,657	1,132	0,991
4	1,484	1,021	0,816
6	1,396	0,952	0,903

Even though those had considerably lower MgO contents, extension of time made them stronger and more brittle. According to Table 6.8, it could be understood that longer period of curing plus MgO addition made sand stiffer as the behaviour where greater resistance along with lower deformation ability existed, leading to higher brittleness. it can be stated that curing time and MgO amounts turned to saturated uniform sand from ductile to brittle behavior. It indicated that Young's Modulus of treated soil was increased with the additional MgO and longer period of curing.

When MgO portions and curing periods were expanded, deformation capacity of samples became smaller. The durability of cemented samples with all MgO was maximum under 3-days of curing compared to other. The most apparent reason was existence of water, uncompleted hydration reaction, insufficent interlinkage between grains and ductile behaviour during loading.

Since the interference between grains contains with not only $Mg(OH)_2$-based bonds but also water , transmission of loading was not performed appropriately. Time for decomposition of sample took longer time.

On the other hand, under 7 and 14 days, where water was not presented, total deformations were diminished with increasing rates, because an external surcharge could have been conveyed lightly throughout the sample samples and by the development of brittleness in the sample, strain at the end was slightly reduced. The drop rates of durability under 14-days were much higher than that under 7-days. Based on this, it was inferred that durability of MgO treated sand was related with its stifness behaviour and when brittleness was increased, the resistance period of samples aganist loadings was shorter.

Even if durability had positive meaning for the many engineering branches, it meant the huge deformation under very low strength for geotechnical engineering. To prevent catastrophic damages to civil and natural life during liquefaction, the choice which gave a minimum durability must be selected.

In this study, it would be the sample with 6% MgO under 14 days of curing, because under this condition, mechanical, hydrological, pyshical properties and durability charachteriscs of loose saturated liquefable sand was substantially developed by decreasing water content level, supporting maximum UCS value, providing minimum durability level and generating more brittle behaviour.

To sum up, the addition of magnesium oxide inot fully saturated liquefiable loose clean sand was useful ;

- to remove saturation after adequate period of time,
- to constitute well-connected and strong-bonded network in the complex
- to improve elastic charachteristics by making soil more brittle,
- to detect occurrence of the deformation before it exceeds permissible limits through dropping strain value at failure,
- to prevent excessive deformation after failure,
- to enhance mechanical compressive strength into considerable level.

Moreover, longer period of treatment supported to lower axial deformation at failure, strength- gaining, reduction of saturation, cementation of soil particles, stifness behaviour and elasticity and durability, since hydration reactions between MgO and water particles required enough time to form $Mg(OH)_2$ precipitates and further products formed from interactions among $Mg(OH)_2$ and other chemicals on the system.

7. CONCLUSION

In this study, fine uniform loose sand, which is susceptible to static liquefaction, were enhanced with different rates of magnesium oxide towards mechanical, physical, and hydrological characteristics. In general, the purposes were stated as lowering the saturation level, excessive deformation happened during liquefaction and increasing unconfined compressive strength of soil. For this reason, several unconfined compression tests were implemented on treated sand samples cured for 3,7 and 14 days.

According to the results, the initiation of liquefaction was prevented through removal of water content within soil and the permeability of loose fine sand was nearly disappeared ,depending on curing conditions such as 7 and 14-days, due to the physical properties of treated samples having solid column shape. It was observed that increment of MgO amount develops the mechanical resistance, stiffness and saturation degree against liquefaction.

In case of extension of curing, the impact on the soil was realized with enhancement in UCS value, development in stifness and reduction in water content. Despite this, durability in terms of axial strains decreased with increase in brittleness, so excessive deformation would be prevented.

Plus, to decrease the environmental effect of chemicals, magnesium oxide solid was produced without emitting excess carbon dioxide gasses to the atmosphere. Moreover, the energy needed to obtain magnesium oxide solid was much more than the cement or other chemicals. Therefore, the usage of magnesium oxide is very economical.

Furthermore, there is no any toxicity of magnesium oxide given into the soil and environment. As MgO usage on buildings material industry, application of magnesium oxide in the liquefiable regions was good option to control liquefaction causes. Through hydration ability with water, magnesium oxide, in solid form, might be given to the inaccessible locations under prior buildings or towards buried structures.

In future, there would be development about how magnesium oxide should be implemented to soil underground surface. The most economical and practical solution could be offered to utilize the effect of magnesium oxide on all liquefiable soil. As the novel study, this encouraging chemical exhibits its importance for geotechnical engineering and more comprehensive efforts are still needed.

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