EFFECTS OF FLY ASH AND DESULPHOGYPSUM ON THE GEOTECHNICAL PROPERTIES OF ÇAYIRHAN SOIL

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I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

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This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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ABSTRACT

EFFECTS OF FLY ASH AND DESULPHOGYPSUM
ON THE GEOTECHNICAL PROPERTIES OF ÇAYIRHAN SOIL

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Collapse in soils occur when a partially unstable, partially saturated open fabric under high enough stress causing a metastable structure with large soil suction, or in the presence of a bonding or cementing agent, is allowed to free access to additional water. Such excess water reduces soil suction and weakens or destroys the bonding, thus causing shear failure at the interaggregate or intergranular contacts, consequently, the soil collapses. In this study, the collapsible soils found in the Çayırhan Thermal Power Plant area has been stabilized by using the desulphogypsum, and fly ash obtained from the Çhayırhan Thermal Power Plant. An extensive laboratory testing program has been undertaken to provide information on the geotechnical properties of collapsible soils treated by Çayırhan fly ash and desulphogypsum. At the end of the test program, it has been seen that the collapsible soil (compacted) can be stabilized by adding fly ash and desulphogypsum. Although a significant change on the collapse potentials was not observed when fly ash and desulphogypsum added samples were compared with compacted sample without stabilization, but there is an increase in unconfined compressive strength values due to stabilization.

Keywords: Collapsible Soils, Metastable Soils, Hydrocompression, Hydrocollapse, Fly Ash, Desulphogypsum

ÖZ

UÇUCU KÜL VE DESÜLFOJİPSİN ÇAYIRHAN ZEMİNİNİN GEOTEKNİK ÖZELLİKLERİ ÜZERİNDEKİ ETKİLERİ

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Kısmen kararsız, kısmi doygun dokunun yüksek emme basıncı ile yarı kararlı hale geçmesine yetecek kadar yüksek bir basınç altında yada çimentolaşma faktöründen dolayı bir bağ oluşması durumunda, bu dokunun suyla teması halinde zeminde çökme oluşur. Bu artık su, emme basıncını düşürerek ve bağı zayıflatarak veya yok ederek taneler arası bağlantılarda kayma kırılmasına sebep olur. Buna bağlı olarak zemin çöker. Bu çalışmada, Çayırhan Termik Santrali alanında bulunan çökebilen zemin, Çayırhan Termik Santralinden elde edilen uçucu kül ve desülfojips ile stabilize edilmiştir. Çayırhan uçucu külü ve desülfojips ile iyileştirilen çökebilen zeminin geoteknik özellikleri hakkında bilgi sahibi olabilmek için yoğun bir laboratuvar test programı uygulanmıştır. Test programının sonunda, çökebilen zeminlerin (sıkıştırılmış) uçucu kül ve desülfojips katılması ile stabilize edilebileceği görülmüştür. Uçucu kül ve desülfojips eklenen numunelerin çökme potensiyellerinde, sıkıştırılmış numeneyle karşılaştırıldığında, çok belirgin bir değişim gözlenmemiş olsa da, stabilizasyondan ötürü serbest basınç mukavemetlerinde artış görülmektedir.

Anahtar Kelimeler: Çökebilen Zeminler, Kararsız Zeminler, Hidrosıkıştırma, Hidroçökme, Uçucu Kül, Desülfojips

To
My Meaning of Life

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CHAPTER 1

INTRODUCTION

In arid and semiarid areas of the world, geotechnical engineers often encounter deposits of collapsible soils. Collapsible soils are strong in their natural state; however, if they become wet, these soils quickly consolidate. Collapsible soils are stable if they remain wet, so they are also called metastable soils, and the process of collapse is called hydroconsolidation, hydrocompression, or hydrocollapse (Coduto, 1994).

Collapse in soils occurs when a partially unstable, partially saturated open fabric under a high enough stress causing a metastable structure with large soil suction, or in the presence of a bonding or cementing agent, is allowed free access to additional water. The amount of collapse usually increases with the initial applied pressure and decreases with initial water content and dry unit weight (Basma and Tuncer, 1992).

Curtin (1973) gives an interesting illustration of collapsible soils. In California’s San Joaquin Valley, a collapsible soil deposit of 75 meters is wetted for 484 days and a settlement of 4.1 meters are observed after wetting. In another case, irrigation of lawns and landscaping, and poor surface drainage around a building in New Mexico caused the wetting front more than 30 meters into the ground, which resulted 25-50 mm of settlements (Coduto, 1994).

Çayırhan Thermal Power Plant Units I and II are directly founded on collapsible silty soils without any special precautions, the foundations being formed as mat foundations. Some collapse settlements resulting in severe cracking of the structural system and foundations were observed several years after the completion of construction. Cement injection was utilized in order to reduce and eliminate the further collapse settlements. However the injection of cement and water increase the collapse settlements indicating that it was not the proper remedial measure. The foundations of Units III and IV which were built later were designed as pile foundation upon this bad experience faced in Units I & II (Private communications with Dr. Oğuz Çalışan).

On the other hand, the increasing demand for energy has resulted in construction of many coal-fired power plants in Turkey. This development brought it with the problem of safe disposal or beneficial utilization of large quantities of by-products from these power plants (Çokça, 2001). Çayırhan Thermal Power Plant is located at 120 km from Ankara and 22 km from Beypazarı (Figures 1.1. and 1.2.). The plant covers a total area of 5,032,000 m^2. It has four boiler units, two of them (Units I and II) with 150 MW capacity and two of them (Units III and IV) with 160MW capacity. Units I and II have been working since 1987 and Units III and IV have been working since 1998. All of the four units are equipped with flue gas desulphurization systems. These four units, with a total capacity of 620MW, use 5,000,000 tons of lignite coal and generate 4,200,000,000 kW-h electricity per year. The lignite coal, extracted from the underground mines of the Beypazarı Basin, is of low calorific value (2200 kcal/kg), high dust (30 – 45%), and high sulphur (4 – 5%) content. As a result of their electricity generation the four units of Çayırhan Thermal Power Plant produce 1,350,000 tons of fly ash and 680,000 tons of desulphogypsum annually. Fly ash and desulphogypsum are collected by means of electrostatic precipitators and are sent through 2.5 km transfer bands into open stock areas which now cover a total area of 1,137,000 m^2. Less than 1% of fly ash and none of the desulphogypsum are productively employed. The plant is estimated to work for minimum another 20 years, and it is conceivable to mention that this will duplicate the fly ash and desulphogypsum stocks. These stocks pose a serious problem in terms of both land use and potential environmental pollution. An effective utilization of these industrial by-products must be regarded as economically and environmentally beneficial (Çetiner, 2004).

Figure 1.1 Location of Çayırhan in Turkey

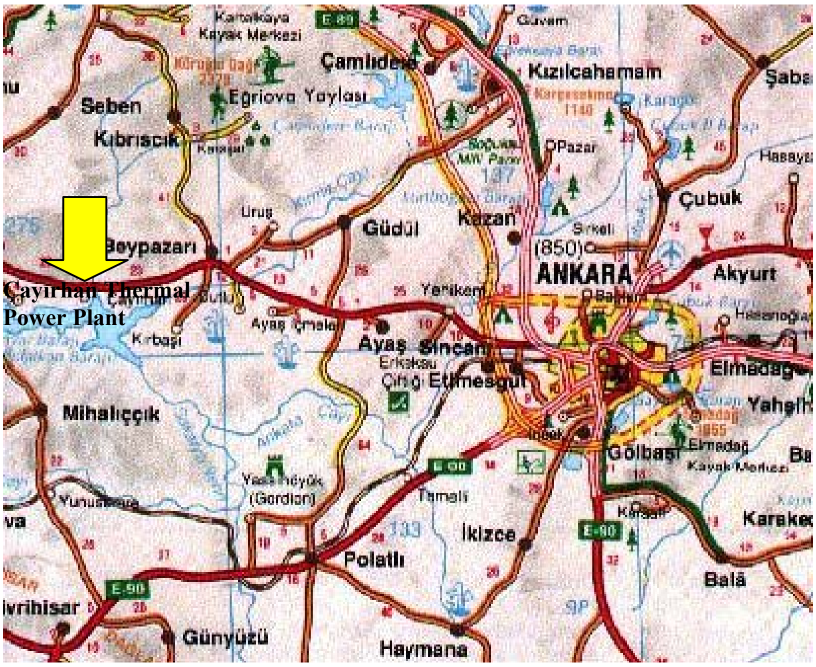

Figure 1.2. Location of Çayırhan in Ankara

In this study, the collapsible soil was stabilized using the fly ash and desulphogypsum obtained from Çayırhan Thermal Power Plant. An extensive laboratory testing program was undertaken to provide information on the geotechnical properties of collapsible soils treated with Çayırhan fly ash and Çayırhan desulphogypsum.

CHAPTER 2

REVIEW OF COLLAPSIBLE SOILS

Collapsible soils generally consist of sand and silt particles arranged in honeycomb structure as shown in Figures 2.1. Sometimes gravel is also present. The loose structure shown in Figure 2.1, is held together by small amounts of water-softening cementing agents, such as clay or calcium carbonate. As long as the soil remains wet, these cementing agents produce a strong soil that is able to carry loads. However, if the soil becomes wet, the cementing agents soften and the honeycomb structure collapse as shown in Figure 2.2 (Coduto, 1994).

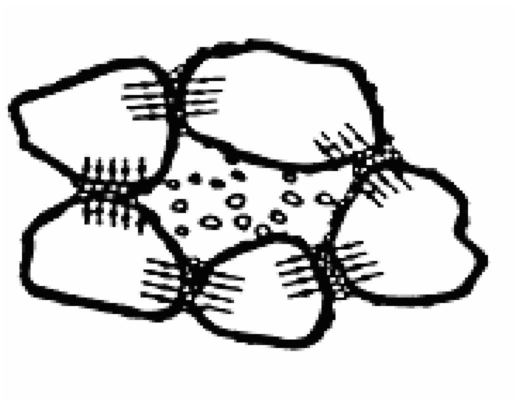

Figure 2.1. Structure of Collapsible Soils (Loaded Structure before Inundation) (Houston, et al., 1988)

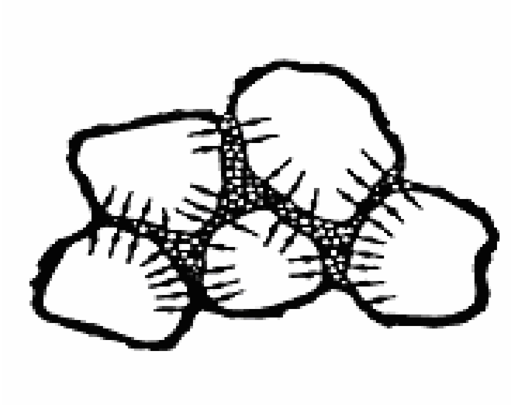

Figure 2.2. Structure of Collapsible Soils (Loaded Structure after Inundation) (Houston, et al., 1988)

2.1. Origin and Occurrence of Collapsible Soils

The most extensive collapsible soil deposits are wind-deposited sands and silts. Alluvial flood plains, mud flows, colluvial deposits, residual soils and volcanic tuffs are also soils that can be collapsible.

2.1.1 Alluvial and Colluvial Soils

Some alluvial soils i.e. transported by water and some colluvial soils i.e. transported by gravity can be highly collapsible. Short bursts of intense precipitation often induce rapid down-slope movements of soil known as flows. While moving, the soil is nearly saturated and has a high void ratio. Upon reaching the destination, the soil dries quickly by evaporation, and capillary tension draws the pore water toward the particle contact points, bringing clay and silt particles and soluble salts with it (Figure 2.3). Once the soil becomes dry, these materials bond the particles together, thus forming the honeycomb structure.

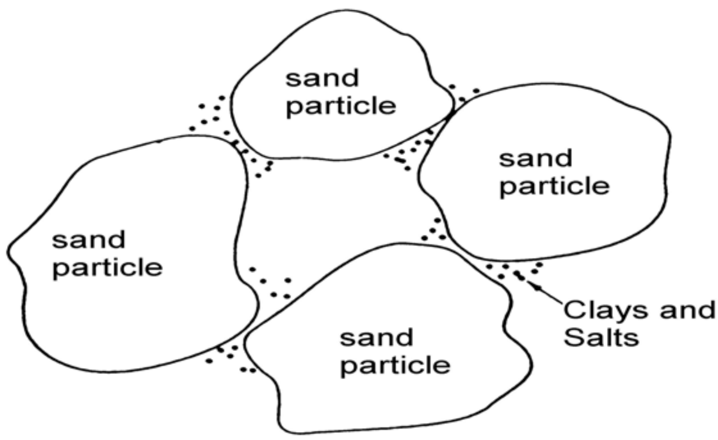

Figure 2.3. Forming of Collapsible Soils (Coduto, 1994)

After the next flow, more honeycomb structured soil forms. The new flowed layer dries rapidly by evaporation, too, so the previous deposited soil remains dry. Thus, deep deposits of collapsible soil can form. These deposits are often very erratic, and may include inter-bedded strata of both collapsible and non-collapsible soils (Coduto, 1994).

2.1.2. Aeolian Soils

Soils deposited by wind are known as aeolian soils. This category includes wind-blown sand dunes, loess, volcanic dust deposits, etc. Loess i.e. aeolian silt or sandy silt, is the most common aeolian soil which covers much of the earth's surface. It is found in the United States, central Europe, China, Africa, Australia, the former Soviet Union, India, Argentina, and elsewhere (Pye, 1987).

Collapsible loess has a very high porosity (about 50%) and a correspondingly low unit weight (11-14 kN/m^3). The individual particles are usually covered by clay, which acts as cementing agent to maintain the loose structure. Loess deposits are generally much less erratic than other types of collapsible soils, but they are often much thicker. Deposits about 60 meters of thick are not unusual (Coduto, 1994).

2.1.3. Residual Soils

Residual soils are formed in-place by weathering of rock. Sometimes this process involves decomposition of rock minerals into clay minerals that may be removed by leaching, leaving a honeycomb structure and a high void ratio. When this structure develops, the soil is prone to collapse. Residual soils are likely to have greatest amount of spatial variation, thus making it more difficult to predict the collapse potential (Coduto, 1994).

2.2. Identification of Collapsible Soils

Many different techniques are used to identify the collapsible soils. They may be divided into two main groups; indirect methods and direct methods.

2.2.1. Indirect Methods

The indirect methods assess the collapse potential by correlating it with other engineering properties such as unit weight, Atterberg limits or percent clay particles. The results of such efforts are usually a qualitative classification of collapsibility, such as “highly collapsible”. Although these classifications can be useful, they provide little quantitative estimates of potential settlements (Lutenegger and Saber, 1988). In addition, most of them have been developed for certain types of soils, such as loess, and cannot necessarily be used for other types, such as alluvial soils. For example, if the unit weight of the soil is about 11-14 kN/m^3, than the soil can be treated as collapsible soil (Coduto, 1994). However, direct methods have to be used for further evaluations.

2.2.2. Direct Methods

Direct methods involve actually wetting the soil, either in laboratory or in-situ, and measuring the corresponding strain. The results of such tests can be extrapolated to the entire soil deposit and the potential settlements can be predicted.

2.2.2.1. Single Oedometer Test

This method is generally used for assessing collapse potential. In this method, each test requires only one sample.

An undisturbed soil sample in an oedometer with in-situ moisture content is consolidated with stress increments. When the applied vertical stress becomes equal or slightly higher than the overburden pressure, the sample is inundated. The strain observed after inundation is called hydrocollapse strain, ϵ_w. After hydroconsolidation, additional stress increments are applied to allow the soil consolidate (Houston, et al., 1988). The results of such a test are shown in Figure 2.4.

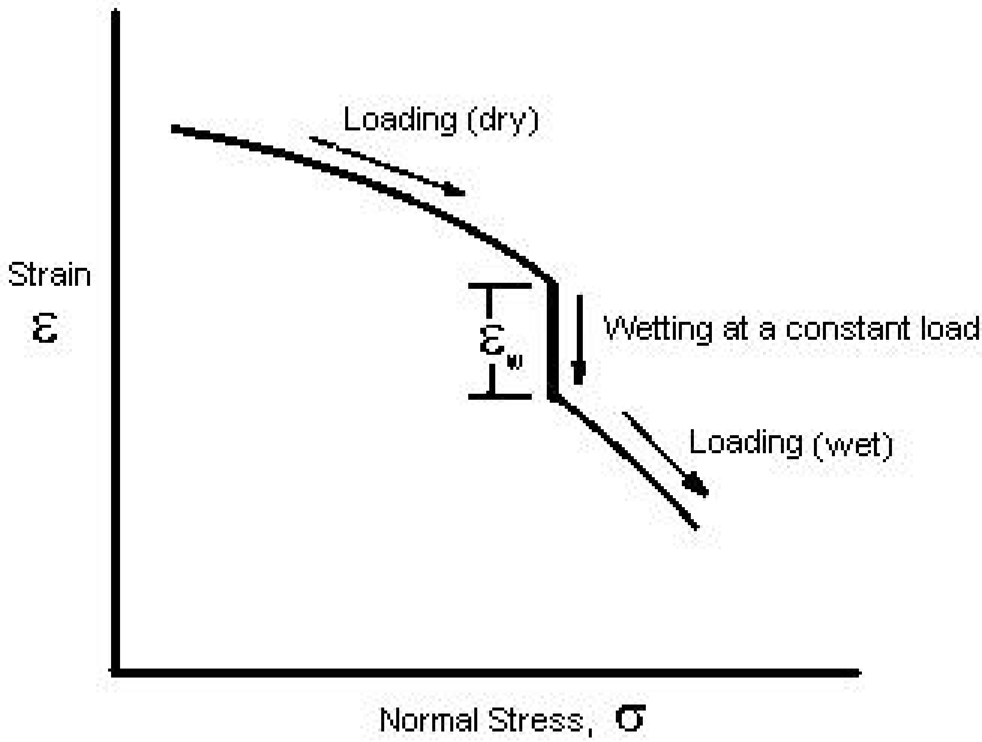

Figure 2.4. Single Oedometer Test Results (Houston, et al., 1988)

The single oedometer test is faster and more closely simulates the actual loading and wetting sequence that occur in the field. However, this method provides less information, since it only gives the hydrocollapse potential at one normal stress. Therefore, the soil should be at a normal stress as close as possible to that which will be present in the field (Coduto, 1994).

2.2.2.2. Double Oedometer Test

Two parallel oedometer tests on identical soil samples are used in this method. The first test is performed on a sample at its in-situ moisture content; the second on a soaked sample. Test results are plotted together (Figure 2.5.). The vertical distance between the results represents the potential hydrocollapse strain, ϵ_w, as a function of normal stress (Jennings and Knight, 1975).

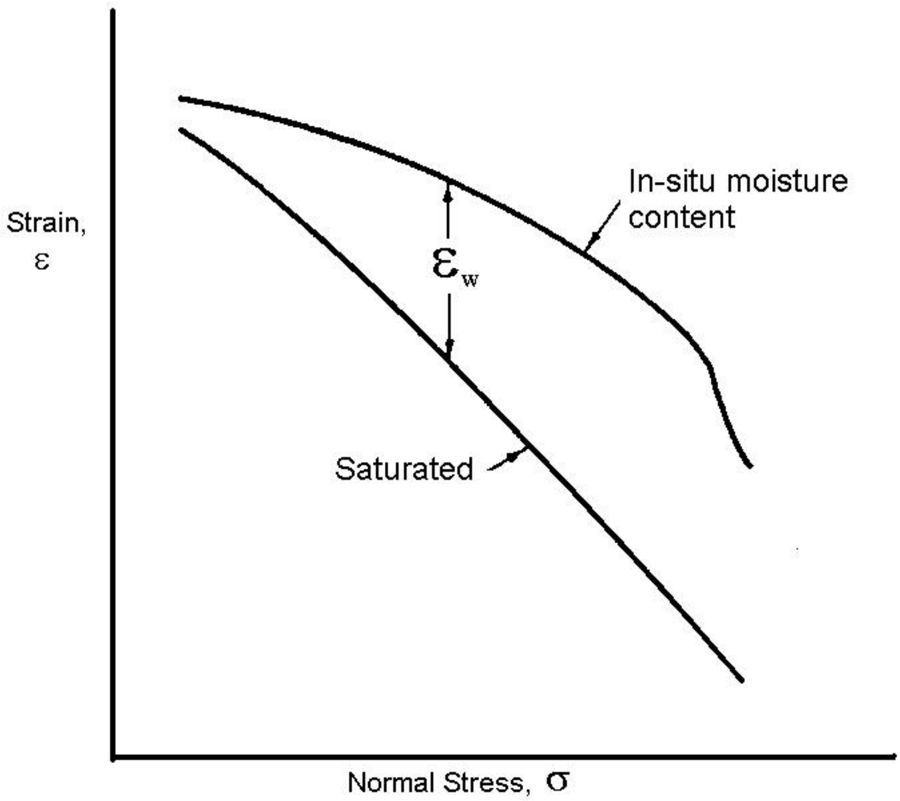

Figure 2.5. Double Oedometer Test Results (Coduto, 1994)

The criteria for evaluation of laboratory test results are shown in Table 2.1.

Table 2.1. Classification of Soil Collapsibility (Jennings and Knight, 1975)

Potential Hydrocollapse Strain, ϵ_w (%)	Severity Problem
0-1	No problem
1-5	Moderate problem
5-10	Trouble
10-20	Severe trouble
>20	Very severe trouble

2.2.2.3. In-situ Collapse Tests

Gravelly soils pose special problems because they are very difficult to sample and test, yet they still may be collapsible. For evaluation of these soils it may be necessary to conduct in-situ collapse tests. Some of the tests have consisted of a large-scale artificial wetting with associated monitoring of settlements (Curtin, 1973). Some others have consisted of small-scale wetting in the bottom of borings (Mahmoud, 1991).

2.2.2.4. Other Criteria for Evaluating Collapsible Soils

a) Soviet Building Code

$$L = \frac{e_0 - e_L}{1 + e_0}$$

where;

e_0 = natural void ratio

e_L = void ratio at liquid limit

For natural degree of saturation less than 60%

If L>0.1, soil is collapsible.

b) Jennings and Knight

Jennings and Knight use a single oedometer test with an undisturbed soil sample at natural moisture content. The process is to load the specimen by steps up to 200 kPa. After reaching 200 kPa the specimen is saturated by flooding and left 24 hours. The test provides e_1 (void ratio before flooding) and e_2 (void ratio after flooding) (Jennings and Knight, 1975).

$$C_p = \Delta\varepsilon = \frac{e_1 - e_2}{1 + e_0}$$

where;

e_0 = natural void ratio

$\Delta\varepsilon$ = vertical strain

The C_p criteria are the same as Table 2.1.

CHAPTER 3

IMPROVEMENT OF COLLAPSIBLE SOILS

In general, collapsible soils are easier to deal with than expansive soils because collapse is a one way process, whereas expansive soils can shrink and swell again. Many mitigation methods are available, several of which consist of densifying the soil, thus forming a stable and strong material. Some of the methods of mitigation are (Houston and Houston, 1989; Mitchell, 1981):

- Removal of collapsible soil

Sometimes the collapsible soil can simply be excavated and the structure then may be supported directly on the exposed non-collapsible soil. Lowering the grade of the building site or using some basements are the methods for excavating. This method is very effective when the collapsible soil extends only a shallow depth.

- Avoidance of wetting

Collapse will never occur unless the soil is wetted. So, when working with collapsible soils, some extra measures are taken to minimize the infiltration of water into the ground. For some structures, such as electric transmission towers, simple measures such as this will be sufficient. However, the probability of success is much less when dealing with foundations for buildings because there are many opportunities for wetting, and consequences of settlements are more expensive to repair. Therefore, in most cases, this technique is combined with other preventive measures.

- Transfer of load to the stable strata below

If the collapsible soil deposit is thin, then it may be feasible to extend spread footing foundations through the stable strata. When the deposit is thick, deep foundations may be used for the same purpose. In either case, the ground floor would need to be structurally supported.

- Injection of chemical stabilizers or grout

Many types of soils, including collapsible soils, can be stabilized by injecting special chemicals or grout. These techniques strengthen the soil structure so wetting will not cause it to collapse. This method is generally too expensive to use over large volumes of soil, but it can be useful to stabilize small areas or as a remedial measure beneath existing structures.

There are three modes of injection.

Permeation grouting in which the grout fills the soil pores. There is essentially no change in the volume or structure of the original ground. This type of grouting can generally only be accomplished in soils coarser than fine sands and in fissured rocks.

Displacement grouting in which a stiff mixture fills voids and compresses the surrounding soil.

Encapsulation in which naturally fragmented ground or ground fractured hydraulically under high grout fluid pressures is injected by grout which coats but does not permeate the individual chunks of soil. A lens structure in the form of cardhouse is formed.

Permeation grouts are of two types. Particulate grouts are made up of cement, soil, or clay and mixtures of these. Chemical grouts are composed of various materials in solution. Displacement or compaction grouts are stiff, low slump (0 to 50 mm) mixtures of cement, soil, and/or clay and water. Lime slurries are the most commonly used encapsulation type grouts, however, there is no inherit reason, except perhaps for economics, why other chemicals could not be used.

Neat cement and soil-cement grouts are the most commonly used particulate grouts, although soil-water grouts have been used in some cases. In water-cement grouts water:cement ratios of 0.5:1 to 6:1 have been used. With low water:cement ratios there is less segregation and filtering, and higher strengths are obtained, but they are harder to inject than grouts with a higher water content. Chemical additives are sometimes used to facilitate penetration, to prevent cement flocculation, and to control set times. Set times can be as short as 30 seconds or very long.

In soil-cement grouts a soil volume of four to six times the loose volume of cement is common. Water volumes from one third to twice the soil volume per bag of cement are used. The low water content mixes are typical of high viscosity displacement grouts. Zero slump compaction grouts with 30 to 60 second gel times can be made using cement, clay, and fly ash mixes with an alkaline accelerator. If bentonite is used, expanded particles may collapse if the ground water has a high salt content. Care should be taken in the use of cement in the presence of sulfate-bearing soils of ground water.

Chemical grouts offer the advantages over particulate grouts that they can penetrate smaller pores, they have a lower viscosity, and there is a better control of set time. On the other hand their technology is more complex and costs are high. Soils containing less than 10 percent fines can usually be permeation grouted with chemicals. If the fines content is greater than 15 percent effective chemical grouting may be difficult. For fines content greater than 20 percent permeation grouting will not be possible, but chemical grouts can be distributed along and through hydraulic fractures.

The most common chemical grout classes are silicates, lignins, resins, acrylamides, and urethanes. Hundreds of different formulations have been developed within these classes. Of them, however, the silicates account for more than 90 percent of present chemical grout use, the others being limited for reasons of cost and toxicity.

In displacement (compaction) grouting, highly viscous soil, cement, and water displacement or compaction grout acts as a radial hydraulic jack which compresses the surrounding soil. The hardened grout mixture is a bulb of strong, relatively incompressible material. Displacement grouting can be used in partly saturated soil masses and loose materials containing void spaces. It is used to correct differential settlements or to provide underpinning and ground strengthening adjacent to open excavation or tunneling activities. Available equipment can develop up to 2.5 to 3.0 MN/m^2 pumping pressure, and zero slump grout can be pumped distances in excess of 30 m. To be effective, compaction grouting should not be undertaken at depths less than 1 to 2 m unless there is an overlying structure to provide confinement.

The basis for jet grouting is a special high speed water jet acting under a nozzle pressure of 15 to 70 MPa. Alternatively, poor soils can be removed by in-situ excavation and replaced by a mortar grout to form hard, impervious columns, panels or sheets. Jet grouted columns up to 3 m in diameter are possible. The use of air jetting in conjuction with grout jetting can yield diameters up to twice as great, for a given jet pressure, as the grout jet alone. The method offers the advantages of both close control over the zones being treated and applicability to clay as well as sands.

Evaluation of effectiveness; precise determination of exactly where all the grout went in the ground is usually not possible. Assessment of grouting effectiveness is usually made on the basis of grout take records and the results of in-situ tests and laboratory tests on recovered samples. Among the tests that have been used to evaluate grouting done for ground strengthening purposes are the cone penetration test, the standard penetration test, the pressuremeter test, plate load tests, and compression and shear wave velocity tests. Acoustic emission monitoring during grouting has been used recently as a means for detection of hydraulic fracturing and location of grout flow.

- Prewetting

If the collapsible soils are identified before construction begins, they can often be remedied by artificially wetting the soils before construction. This can be accomplished by sprinkling or ponding water at the ground surface, or using trenches or wells. This method is especially effective when attempting to stabilize deep deposits, but may not be satisfactory for shallow soils where loads from the proposed foundations may significantly increase the normal stress.

- Compaction with rollers or vehicles

Collapsible can be converted into excellent bearing material with little or no collapse potential by simply compacting them. Sometimes, this compaction may consist simply of passing heavy vibratory sheepsfoot rollers across the ground surface, preferably after first prewetting the soil.

- Vibrocompaction and Compaction Piles

These methods for deep compaction of cohesionless soils are characterized by the insertion of a cylindrical or torpedo-shaped probe into the ground followed by compaction by vibration during withdrawal. In a number of the methods a granular backfill is added so that a compacted sand or gravel column is left behind within a volume of sand compacted by vibration. Sinking of the probe to the desired treatment depth is usually accomplished using vibratory methods, often supplemented by water jets at the tip. Injection of air at the same time has been found to facilitate penetration to large depths. Upwards directed water jets along the sides has also been found helpful in some cases. Compaction piles of sand and gravel formed by these methods are also used in soft cohesive soils, in which case they function as compression and shear reinforcement. Ground treatment depths of 20 m can be achieved routinely by these methods. Depths in excess of 30 m can be attained in some cases.

A brief description of some of the more extensively used vibro-compaction methods is given below.

The Terraprobe method, uses a Foster Vibrodriver pile hammer on top of a 0.76 m dia. Open tubular probe (pipe pile) that is 3 to 5 m longer than the desired penetration depth. The unit operates at frequency of 15 Hz and a vertical amplitude of 10-25 mm. About 15 probes per hour can be done at spacings of 1 to 3 m. it is of marginal effectiveness in the upper 3 to 4 m of the zone to be densified.

Vibro-rods are also driven using a vibratory pile driving hammer. Several cycles of insertion and withdrawal are used in the densification process.

- Compaction by heavy tamping

Soil compaction by heavy tamping involves repeated dropping of heavy weights onto the ground surface. The method also termed dynamic compaction, dynamic consolidation, or pounding. When applied to partly saturated soils, the densification process is essentially the same as that for impact (Proctor) compaction in the laboratory. In the case of saturated cohesionless soils liquefaction can be induced, and the densification process is similar to that accompanying blasting and vibrocompaction. The effectiveness of method in saturated, fine-grained soils is uncertain; both successes and failures have been reported. It would appear that in such materials a breakdown in the soil structure, the generation of excess pore water pressures, and the formation of drainage channels by fissuring may be required. Heavy tamping has been especially effective for compaction of waste and rubble fills.

The pounders used for heavy tamping may be concrete blocks, steel plates, or thick steel shells filled with concrete or sand and may range from one or two up to 200 tons in weight. Drop heights up to 40 m have been used. The pounders are usually square or circular in plan and have dimensions of up to a few meters depending on weight required, material, and the dynamic bearing capacity at surface of the ground to be treated. More streamlined shapes have been used for underwater tamping.

For large area compaction several repetitions at points spaced several meters apart in a grid pattern are applied. A typical treatment will result in an average of 2 to 3 blows/m². Two or three coverages of an area may be required, separated by time intervals dependent on the rate of dissipation of excess pore water pressure and strength regain. The time interval required between coverages may range from days or freely draining coarse sands to weeks for finer-grained soils. The ground surface is usually leveled between covarages. To insure uniformity and high density in the near surface zone, surface "ironing" is used. Small impacts by the pounder are made over the entire surface. Surface settlements may be from two to five percent of the thickness of the zone being densified per coverage.

When heavy tamping is use to prepare ground for support of relatively light (low rise) structures on shallow foundations, treatment is sometimes made only at footing locations. This can be an economical and effective means for minimizing total and differential settlements.

The depth of influence should depend on factors in addition to the impact energy. Soil type might be expected to be the most important. A crane drop is less efficient than a free drop. The presence of soft layers has a damping influence on the dynamic forces. Definition of depth of influence is itself subjective and depends both on the method of measurement and the engineer's definition of what constitutes a measurable ground improvement.

The amount of soil improvement that develops in any case depends on soil type, water conditions, and input energy per unit area. Finer-grained soils cannot be strengthened to the same level as can coarser materials. Soft layers of clay and peat inhibit high compaction of adjacent cohesionless material because of their flexibility. A review of available cases suggests that there may be a definable maximum level of improvement.

An additional concern relative to heavy tamping, and blasting as well, is whether damage may occur to facilities located beyond the edges of the area being densified because of the large impact energies. Measurements of vibration frequencies have given values in the range of 2 – 20 Hz.

- Vibroflotation

This technique consists of penetrating the soil with a vibrating probe equipped with water jets. The water softens the soil and the vibrations help the collapse process. The vertical hole formed by the vibroflot is also filled with gravel.

The equipment for the vibroflotation consists of three main parts: the vibrator, extension tubes, and a supporting crane. The vibrator is a hollow steel tube containing an eccentric weight mounted on a vertical axis in the lower part so as to give a horizontal vibration. Vibrator diameters are in the range of 350 to 450 mm and the length is about 5 m including a special flexible coupling. One vibrator weighs about 20 kN. Units developing centrifugal forces up to 160 kN and variable amplitudes of up to 25 mm are available. Most usual operating frequencies are 30 Hz and 50 Hz. The extension tubes have a slightly smaller diameter than the vibrator and a length dependent on the depth of penetration required.

Vibroflot sinking rates of 1 to 2 m/min and withdrawal/compaction rates of about 0.3 m/min are typical. Water pressures of up to 0.8 MPa and flow rates up to 3,000 l/min may be used to facilitate penetrate. Sand backfill is consumed at a rate of up to 1.5 m^3/m during the compaction process. The zone of improved soil extends from 1.5 m to 4 m from the vibrator, depending upon soil type and vibroflot power.

- Deep blasting combined with prewetting

This is the stabilizing the collapsible soil by detonating buried explosives.

Deep compaction by detonating of buried explosives can provide a rapid, low cost means for soil improvement in some cases. The general procedure consists of:

1- Installation of pipe by jetting, vibration, or other means to desire depth of charge placement
2- Placement of charge in pipe
3- Backfilling the hole
4- Detonation of charges according to a pre-established pattern.

In some cases the pipe is withdrawn prior to detonation of the charges. In others it is reclaimed after the blast, a new section is welded to the bottom, and it can be used again. The explosives used include dynamite, TNT, and ammonite.

Saturated, clean sands are well-suited for densification by blasting. Success in any case depends on the ability of the shock wave generated by the blast to break down the initial structure, and create a liquefaction condition for a sufficient period to enable particles to rearrange themselves in a denser packing. It follows, therefore, that the stronger the sand initially, the larger the charges that will be required for effective densification. Thus, the greater the depth to which densification is needed and the higher the initial equivalent relative density, the greater the explosive energy required.

There appear to be no generally accepted theoretical design procedures for densification by blasting, and field trials are usually used prior to production blasting. The general guidelines are as follows:

1- Charge size: <1 to 12 kg
2- Depth of burial: >1/4 depth to bottom of layer to be treated; 1/2 to 3/4 of depth common.
3- Charge spacing in plan: 5 – 15 m
4- Number of coverages: 1 – 5 with 2 – 3 usual. Each coverage consists of a number of individual charges. Successive coverages are usually separated by hours or days.
5- Total explosive use: 8 – 150 gr/m^3, 10 – 30 gr/m^3 typical
6- Surface Settlement: 2 to 10% of layer thickness.

It has been possible by blasting to densify sands to equivalent relative densities of 75 to 80 percent. In some case, however, the results may be erratic, initially dense zones may be loosened, and the method is not likely to be effective in the upper one or two meters below the ground surface. Typical behavior may be summarized as follows:

- Almost immediate settlements of the ground surface, with little further settlements with time.
- Initially loose zones show little immediate change in penetration resistance. Penetration resistance increases slowly with time until after several weeks the material indicates a marked improvement in the properties compared to its initial condition.
- Zones which are initially very dense may be permanently loosened or weakened by the blast; however, the resultant condition is still likely to be satisfactory.

- Ultimately, an effective blasting program results in a deposit in which all the initially loose zones have been suitably improved.

A hydro-blasting technique has been used very successfully and economically for compaction of collapsible loess deposits. Although collapse of the loess can often be accomplished by flooding alone, it has been found that more uniform results can be achieved more quickly and economically by this method. The procedure consists of first cutting a contour trench 0.2 m to 0.4 m wide and several meters deep around the perimeter of the area to be densified. Boreholes spaced a few meters apart in a grid pattern are then used to pump water into the loess, over a period of several days, ideally until the water content is increased to above the liquid limit. Slurry walls or plastic membranes can be installed to prevent lateral migration of the water and softening of adjacent ground.

Explosive charges of about 5 kg each are then inserted at spacings at three to six meters in grid patterns and detonated. Surface settlements of up to 10 percent of the layer thickness and reduction in porosity of several percentage points are not uncommon. Areas of 1000 m^2 to 10,000 m^2 involving 10,000 – 100,000 m^3 of loess can be treated at one time.

- Controlled wetting

This method is similar with prewetting method, however, it differs in that the wetting is much more controlled and often concentrated in certain areas. This would be used most often remedial measure to correct differential settlements that have accidentally occurred as a result of localize wetting.

3.1. Fly Ash Stabilization

To meet the increasing demand for electricity, large quantities of coal are being burnt in thermal power stations. Combustion of coal results in a residue consisting of inorganic and organic constituents which are not burned completely. The inorganic mineral constituent from ash is generally fly ash which is about 80% of this ash. Environmentally safe disposal of large quantities of ash is not only tedious but also expensive. To avoid the problems about disposal, fly ash has to be utilized. The use of fly ash as a soil-stabilizing agent is beneficial for improving the engineering properties of the soil, while at the same time it provides an opportunity for the utilization of an industrial waste that will otherwise require costly disposal (Ferreira et al. 2003; Nalbantoğlu and Güçbilmez, 2002; Sivapullaiah et al. 1998).

Fly ash produced from the burning of pulverized coal in a coal-fired boiler is a fine-grained, powdery particulate material that is carried off in the flue gas and collected from the flue gas by means of electrostatic precipitators (TFHRC, 2003; Vassilev et al. 2003).

Fly ash is useful in many construction applications because it is a pozzolan, meaning it is a siliceous or alimino-siliceous material which in itself possess little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties (ASTM D5239-92, 1993).

A microscopic view of fly ash reveals mainly glassy spheres with some crystalline and carbonaceous matter. The principal chemical constituents are silica (SiO_2), alumina (Al_2O_3), ferric oxide (Fe_2O_3), and calcium oxide (CaO). Other components are magnesium oxide (MgO), sulfur trioxide (SO_3), titanium oxide (TiO_2), alkalies (Na_2O and K_2O), phosphorous oxide (P_2O_5), and carbon (related to loss on ignition). Water added to fly ash usually creates an alkaline solution, with pH in the range 6 to 11.

Because of the variations in coals from different sources, as well as the differences in the design of coal-fired boilers, not all the fly ash is the same. Factors affecting the physical, chemical, and engineering properties of fly ash include (TFHRC, 2003):

- Coal type and purity
- Degree of pulverization
- Boiler type and operation
- Collection and stockpiling methods

Two classes of fly ash are defined in ASTM C 618: Class F fly ash, and Class C fly ash. Class F fly ash is normally produced from burning anthracite or bituminous coal. This class fly ash has pozzolanic properties. Class C fly ash is normally produced from burning lignite or sub-bituminous coal. This class of fly ash, in addition to having pozzolanic properties, also has some self-cementing properties, meaning that it has ability to harden and gain strength in the presence of water alone. Typical chemical compositions of Class F and Class C fly ashes are given in Table 3.1.

Table 3.1. Typical Chemical Compositions of Class F and Class C Fly Ashes (expressed as percent by weight) (TFHRC, 2003).

Component	Class F Fly Ash	Class C Fly Ash
SiO_2	20 – 60	40 – 60
Al_2O_3	5 – 35	10 – 30
Fe_2O_3	10 – 40	4 – 15
CaO	1 – 12	5 – 30
MgO	0 – 5	1 – 6
SO_3	0 – 4	0 – 4
Na_2O	0 – 4	0 – 6
K_2O	0 – 3	0 – 4
Loss on Ignition	0 – 15	0 – 3

ASTM D 5239 classifies fly ashes into three categories according to their soil stabilization performances:

1) Non Self-Cementing (Class F) Fly Ash Stabilization

Non self-cementing fly ash, by itself, has little effect on soil stabilization. It is a poor source of calcium and magnesium ions. The particle size of fly ash may exceed that of the voids in fine-grained soils, precluding its use as a filler. However, this fly ash in poorly graded sandy soils may be a suitable filler and, as such, may aid in compaction, may increase density, and may decrease permeability.

2) Non Self-Cementing (Class F) Fly Ash Mixed with Cement or Lime

The advantage of adding fly ash to fine-grained soils, along with cement or lime, is for its pozzolanic properties and improved soil texture. The use of this fly ash is suitable with clays requiring lime modification, provided lime is added to promote the pozzolanic reaction.

3) Self-Cementing (Class C) Fly Ash Stabilization

This fly ash is a better source of calcium and magnesium ions although not as good as lime or Portland cement. Self-cementing fly ash contains varying amounts of free (uncombined) lime (0 to 7% CaO by weight) that can provide cation exchange and ion crowding to fine-grained soils when used in significant amounts. It has been used successfully to control swell potential of expansive soils. It has also been used to stabilize coarse-grained soils.

Effect of fly ash can be inspected in short and long term considering fly ash reactions (Türker, 2001).

a) Short term fly ash reactions

If soil is not in good gradation and there is a gap of silt sized particles in gradation curve, fly ash application will improve soil gradation. Immediate effect of the introduction of the fly ash to the soil (including the lime, CaO, already present in the fly ash) is to cause flocculation and agglomeration of the clay particles due to ion exchange at the surfaces of the soil particles.

Hydration chemistry of the fly ash is governed by the amount and type of calcium compounds in the fly ash and type of calcium compounds in the fly ash and the extent to which calcium exist in crystalline form. Due to textural changes caused by these reactions within the soil, the strength and the moisture stability of the soil is improved. These improvements are reflected improved workability and immediate strength development.

b) Long term fly ash reactions

The long term reactions are accompanied over a period of time (many weeks, months, or even years may be required for completion of these reactions) depending upon the rate of chemical breakdown and hydration of silicates and aluminates. This result in further amelioration and binds the soil grains together by formation of the cementitous materials. In order for cementation to occur there must be sufficient source of pozzolans available. Pozzolans are a source of silica and alumina with high surface area which are source of silica available for hydration by alkali earth hydroxides to form cementitous products in the presence of moisture at ordinary temperatures.

Fly ash stabilization is used in soils, which does not have sufficient pozzolans for lime stabilization. All the silica present in fly ash is not readily available for reaction with lime. The significance of fly ash is largely because of it is a source of reactive silica. This reactive silica in fly ash appears to be due to presence of a special microstructure of quartz named silica, symbolized as W, having a micro-amorphous fibrous silica structure along with an amorphous silica structure. The rest of silica present in fly ash is in crystalline form of quartz or in association with alumina as mullite and is not readily available for reactions with free lime (Türker, 2001).

3.2. Desulphogypsum Stabilization

In the last three decades, there has been a continuous effort by electric utility companies to reduce sulfur dioxide (SO_2) emissions from coal burning power plants (Sahu et al. 2002). To achieve the desired concentration of sulfur dioxide in the exhaust gases, they are processed in desulphurization plant. The most widely used method of removal of sulfur dioxide is the treatment of the flue gas with calcium oxide (CaO). In this process, also known as flue gas desulphurization (FGD), calcium reacts with sulfur dioxide to produce hannebachite ($CaSO_3.1/2H_2O$) and/or gypsum ($CaSO_4.2H_2O$). The resulting gypsum is named desulphogypsum (Oman et al. 2002). The overall FGD reaction can be represented by the following (Chen, 1995):

$$CaO + H_2O \longrightarrow Ca(OH)_2$$
$$SO_2 + H_2O \longrightarrow H_2SO_3$$
$$H_2SO_3 + Ca(OH)_2 \longrightarrow CaSO_3.2H_2O$$
$$CaSO_3.2H_2O + 1/2O_2 \longrightarrow CaSO_4.2H_2O$$

FGD process generate voluminous desulphogypsum solid wastes that are usually landfilled, occupying thousands of acres of land and creating serious land pollution problems (Tao et al. 2001). The American Coal Ash Association reported for United States that less than 10% of desulphogypsum is currently used beneficially for gypsum binders, plasters and plasterboards manufacture, as well as an additive in Portland cement production (Clark et al. 2001; Galos et al. 2003). Utilization of desulphogypsum in geotechnical applications will be useful in decreasing the excessive stocks which cause environmental pollution, besides it will also provide a new and economical way to improve the engineering properties of soils.

Having the same chemical composition with natural gypsum, desulphogypsum contains impurities such as the finer fractions of fly ash. These impurities may be located in the crystal structure of desulphogypsum or may be sticked to the surface of the crystal structure. Chemical composition of these impurities vary according to the type and properties of the fuel and sorbent used, and the type of boiler (Galos et al.2003; Özkul, 2000; Sahu et al. 2002).

In general, including its impurities, desulphogypsum can be characterized as an alkaline material consisting of excess sorbent (either calcitic or dolomitic limestone), calcium oxide, calcium hydroxide (portlandite), calcium sulfate (anhydrite), calcium sulfite, magnesium sulfate (epsomite), magnesium oxide (periclase), and fly ash (Çetiner, 2004).

Various forms of lime have been utilized as a soil stabilizing agent for many years including products with varying degrees of purity. However, the most commonly used products are hydrated high calcium lime $Ca(OH)_2.MgO$, calcitic quick lime CaO, and dolomitic quick lime CaO.MgO. Quick lime is used widely for soil stabilization. The type of the lime used as a stabilizing agent varies from country to country. Although using quick lime is more popular in Europe, hydrated lime is used mainly for stabilization but proportion of quick lime that is used increased in recent years (İpek, 1998). One third of the desulphogypsum consists of calcitic quick lime.

When lime added to a soil, hydration of lime causes an immediate drying of the soil. Anhydrous quick lime will have a more pronounced drying effect than hydrated lime. Consequently, lime can prove to be an effective construction expedient for drying the wet sites (İpek, 1998).

The principal use of the addition of fly ash and desulphogypsum to soil is like addition of lime to soil. Dry hydrated lime can be spread uniformly by a mechanical spreader or from bags emptied in piles and leveled off by a drag pulled by a tractor. After sprinkling dry lime with water, preliminary mixing is required to distribute the lime thoroughly throughout the soil to the proper depth and width and to pulverize the soil to 50 mm. During mixing, water is added to bring the soil slightly above the optimum moisture content (Bell, 1993).

Lime slurries of varying concentrations, depending on the percentage of lime required and the optimum moisture content, are also applied to the soil. At higher concentrations than 30% there is difficulty in pumping and handling the slurry spray bars. Forty percent is the maximum pumpable slurry (Bell, 1993).

The mixtures should be compacted to high density in order to develop maximum strength and stability. This necessitates compacting at or near the optimum moisture content (Bell, 1993).

CHAPTER 4

EXPERIMENTAL STUDY

4.1. Purpose

The purpose of the experimental study is to investigate the effects of the addition of fly ash and desulphogypsum on grain size distribution, Atterberg limits, unconfined compressive strength, and collapse potential of a collapsible soil; and to investigate the effect of curing on collapse potential and unconfined compressive strength of a collapsible soil treated with fly ash and desulphogypsum.

4.2. Material

Collapsible Soil: Collapsible soil was obtained from the area of Çayırhan Thermal Power Plant. The soil is sampled undisturbed according to the TS 1901 Methods for Boring and Obtaining Disturbed and Undisturbed Samples for Civil Engineering Purposes.

The chemical analysis of collapsible soil was done in the laboratory of the Department of Chemistry in METU. The results of the chemical analyses are presented in Table 4.1.

Table 4.1. Results of the Chemical Analysis of Collapsible Soil (expressed as percent by weight)

Component	Collapsible Soil
$CaCO_3$	24.30
CaO	13.65
SiO_2	53.80
R_2O_3	7.20
Loss on Ignition	1.05

Fly Ash: Fly ash was taken from Çayırhan Thermal Power Plant. Fly ash was passed through No. 4 sieve before usage. Fly ash is Class C, its specific gravity is 2.13.

Desulphogypsum: Desulphogypsum was taken from Çayırhan Thermal Power Plant. Desulphogypsum was passed through No. 4 sieve before usage. Specific gravity of desulphogypsum is 3.24.

The chemical analyses of Çayırhan fly ash and Çayırhan desulphogypsum were done by 'Cement Producers Association of Türkiye'. The results of the chemical analyses are presented in Table 4.2.

Table 4.2. Results of the Chemical Analysis of Çayırhan Fly Ash and Desulphogypsum (expressed as percent by weight)

Component	Çayırhan Fly Ash	Çayırhan Desulphogypsum
SiO_2	50.38	2.03
Al_2O_3	14.06	0.52
Fe_2O_3	9.90	0.21
CaO	13.25	31.91
MgO	1.20	0.42
SO_3	3.16	43.13
Na_2O	3.18	-
K_2O	1.97	-
TiO_2	0.90	-
P_2O_5	0.58	-
Loss on Ignition	0.86	20.88

4.3. Preparation of Samples

The collapsible soil been used in this study is designated as 'Sample A'. In the beginning of preliminary studies the soil is tested if it is collapsible or not. After concluding the soil is collapsible, fly ash and desulphogypsum were examined if they are capable for stabilization of collapsible soils.

Disturbed samples of collapsible soil and fly ash used in this study were oven-dried for one day at 60°C; desulphogypsum was oven-dried for one day at 30°C, and were ground so that they could pass through No. 4 sieve. Each sample was prepared by mixing a calculated amount of stabilizer with Sample A to obtain a sample with predetermined percentage of stabilizer which varied from 0 to 25 percent (by dry weight of the sample) for fly ash and desulphogypsum (Figure 4.1.).

To prepare the samples the predetermined amount of soil and stabilizers were mixed using a trowel. Each time only 500 gr of each sample was mixed, as mixing higher amounts could prevent the particles from distributing uniformly in the mixtures. Then the water needed for the optimum moisture content was added to the sample (Figure 4.1).

For the experiments on cured samples, the samples which were prepared according to the above procedure were put in a plastic bag to prevent loss of moisture and set to cure in the desiccator for 7 days and 28 days. The curing temperature in the desiccators was approximately 23^{o}C. After 7 days and 28 days, the cured samples were taken out of the plastic bag and prepared for the Collapse and Unconfined Compressive Tests.

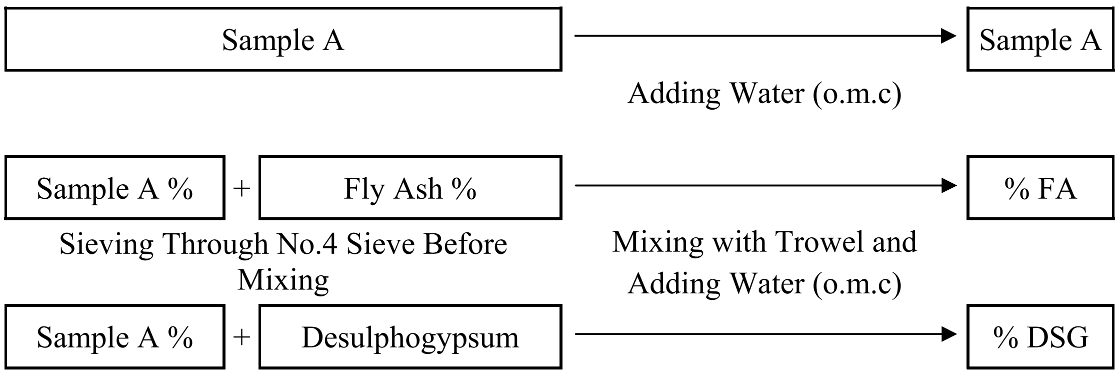

Figure 4.1. Preparations of Samples

4.4. Sample Properties

To determine the sample properties sieve analyses, hydrometer tests, Atterberg limit tests, and specific gravity tests were applied to the samples according to the test procedures specified in ASTM D 2435.

The sample properties are tabulated in Table 4.3.

Clay and silt fractions of the samples could not be determined by hydrometer analyses due to the precipitation of the samples at the bottom of the hydrometer flask within the first few hours of the tests. Hence, grain size distribution curves, and clay and silt fractions are not available.

Soil classification is done according to the Unified Soil Classification System (Figure 4.2). The samples are around the A-line. The increase of fly ash in the samples moves the points towards right side on USCS chart. The increase of desulphogypsum moves the points to the left side on USCS chart.

Table 4.3. Sample Properties

Sample	G_s	LL (%)	PL (%)	PI (%)	SL (%)	SI (%)	USCS	o.m.c. (%)	$\rho_{d\,max}$ (Mg/m^3)
100% A	2.76	44.35	22.81	21.54	31.09	13.26	CL	25	1.558
5% FA	2.74	43.46	25.94	17.52	28.68	14.78	CL	19	1.661
10% FA	2.72	41.97	24.73	17.24	26.91	15.06	CL	19	1.664
15% FA	2.69	41.48	24.34	17.14	26.34	15.14	CL	19	1.640
20% FA	2.66	41.05	24.08	16.97	24.05	17.00	CL	19	1.572
25% FA	2.60	40.66	23.81	16.85	21.28	19.38	CL	19	1.550
5% DSG	2.78	39.23	24.10	15.13	27.29	11.94	CL	20	1.635
10% DSG	2.79	40.32	25.00	15.32	25.83	14.49	CL	18	1.585
15% DSG	2.83	41.92	26.18	15.74	25.45	16.47	ML	17	1.550
20% DSG	2.84	42.83	26.95	15.88	23.53	19.30	ML	14	1.480
25% DSG	2.88	44.87	28.18	16.69	23.14	21.23	ML	13	1.473

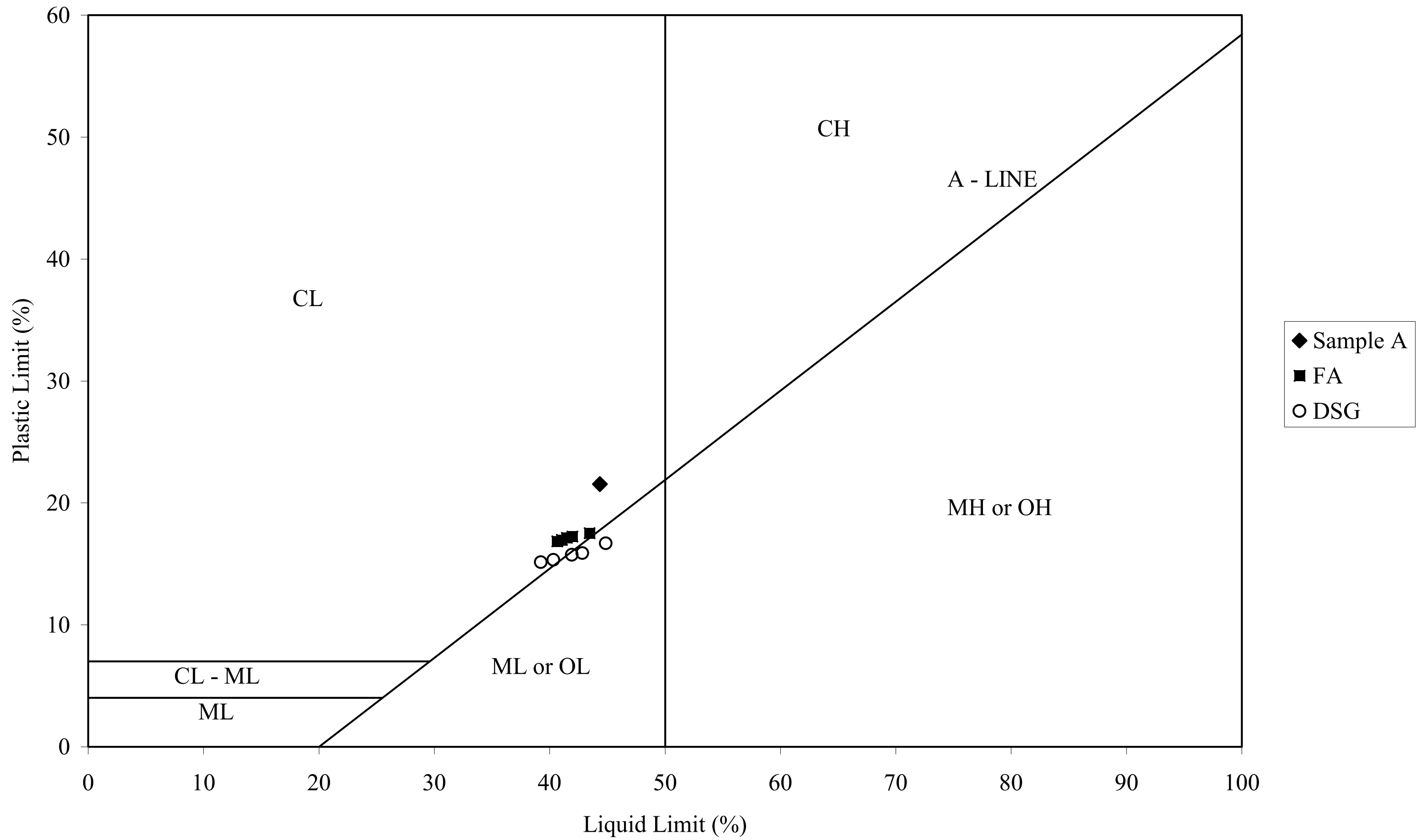

Figure 4.2. Plasticity Chart Unified System

A: Collapsible Soil Sample;

FA: Fly Ash;

DSG: Desulphogypsum

Naming is explained with the following two examples:

5% FA $\longrightarrow$ 95% Sample A + 5% Fly Ash

10% DSG $\longrightarrow$ 90% Sample A + 10% Desulphogypsum

Grain size distribution curves of the fly ash and desulphogypsum added samples are plotted separately (Figure 4.3, Figure 4.4). Grain size distribution curve of Sample A is plotted on both of the graphs to be able to examine the shifting of the curves due to the addition of the stabilizers.

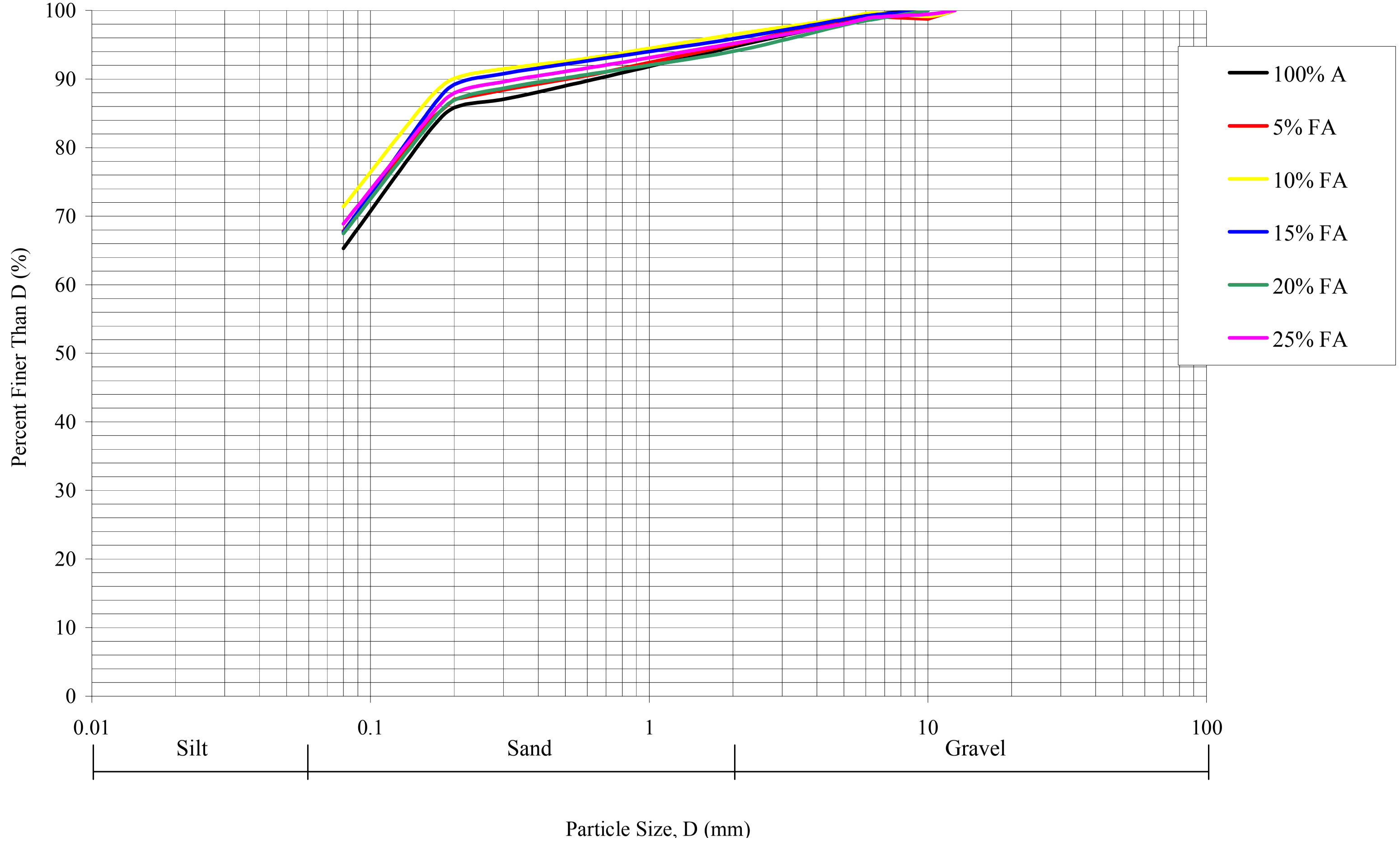

Figure 4.3. Grain Size Distribution Curve of Fly Ash Added Samples

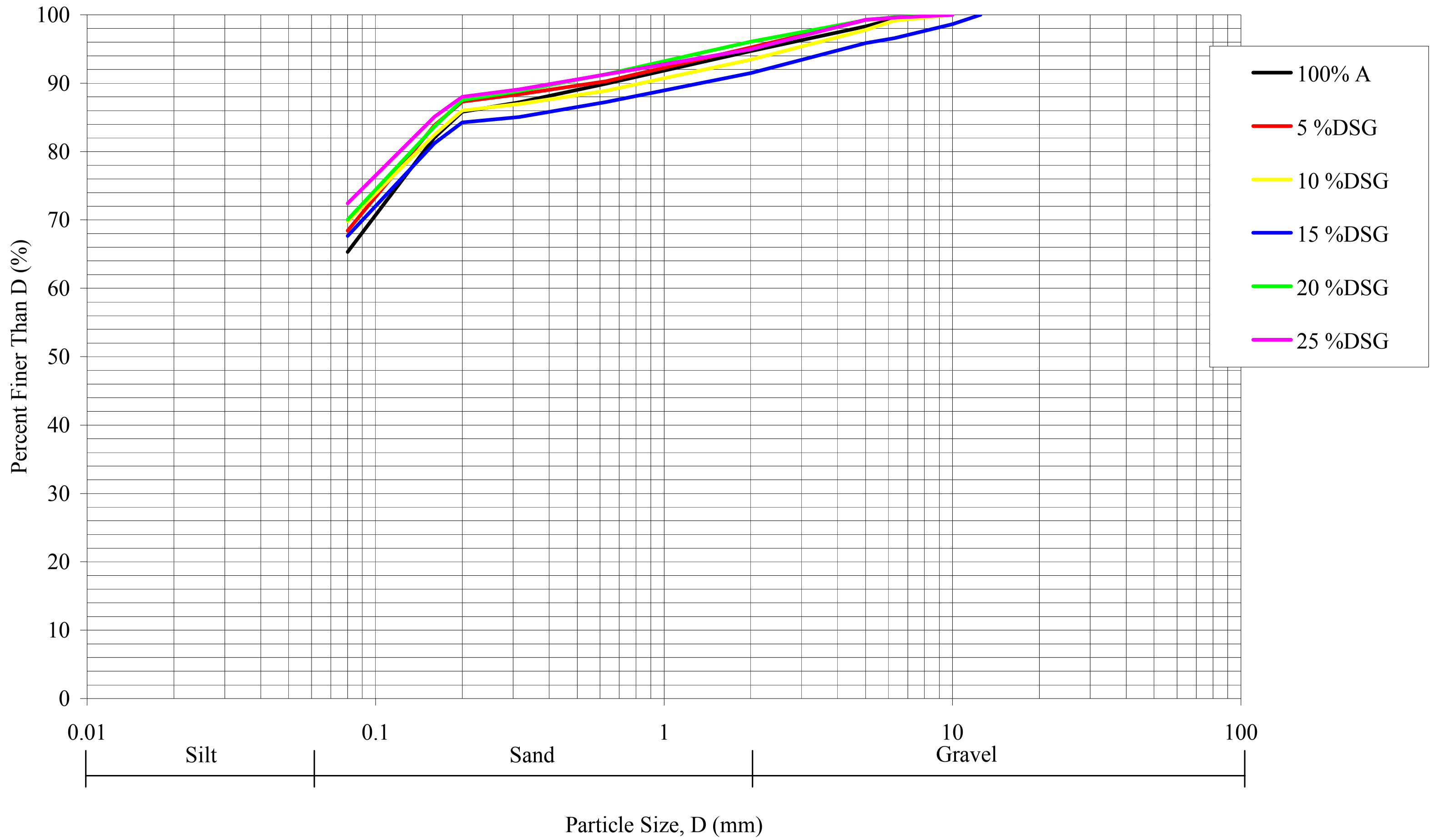

Figure 4.4. Grain Size Distribution Curve of Desulphogypsum Added Samples

4.5. Test Procedure

Single oedometer test is used for the determination of collapse potential. In order to apply this method the samples were prepared as described in Chapter 4.3. The samples were compacted in a guide ring satisfying the maximum dry density at optimum moisture content. The samples were then transferred into the consolidation ring with the help of the guide ring. The compaction and transferring procedure is shown in Figure 4.5. First the calculated amount of soil sample was placed in the guide ring and the piston was placed on the guide ring with Part C of it in contact with the sample. Then the sample was compressed by applying pressure from the top of the piston (Part A) using a hydraulic jack till Part B of the piston came into contact with the guide ring (Figure 4.5, Step 1). After compaction finished the piston was removed and the guide ring, with the sample in it, was placed on the consolidation ring. The piston was again placed on the guide ring, this time with Part A of it in contact with the sample. By applying a strong and immediate push with hand from Part C of the piston the sample was pushed through the guide ring into the consolidation ring (Figure 4.5, Step 2).

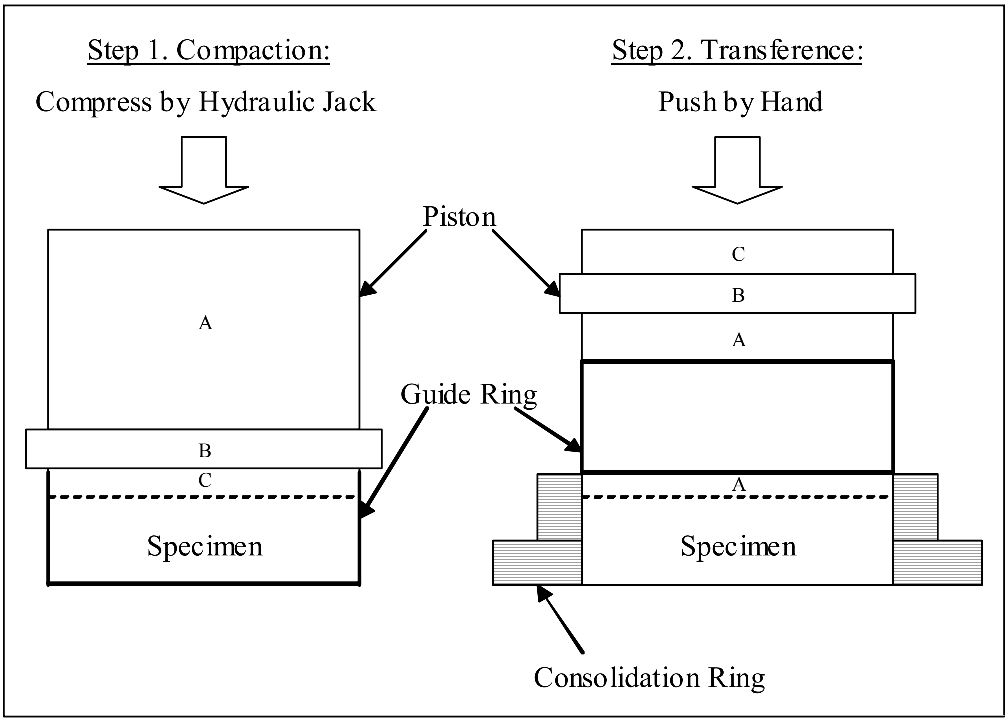

Figure 4.5. Static Compaction Setup (Çetiner, 2004)

4.5.1. Single Oedometer Test

In this study single oedometer test is used according to the "ASTM D 5333 – 92 Standard Test Method for Measurement of Collapse Potential of Soils". In the standard, an undisturbed sample is consolidated in its natural moisture content with the load increments on every hour until reaching 200 kPa. The soil is inundated at 200 kPa, and the strain is observed for 24 hours. After 24 hours, the consolidation is continued. The details of the test are given in the following paragraphs.

The sample, which was compacted in the consolidation ring as explained above, was placed in the oedometer after placing dry filter papers on top and bottom of it. In placing the consolidation ring into the oedometer, air-dry porous stones were also placed on top and bottom of the sample (Figure 4.6). Then, the oedometer was mounted and the dial gauge was adjusted to zero reading. The sample was protected from the dry air by using a wet towel. The sample was loaded up to 5 kPa for 5 minutes, after that the load increments per hour were as 12.5, 25, 50, 100, 200 kPa. After reaching and loading 200 kPa for an hour, the sample was inundated by providing water through standpipes and by pouring water directly from the top of the oedometer. Collapse of the sample started right after the inundation of water. As collapse continued deflections of the dial gauge was recorded. After waiting for 24 hours, consolidation was continued with loads of 400, 800, 1600 kPa. The collapse potential was calculated from the following expression:

Collapse Potential (%) = $\Delta H/H*100$

where ΔH = Change in initial height (H) of the sample

H = Initial height of the sample

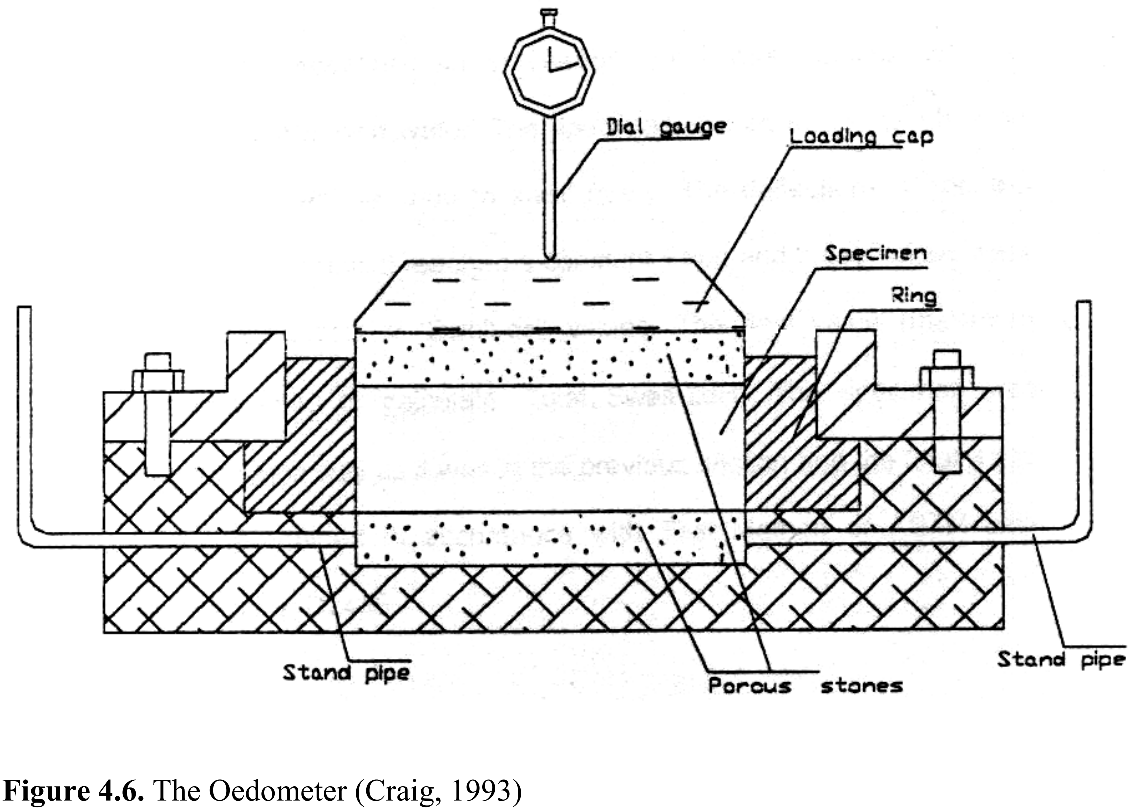

Figure 4.6. The Oedometer (Craig, 1993)

After consolidation was complete the oedometer was dismantled and the consolidation ring was taken out. The filter papers were separated from the surface of the sample. The weight of the sample was measured and the sample was put in the oven to find its dry weight for the final water content determination.

4.6. Experimental Program

Upon the completion of the preliminary tests, the maximum and minimum amount of stabilizers to be added to Sample A were decided. Tests were decided to be performed on seventeen samples (Table 4.4).

Table 4.4. Samples Used in the Experimental Study

Sample A	Fly Ash (FA)	Desulphogypsum (DSG)
100% Sample A (undisturbed)	5% FA + 95% A (5%FA)	5% DSG + 95% A (5%DSG)
100% Sample A (max. dry density & o.m.c.)	10% FA + 90% A (10%FA)	10% DSG + 90% A (10%DSG)
	15% FA + 85% A (15%FA)	15% DSG + 85% A (15%DSG)
	20% FA + 80% A (20%FA)	20% DSG + 80% A (20%DSG)
	25% FA + 75% A (25%FA)	25% DSG + 75% A (25%DSG)

Experimental study was conducted in four phases:

1) Hydrometer tests, Atterberg limit tests, and specific gravity tests were applied to the samples.
2) Collapse and Unconfined Compressive tests were applied to the samples under the condition of no curing.

3) Collapse and Unconfined Compressive tests were applied to the samples after curing was applied for 7 days.

4) Collapse and Unconfined Compressive tests were applied to the samples after curing was applied for 28 days.

4.7. Test Results

The results of the liquid limit, plastic limit, and shrinkage limit tests are presented in Figures 4.7, 4.8 and 4.9 respectively for the fly ash, and desulphogypsum.

Variations of plasticity index (LL - PL) and shrinkage index (LL - SL) for the fly ash, and desulphogypsum added samples are presented in Figures 4.10 and 4.11 respectively.

The effects of the addition of stabilizers on specific gravity (Gs) are given in Figure 4.12.

The effects of the addition of fly ash and desulphogypsum on maximum dry density (ρ_{dmax}) and optimum moisture content (o.m.c.) are given in Figures 4.13 and 4.14 respectively.

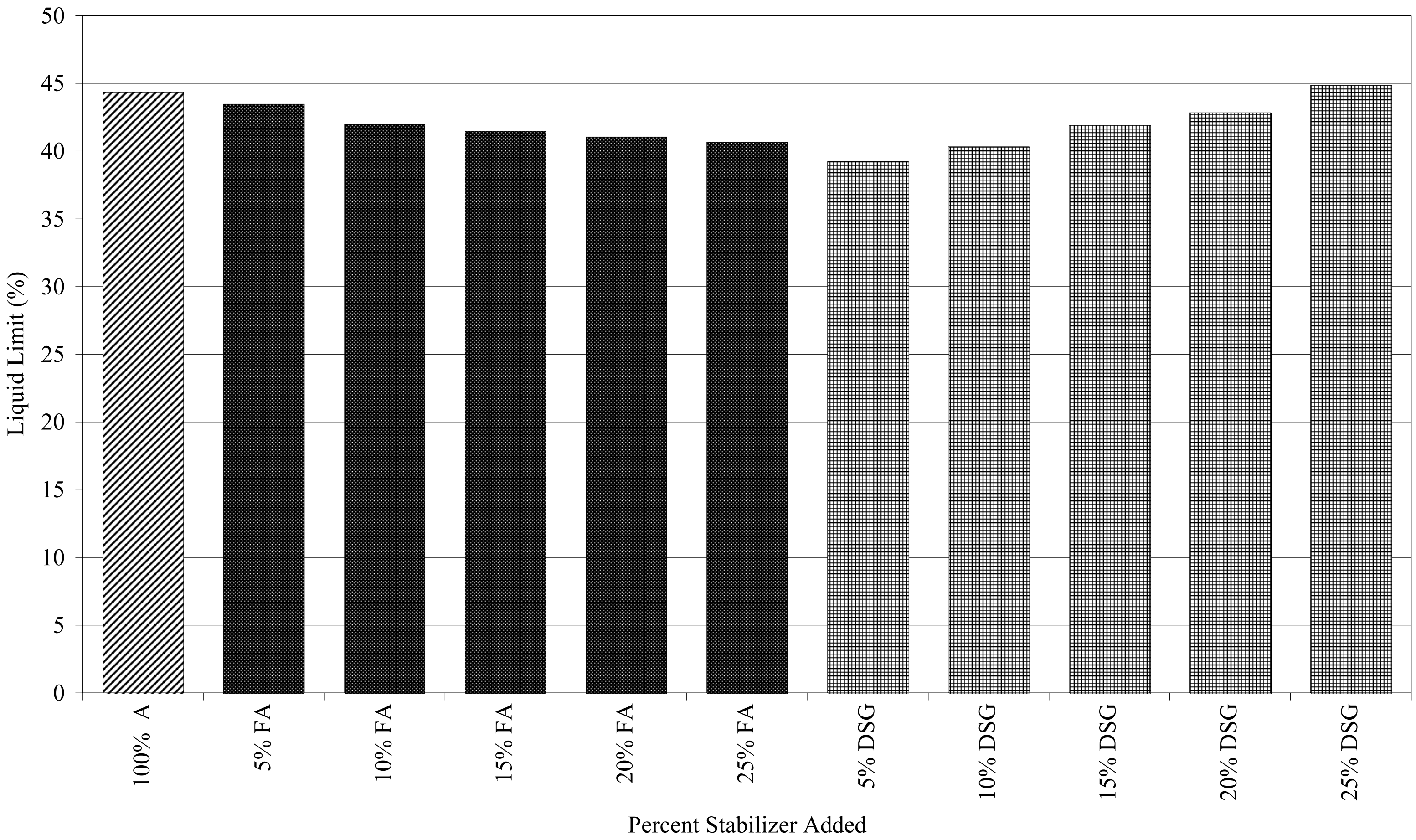

Figure 4.7. Effects of Addition of Fly Ash and Desulphogypsum on the Liquid Limit

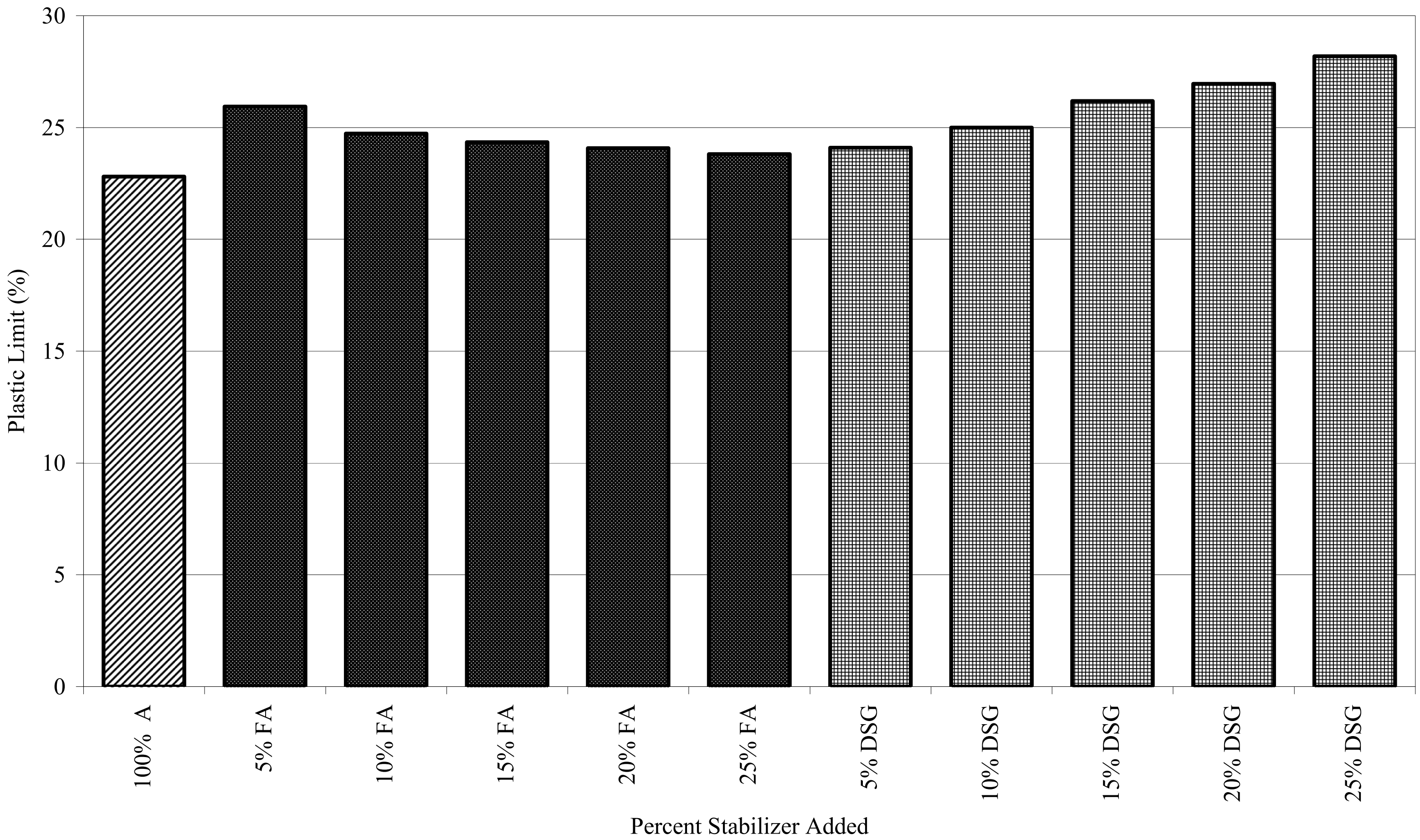

Figure 4.8. Effects of Addition of Fly Ash and Desulphogypsum on the Plastic Limit

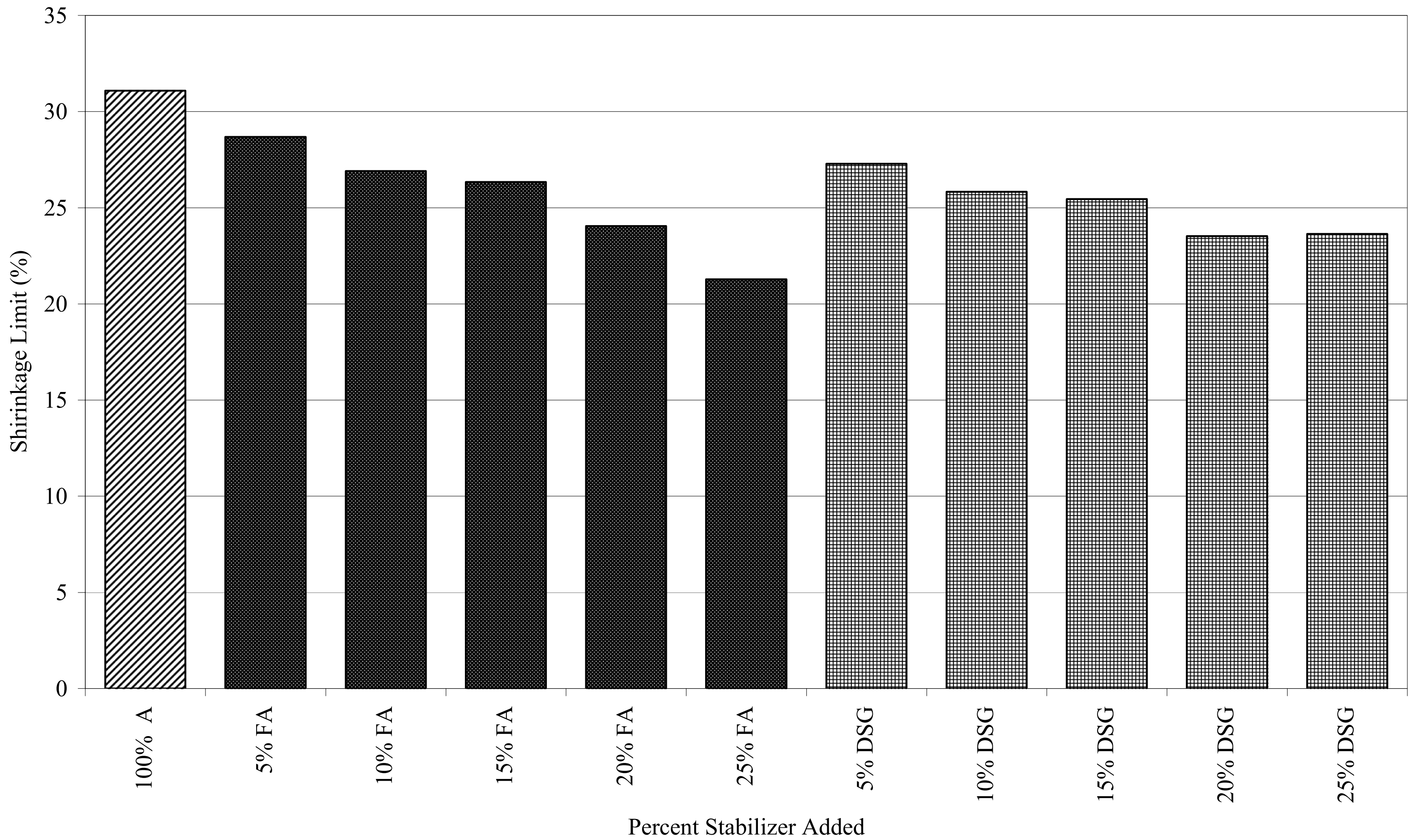

Figure 4.9. Effects of Addition of Fly Ash and Desulphogypsum on the Shrinkage Limit

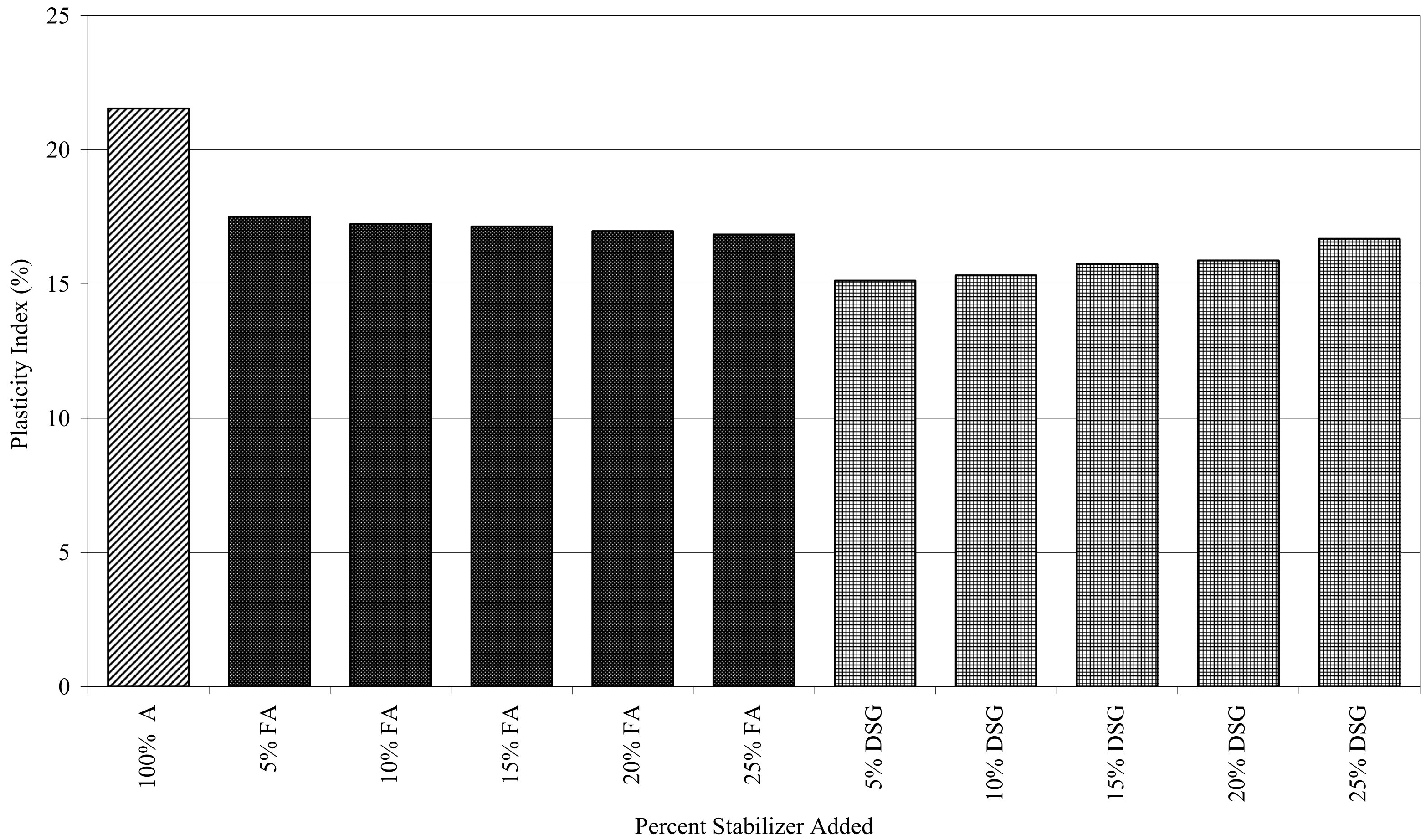

Figure 4.10. Effects of Addition of Fly Ash and Desulphogypsum on the Plasticity Index

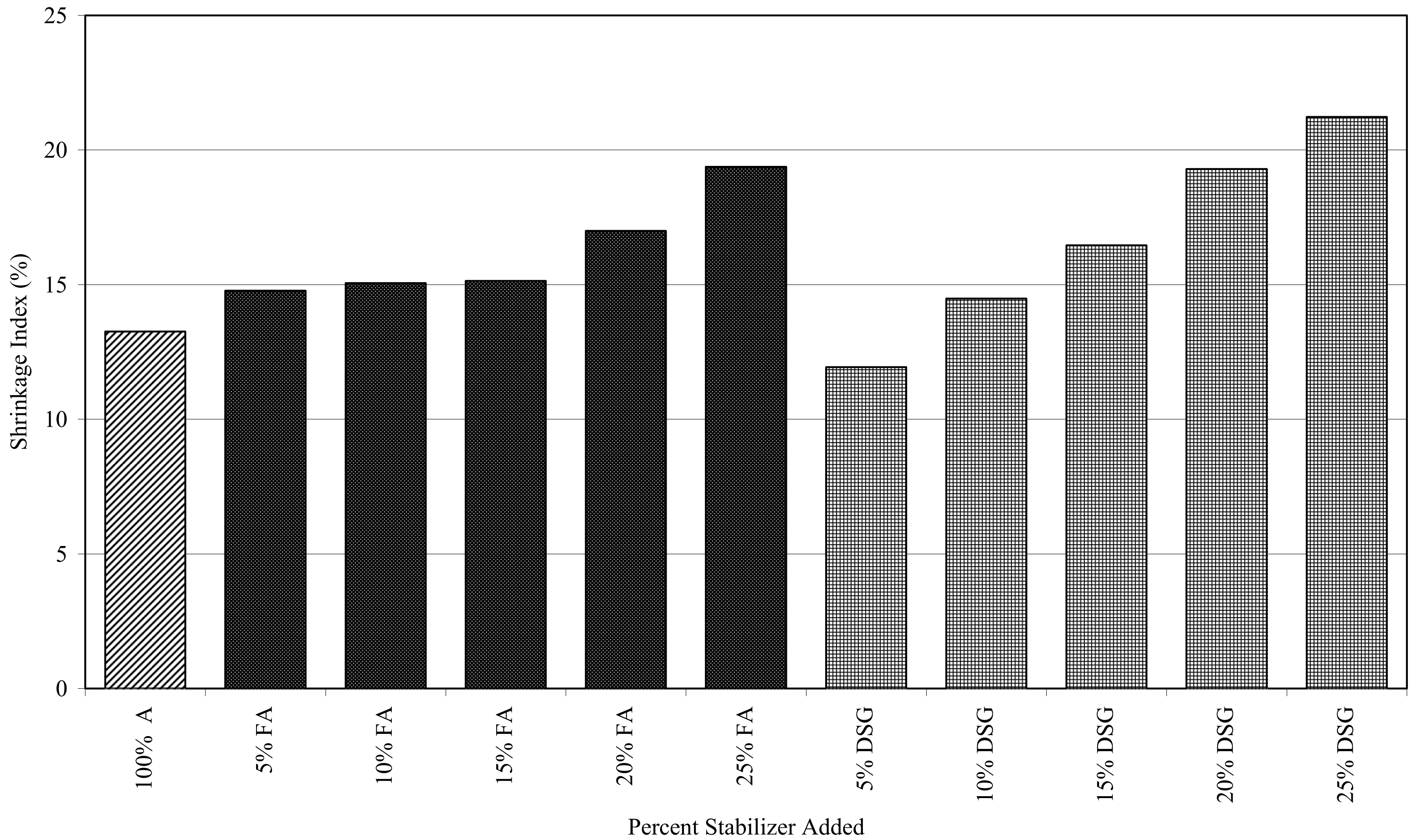

Figure 4.11. Effects of Addition of Fly Ash and Desulphogypsum on the Shrinkage Index

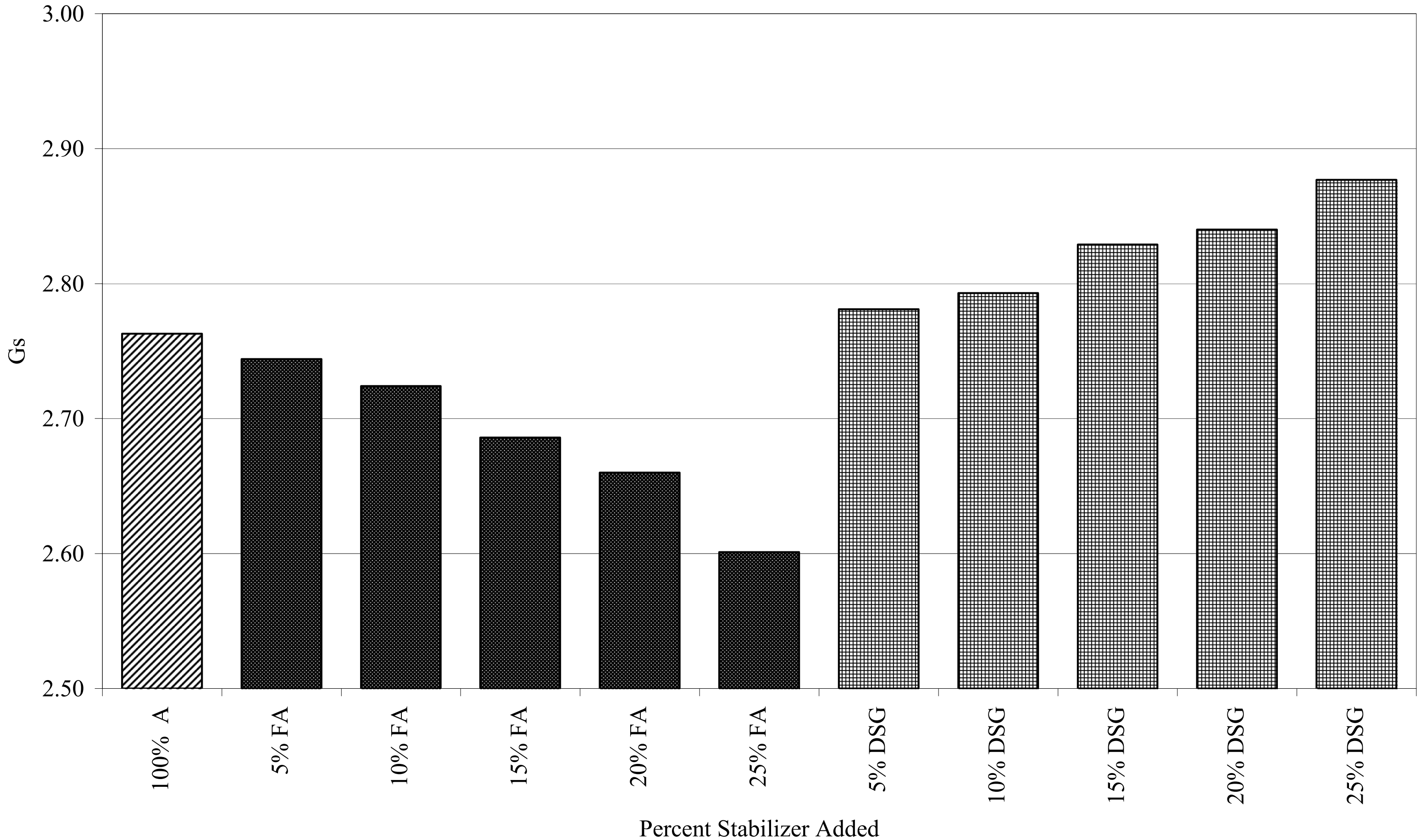

Figure 4.12. Effects of Addition of Fly Ash and Desulphogypsum on the Specific Gravity

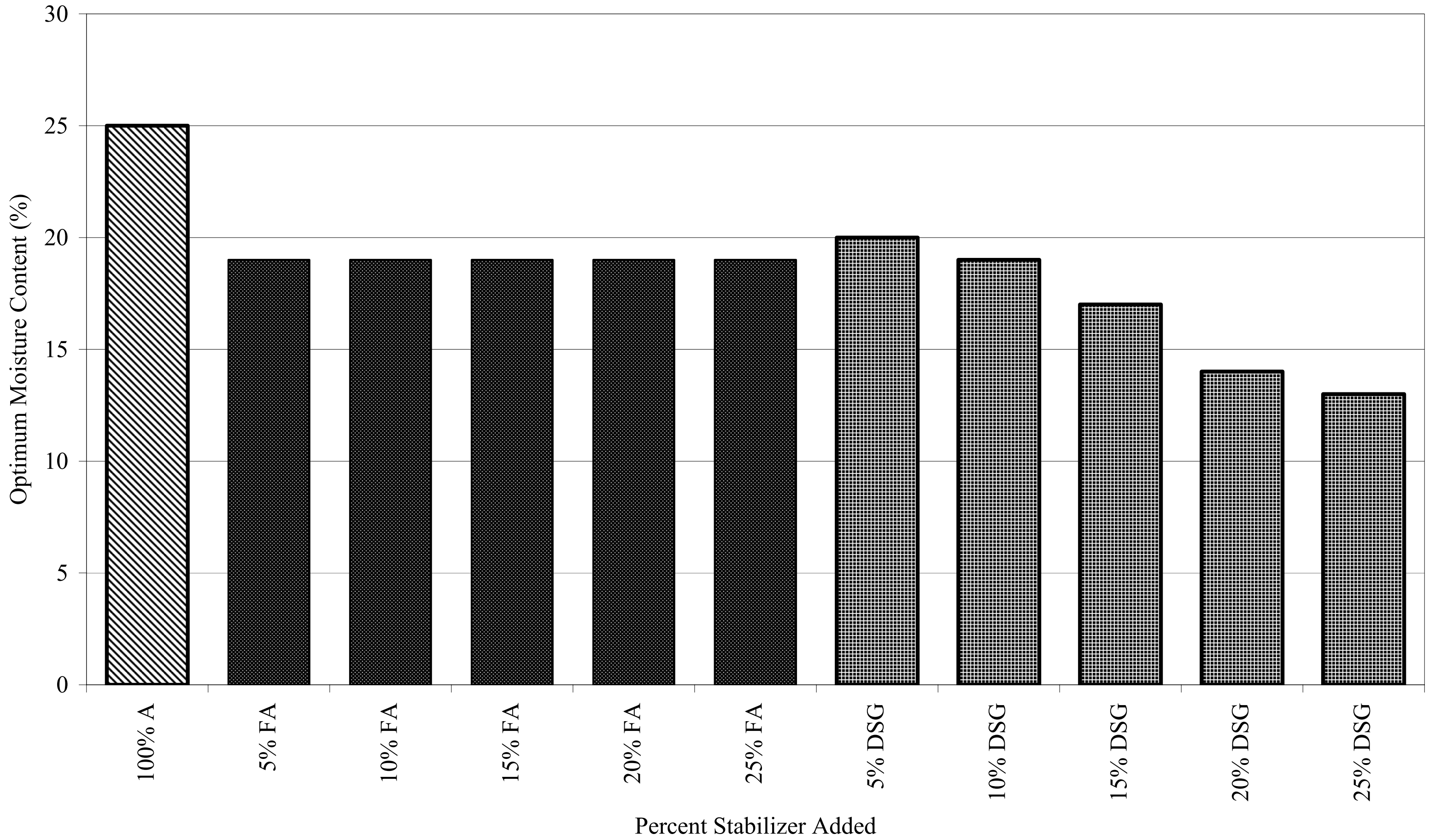

Figure 4.13. Effects of Addition of Fly Ash and Desulphogypsum on the Optimum Moisture Content

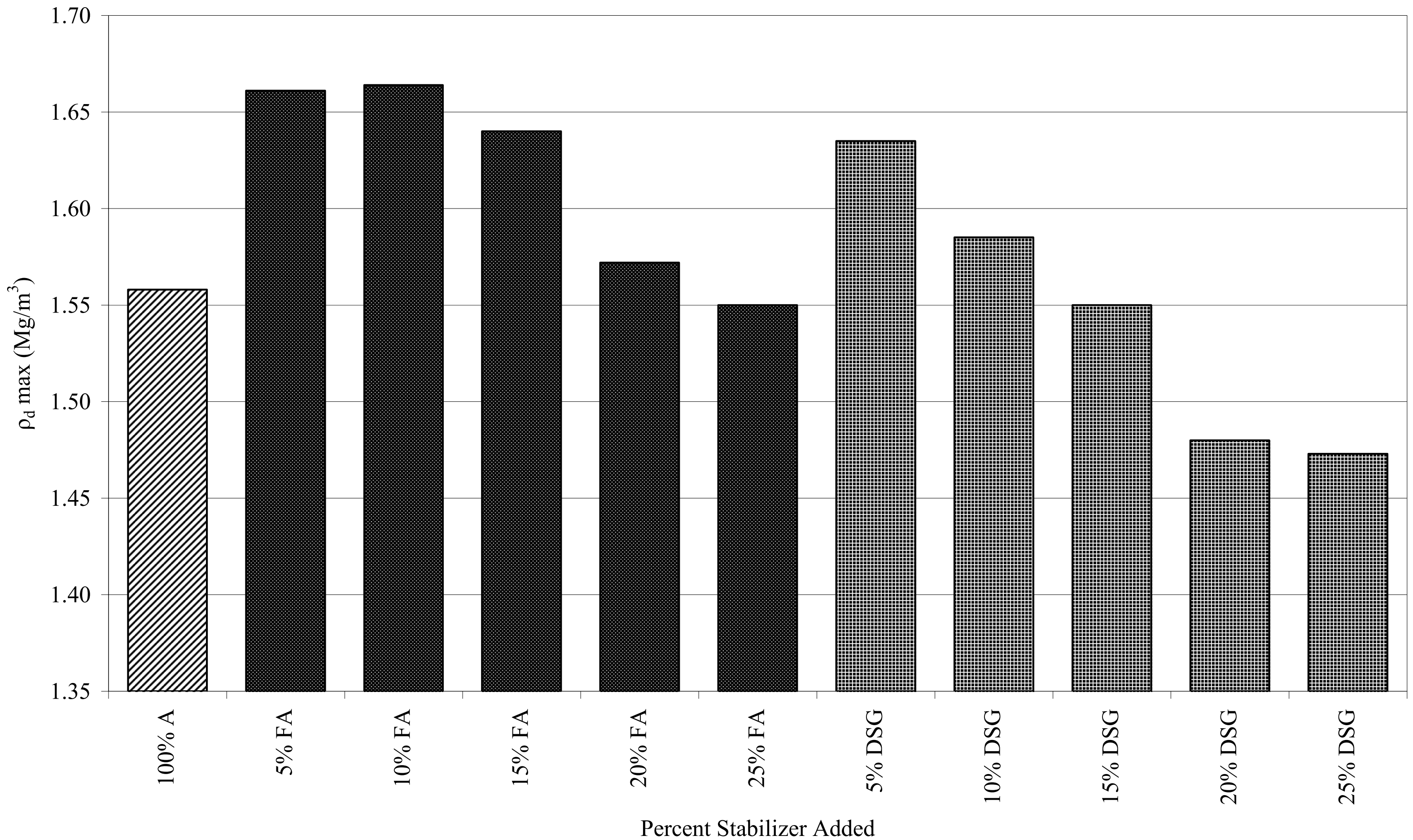

Figure 4.14. Effects of Addition of Fly Ash and Desulphogypsum on the Maximum Dry Density

The collapse potential of the samples is tabulated in Table 4.5. Also the results are shown in Figure 4.15. The consolidation curves of the samples are given in the Appendix A.

Coefficient of volume compressibility (m_v) values of the samples are given in Table A.1.

Table 4.5. Collapse Potential of Samples

	Collapse Potential (%)		
	0 Days Cure	7 Days Cure	28 Days Cure
100 % A	16.47	-	-
100 % A at o.m.c.	0.02	0.02	0.01
5% FA	0.11	0.07	0.07
10% FA	0.22	0.17	0.17
15% FA	0.12	0.23	0.36
20% FA	0.10	0.24	0.54
25% FA	0.08	0.27	0.64
5% DSG	0.05	0.20	0.06
10% DSG	0.17	0.37	0.18
15% DSG	-0.13*	0.06	-0.15*
20% DSG	0.76	1.32	0.71
25% DSG	3.39	4.49	3.41

* (-) sign means expansion.

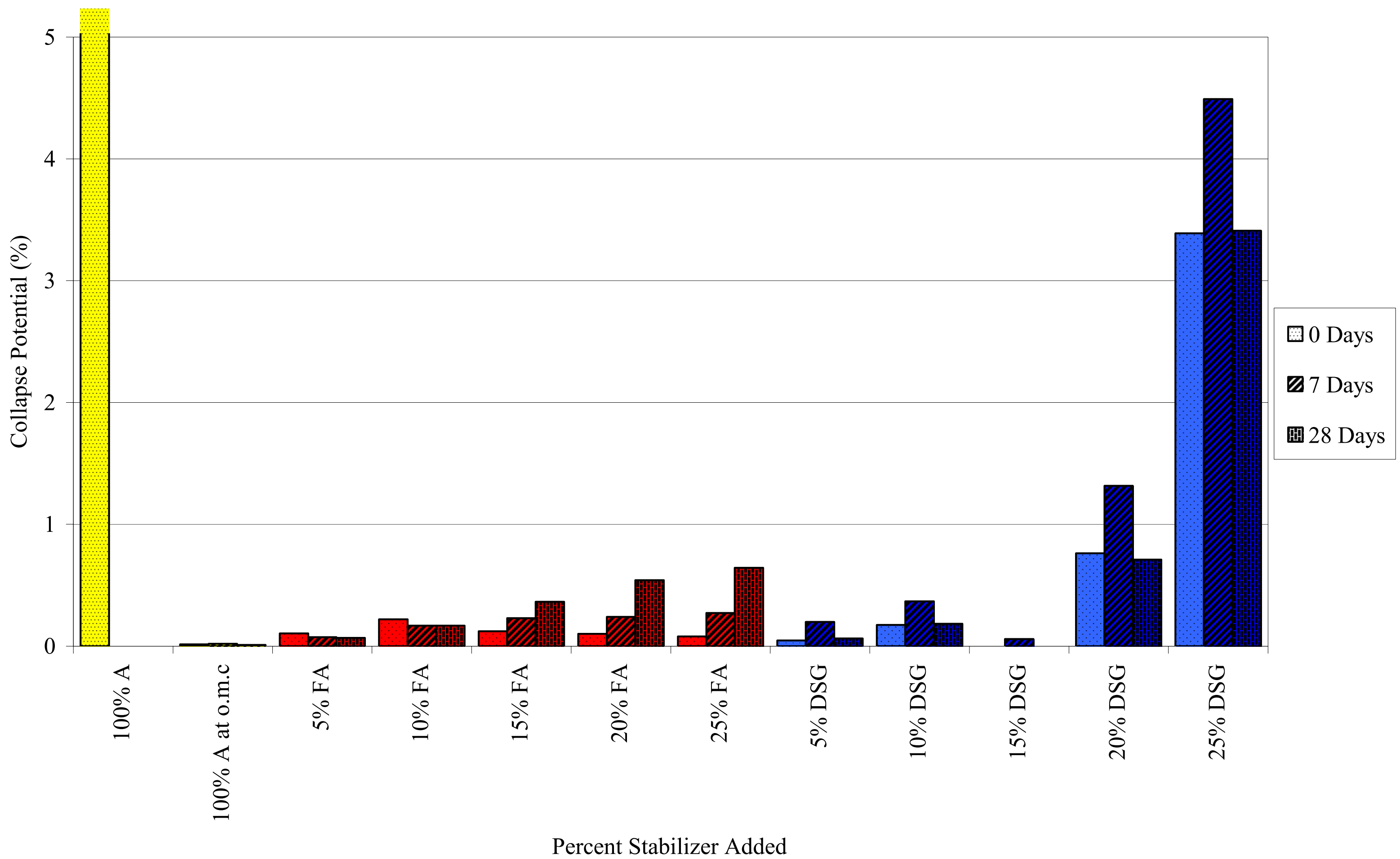

Figure 4.15. Effect of Addition of Fly Ash and Desulphogypsum on the Collapse Potential

The unconfined compressive strengths of the samples is tabulated in Table 4.6. Also the results are shown in Figure 4.16.

Table 4.6. Unconfined Compressive Strength of Samples

	q_u (kPa)		
	0 Days Cure	7 Days Cure	28 Days Cure
100 % A	57.055	-	-
100 % A at o.m.c.	300.873	337.293	371.170
5% FA	319.649	355.191	381.446
10% FA	590.435	615.933	856.312
15% FA	590.287	692.529	889.192
20% FA	352.044	401.599	478.450
25% FA	455.556	606.775	718.884
5% DSG	474.924	519.728	526.558
10% DSG	522.778	571.598	614.125
15% DSG	878.107	932.844	983.805
20% DSG	462.260	470.900	476.660
25% DSG	567.443	699.098	739.557

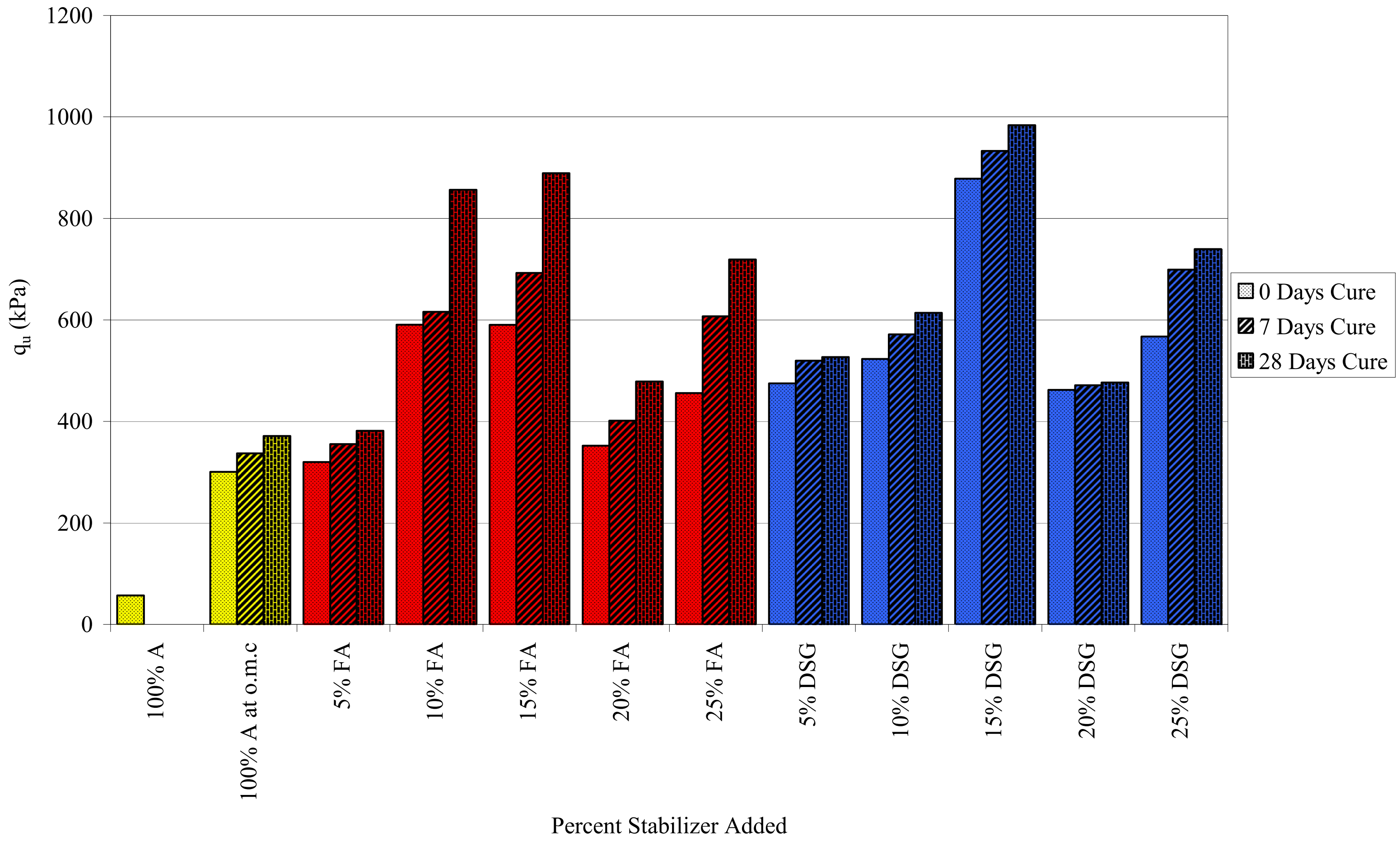

Figure 4.16. Effect of Addition of Fly Ash and Desulphogypsum on the Unconfined Compressive Strength

CHAPTER 5

DISCUSSION OF TEST RESULTS

5.1. Effects of Fly Ash and Desulphogypsum Addition on the Grain Size Distribution of Collapsible Soil

Addition of fly ash shifted the grain size distribution curve of Sample A to the finer side. On the other side, addition of desulphogypsum did not shift the grain size distribution curve excessively (Fig.4.3 and Fig. 4.4). This shifting is a result of adding finer materials to the Sample A.

5.2. Effects of Fly Ash, and Desulphogypsum Addition on the Liquid Limit of Collapsible Soil

Liquid limit values of the samples decreased with increasing fly ash percentages. (Table 4.3 and Fig. 4.7)

Addition of 5% fly ash diminished the liquid limit of Sample A by 0.89%. Reduction continued with increasing fly ash percentages and the maximum fly ash addition (25%) resulted in a 3.69% reduction in the liquid limit of Sample A. (Fig4.7)

For the desulphogypsum added samples, the liquid limit was decreased for the minimum amount of desulphogypsum added, an increasing trend was observed by increasing amount of percent of desulphogypsum (Fig 4.7). Addition of the minimum amount of desulphogypsum (5%) reduced the liquid limit of Sample A by 5.12% and the maximum desulphogypsum additive (25%) caused an increment of 0.52%.

5.3. Effects of Fly Ash, and Desulphogypsum Addition on the Plastic Limit of Collapsible Soil

Addition of minimum amount of fly ash increased the plastic limit of Sample A. However with the increasing amount of fly ash composed a decreasing trend on the plastic limits. (Table 4.3 and Fig. 4.8)

Desulphogypsum addition increased the plastic limit of Sample A by 5.37% at most.

5.4. Effects of Fly Ash, and Desulphogypsum Addition on the Plasticity Index of Collapsible Soil

Plasticity indices of the samples decreased slightly with fly ash addition (Table 4.3 and Fig 4.10).

The maximum amount of fly ash reduced the plasticity index of Sample A by 4.69%.

Addition of 5% desulphogypsum caused a sudden decrease in the plasticity index of Sample A by 6.41%. This is the maximum reduction obtained with the least amount of stabilizer. The maximum amount of desulphogypsum decreased the plasticity index of Sample A by 4.85%.

5.5. Effects of Fly Ash, and Desulphogypsum Addition on the Shrinkage Limit of Collapsible Soil

Addition of all the stabilizers decreased the shrinkage limit of Sample A. (Table 4.3 and Fig 4.9)

The maximum amount of fly ash (25%) decreased the shrinkage limit of Sample A by 9.81%.

Addition of maximum percentage of desulphogypsum (25%) decreased the shrinkage limit of Sample A by 7.65%. The minimum (5%) desulphogypsum addition caused a decrease of 3.80% in shrinkage limit.

5.6. Effects of Fly Ash, and Desulphogypsum Addition on the Shrinkage Index of Collapsible Soil

Shrinkage indices of the samples increased with increasing fly ash percentages (Table 4.3 and Fig 4.11). The maximum fly ash addition increased the shrinkage index of Sample A by 6.12%.

Addition of 5% desulphogypsum decreased the shrinkage index of Sample A by 1.32%. Addition of 25% desulphogypsum increased shrinkage index by 7.97%.

5.7. Effects of Fly Ash, and Desulphogypsum Addition on the Specific Gravity of Collapsible Soil

Specific gravity of the sample is 2.76, specific gravity of the fly ash 2.13. Fly ash addition decreased the specific gravity (G_s) of Sample A as expected. However, since the specific gravity of desulphogypsum is 3.25, desulphogypsum addition increased the specific gravity (G_s) of Sample A. (Table 4.3 and Fig 4.12)

5.8. Effects of Fly Ash, and Desulphogypsum Addition and Compaction on the Optimum Moisture Content of Collapsible Soil

Fly ash addition decreased the optimum moisture content of Sample A. This decreased level stayed constant for all percentages added of fly ash. (Table 4.3 and Fig 4.14)

For desulphogypsum added samples, a decreasing trend was observed with the increasing amount of desulphogypsum. (Table 4.3 and Fig 4.14)

5.9. Effects of Fly Ash, and Desulphogypsum Addition and Compaction on the Maximum Dry Density of Collapsible Soil

Both addition of fly ash and desulphogypsum increased the maximum dry density, however with increasing amount of additives the maximum dry density decreased (Figure 4.14).

For fly ash added samples, the maximum dry density was obtained for 10 % fly ash added sample. The 25 % fly ash added sample gave the minimum ρ_{dmax} value, which was less than 100 % sample A.

For desulphogypsum added samples, the maximum dry density was obtained for 5 % desulphogypsum added sample. The 25 % desulphogypsum added sample gave the minimum ρ_{dmax} value, which was also less than 100 % sample A.

5.10. Effects of Fly Ash, and Desulphogypsum Addition and Compaction on the Collapse Potential of Collapsible Soil

Fly ash addition decreased the collapse potential when compared with the undisturbed sample (Table 4.5 and Fig 4.15)

The best result is taken with the addition of maximum percent of fly ash, i.e. 25%. For the 7 days cured samples, the best result is taken with the addition of minimum percent of fly ash, i.e. 5%. For the 28 days cured samples, the best result is also taken with the addition of 5% fly ash.

Desulphogypsum addition decreased the collapse potential. The addition of 15% desulphogypsum resulted in a swelling about 0.13%. For the 28 days cured samples, a swelling was also observed about 0.15% with adding 15% desulphogypsum. For the 7 days cured samples, the best result is taken with the addition of 15% desulphogypsum.

For the compacted samples, the collapse potential decreased suddenly.

Collapse potential of the samples increase with stabilization compared to 100% Sample A compacted to o.m.c.

5.11. Effects of Fly Ash, and Desulphogypsum Addition and Compaction on the Coefficient of Volume Change of Collapsible Soil

For the compacted samples, as consolidation pressure increases m_v decreases. For the stabilized samples curing has no significant effect, m_v values are at the same order of magnitude.

5.12. Effects of Fly Ash, and Desulphogypsum Addition and Compaction on the Unconfined Compressive Strength of Collapsible Soil

The addition of both fly ash and desulphogypsum increased the unconfined compressive strength of the collapsible soil (Table 4.6 and Fig 4.16).

5% addition of fly ash does not make a significant change compared with compacted sample. The best result among the fly ash added samples is 15% fly ash added sample. The curing also increased the unconfined compressive strength.

For desulphogypsum added samples, the best result is taken for 15% desulphogypsum added sample. A sudden drop is observed on the 20% desulphogypsum added sample.

CHAPTER 6

CONCLUSIONS

The effect of fly ash and desulphogypsum addition on the collapse potential of collapsible soil sample was presented. Fly ash and desulphogypsum were introduced as admixtures up to a maximum of 25% by dry weight of soil. Due to the results of the experiments, the following conclusions are warranted:

1. Addition of fly ash and lime alters the grain size distribution of the collapsible soil sample.

2. Addition of fly ash decreases the plastic index. Addition of desulphogypsum also decreases the plasticity index, and then the plasticity index increases with increasing percent of desulphogypsum added.

3. The shrinkage limit decreases with increasing percents of both fly ash and desulphogypsum.

4. 15% desulphogypsum addition shifts the collapsible soil sample from CL towards ML according to the Unified Soil Classification System.

5. The optimum moisture content drops for both fly ash and desulphogypsum added samples. The optimum moisture content stays stable for the fly ash added samples. On the other hand, for the desulphogypsum added samples

the optimum moisture content drops with the increasing amount of desulphogypsum.

6. The maximum dry density increases with the addition of fly ash, however it then drops and ends with a level less than the dry density of the 100% Sample A. It shows the similar respond against adding desulphogypsum. For the initial adding of the desulphogypsum increases the maximum dry density, however it drops with the increasing amount of desulphogypsum added.

7. The collapse potential of the collapsible soil sample decreases with compaction. Fly ash and desulphogypsum addition don't affect the collapse potential excessively, however addition of the stabilizers increase the unconfined compressive strengths.

8. Up to 15% fly ash added samples, curing decreases collapse potential of stabilized samples. However, samples having more than 15% fly ash, curing has a negative effect on collapse potential. While 28 days curing results are similar to the uncured results, 7 days curing gives worse results for desulphogypsum added samples.

9. Compaction increases the strength of the collapsible soil about 6 times. Adding fly ash also increases the strength of collapsible soil. The addition of 10 % and 15 % fly ash gives the best result among the fly ash added samples. The curing shows a visible increase on the unconfined compressive strengths. The most effective amount of added desulphogypsum is 15 %. This amount is also the best result among all the samples.

10. The unconfined compressive strength of the samples decrease after 15% fly ash and desulphogypsum addition, therefore optimum fly ash and desulphogypsum addition appears to be close to 15%.

11. The fly ash and desulphogypsum stocks pose a serious problem in terms of both land use and potential environmental pollution. The utilization of these industrial by-products for the stabilization of the collapsible soils (for thin collapsible soil layers under the ground surface) near to the thermal power plants may be regarded as economically and environmentally beneficial.

Recommendation for Future Research

In this study, the geotechnical performances of Çayırhan fly ash and Çayırhan desulphogypsum in stabilization the collapsible soils in Çayırhan Thermal Power Plant area were presented. However, past research has established that both fly ash and desulphogypsum may consist of fine particles that contain leachable heavy metals such s arsenic, cobalt, copper, lead, nickel, and zinc (Clark et al. 2001; Ferreira et al. 2003; Tao et al. 2001). Therefore, to define more clearly the conditions for a safe application from an environmental point of view, this research must be extended by performing leachate analyses of the stabilized samples used in this study.

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APPENDIX A

Consolidation test results of samples.

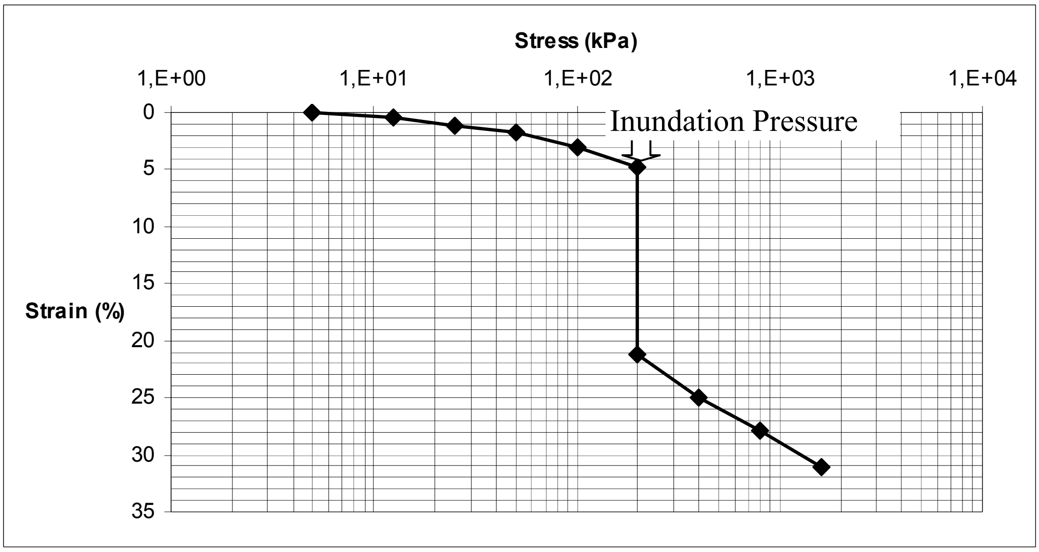

Figure A.1. Consolidation Test Result of 100% A Undisturbed

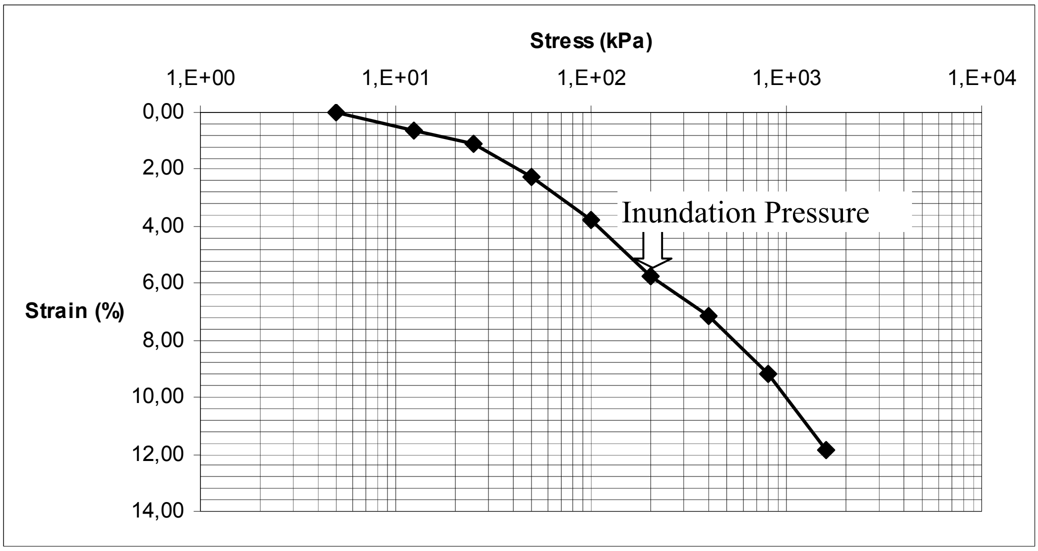

Figure A.2. Consolidation Test Result of 100% A Compacted

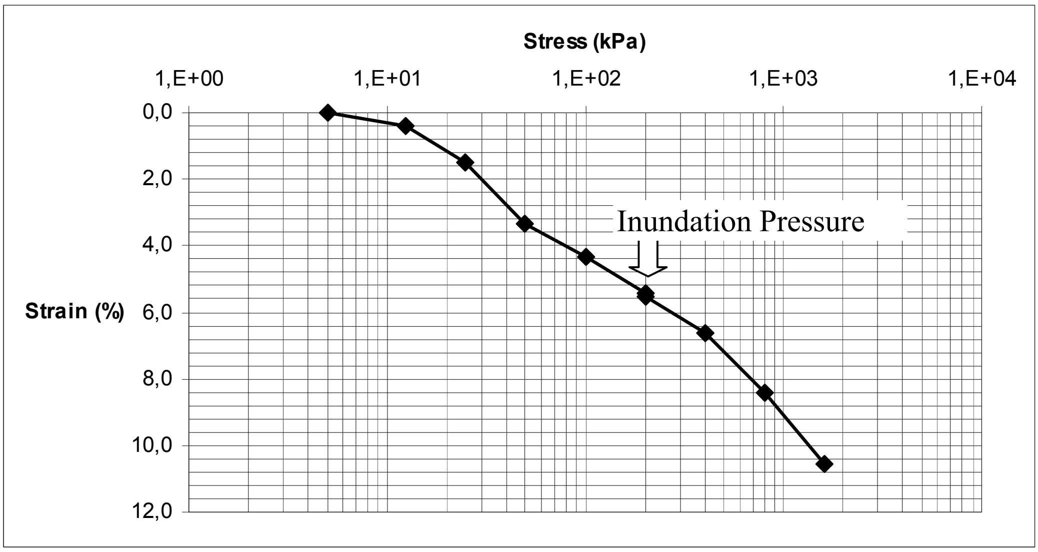

Figure A.3. Consolidation Test Result of 5% FA

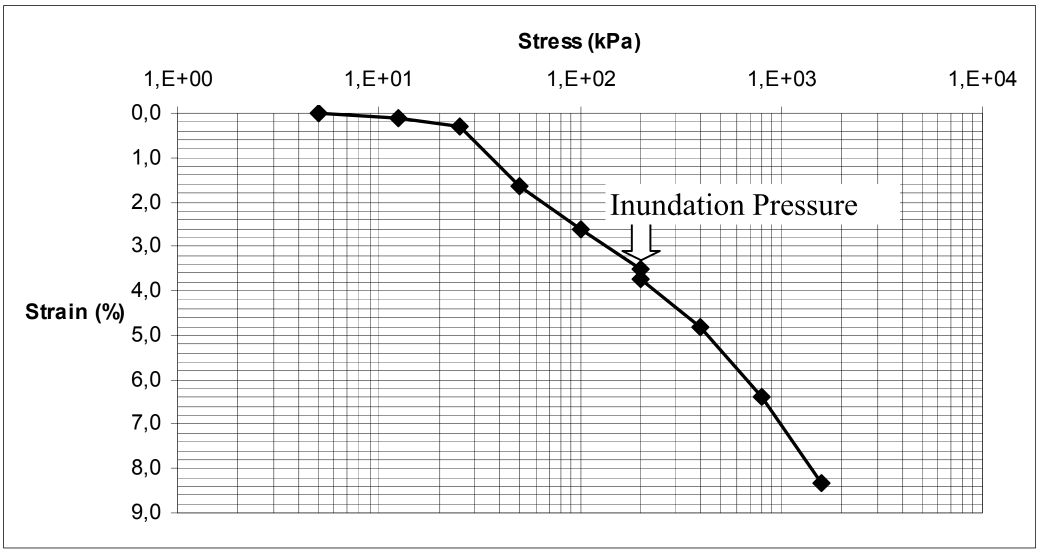

Figure A.4. Consolidation Test Result of 10% FA

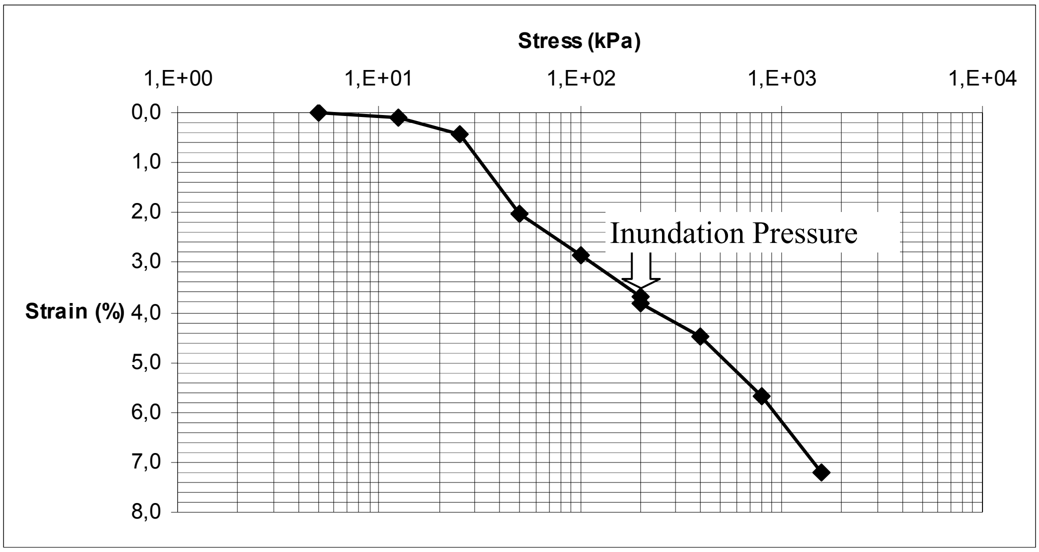

Figure A.5. Consolidation Test Result of 15% FA

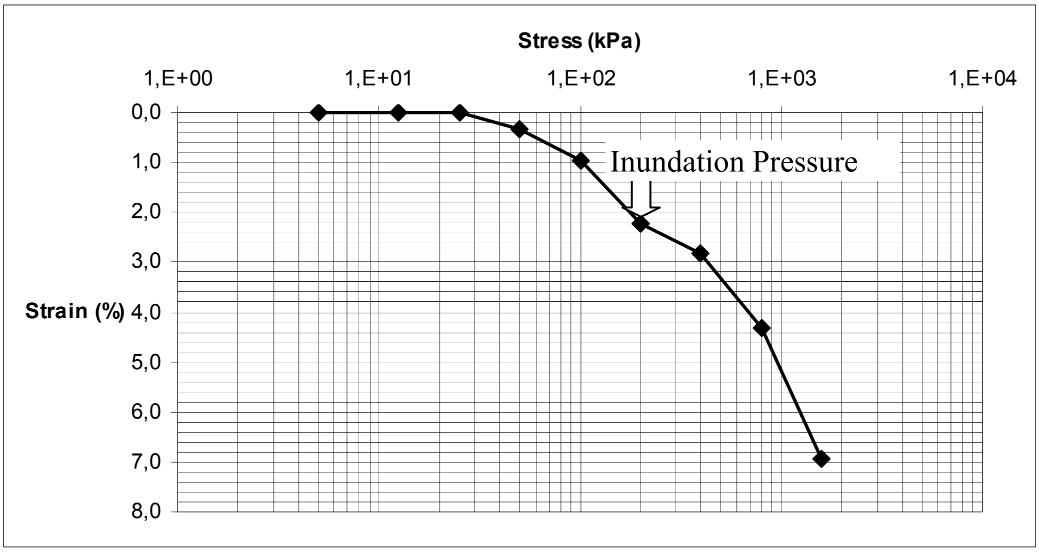

Figure A.6. Consolidation Test Result of 20% FA

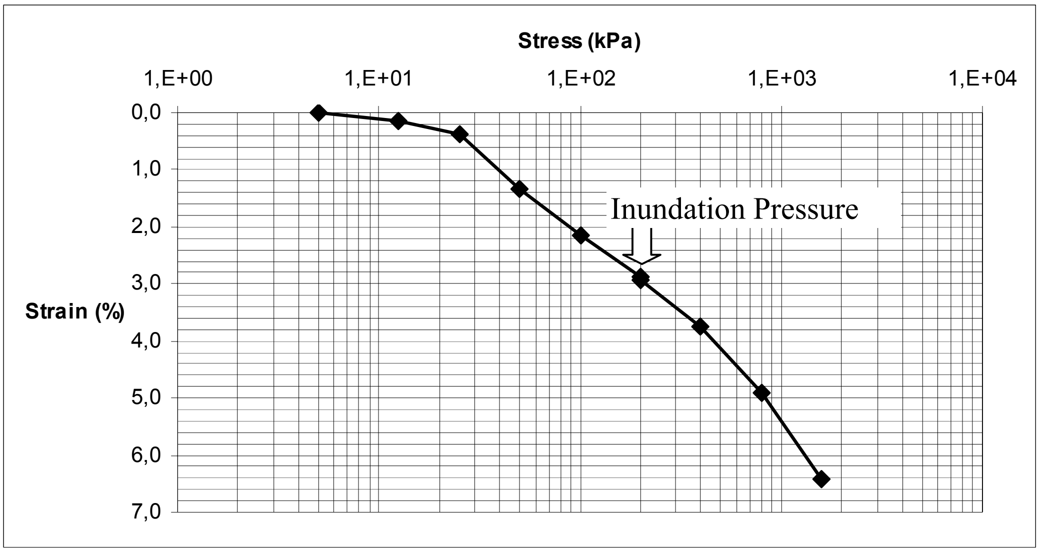

Figure A.7. Consolidation Test Result of 25% FA

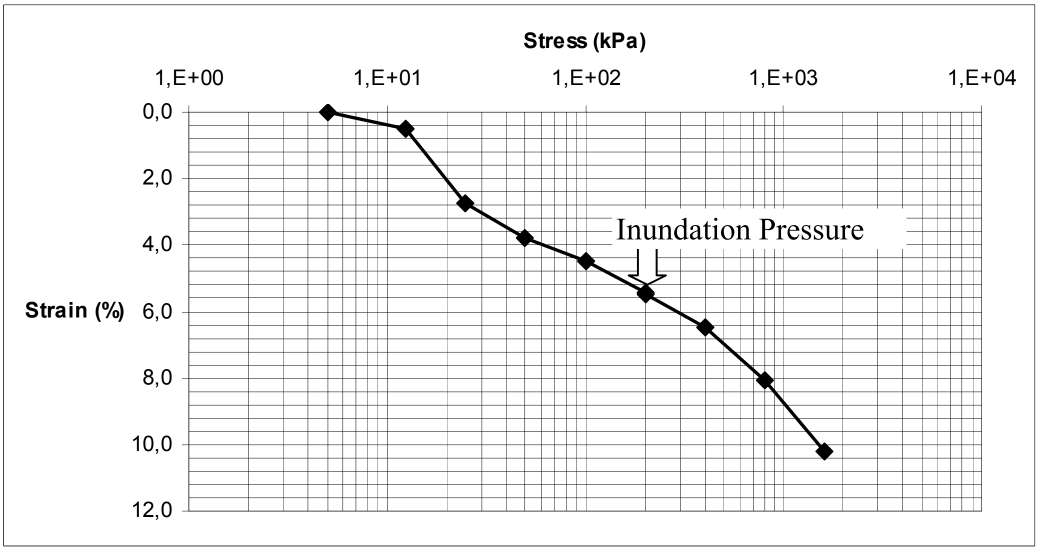

Figure A.8. Consolidation Test Result of 5% DSG

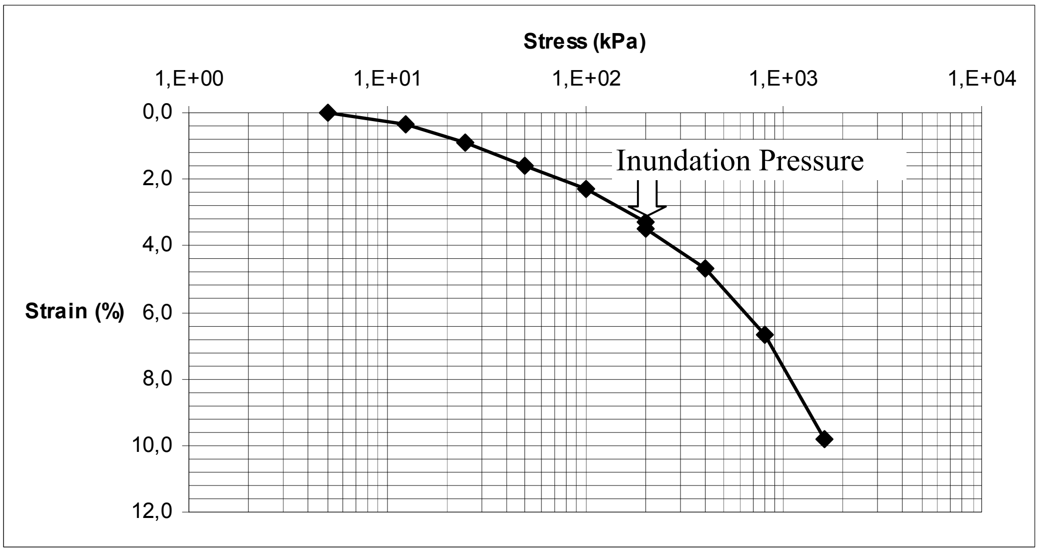

Figure A.9. Consolidation Test Result of 10% DSG

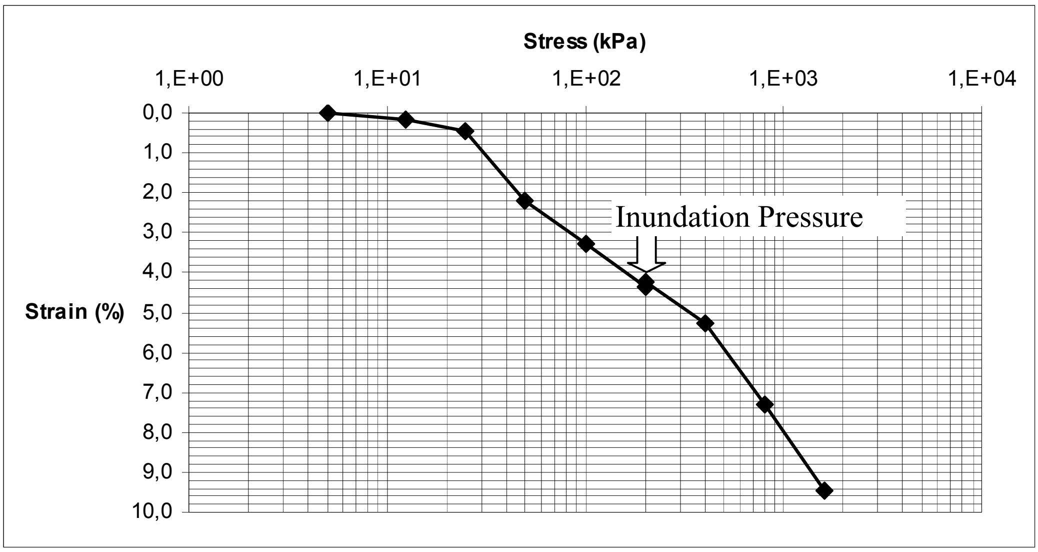

Figure A.10. Consolidation Test Result of 15% DSG

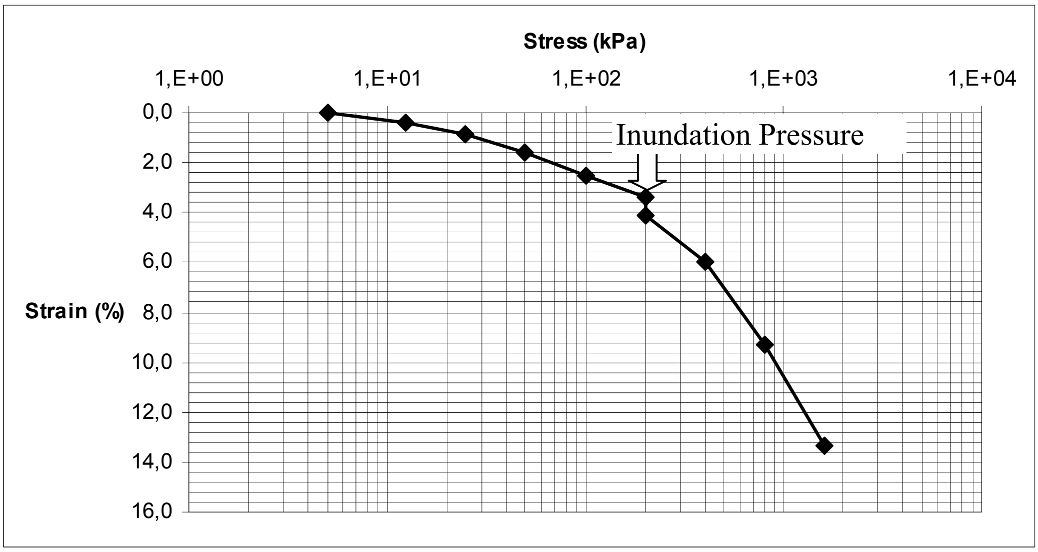

Figure A.11. Consolidation Test Result of 20% DSG

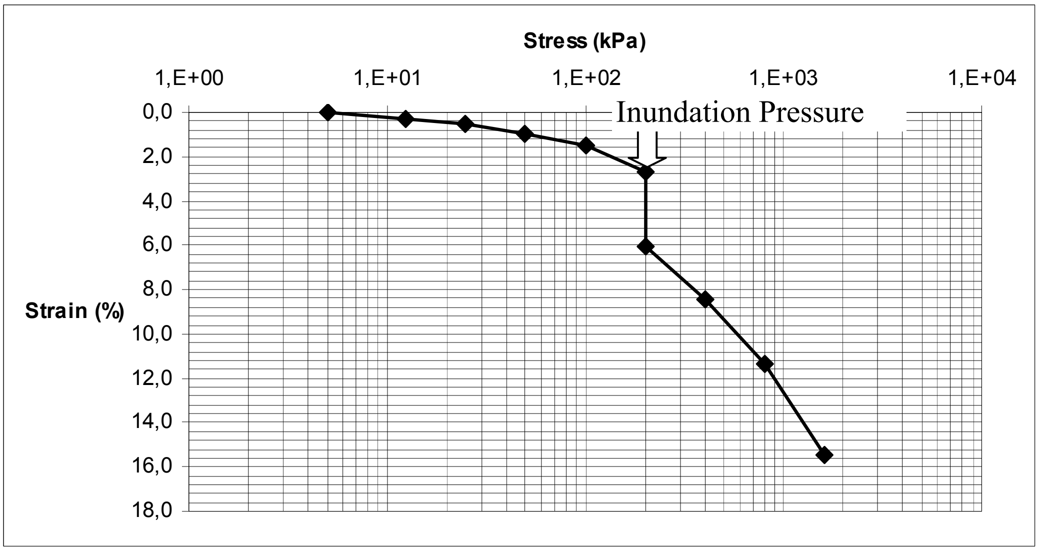

Figure A.12. Consolidation Test Result of 25% DSG

Table A.1. Coefficient of volume compressibility of the samples.

400 kPa		100% A	100% A at o.m.c	5% FA	10% FA	15% FA	20% FA	25% FA	5% DSG	10% DSG	15% DSG	20% DSG	25% DSG
m_v	0 Days	0.0189	0.0070	0.0056	0.0054	0.0051	0.0071	0.0058	0.0050	0.0072	0.0053	0.0091	0.0121
	7 Days		0.0068	0.0052	0.0049	0.0053	0.0076	0.0061	0.0057	0.0078	0.0055	0.0096	0.0121
	28 Days		0.0065	0.0049	0.0047	0.0053	0.0072	0.0063	0.0048	0.0068	0.0056	0.0087	0.0121

800 kPa		100% A	100% A at o.m.c	5% FA	10% FA	15% FA	20% FA	25% FA	5% DSG	10% DSG	15% DSG	20% DSG	25% DSG
m_v	0 Days	0.0075	0.0051	0.0045	0.0039	0.0034	0.0041	0.0029	0.0039	0.0050	0.0051	0.0085	0.0074
	7 Days		0.0050	0.0041	0.0036	0.0038	0.0047	0.0036	0.0042	0.0049	0.0049	0.0079	0.0071
	28 Days		0.0048	0.0036	0.0042	0.0040	0.0043	0.0039	0.0034	0.0047	0.0048	0.0070	0.0071

1600 kPa		100% A	100% A at o.m.c	5% FA	10% FA	15% FA	20% FA	25% FA	5% DSG	10% DSG	15% DSG	20% DSG	25% DSG
m_v	0 Days	0.0043	0.0035	0.0028	0.0025	0.0020	0.0029	0.0020	0.0028	0.0041	0.0048	0.0054	0.0054
	7 Days		0.0033	0.0028	0.0027	0.0024	0.0028	0.0020	0.0029	0.0036	0.0043	0.0054	0.0047
	28 Days		0.0030	0.0026	0.0026	0.0026	0.0030	0.0028	0.0024	0.0036	0.0039	0.0049	0.0049