UTILIZATION OF BITUMINOUS LIMESTONE ASH FROM EL-LAJJUN AREA FOR ENGINEERING APPLICATIONS

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Dedication

To my
Father and Mother,

Wife and Children,

Brothers and Sisters

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ABSTRACT

For the purpose of this research, limy ash was prepared from the El-Lajjun bituminous limestone by direct combustion at 1200, 950 and 525 °C.

The following work aims at minimizing the environmental impact of the solid waste (ash) that is produced after the utilization of the bituminous limestone in thermal power stations and /or retortion processes.

The laboratory tests were selected with respect to construction needs and possible post construction conditions. Utilization of the various types of ash in the stabilization of problematic soils from Jordan as brown soils and the clayey marl has revealed optimum results.

The unconfined compressive strength of the parent brown soil and the clayey marl is raised from 5 kg/cm^2 to 50 and 25 kg/cm^2 respectively. California Bearing Ratio (CBR) value is raised from 4.5 % to 150% for both problematic soil types. Various

mortars and construction elements can be produced at normal room curing temperature without the use of Ordinary Portland cement (OPC).

Low quality sub-base and base course can be mixed with ash to produce cement treated base (CTB) and roller compacted concrete (RCC) without Ordinary Portland cement (OPC). Durable pavements, embankments can be constructed with very long life and low cost. CTB and RCC utilizing ash can be used in dam construction instead of normal soil in earth fill dams.

The high alkalinity of the El-Lajjun ash is considered as a disadvantage to be utilized in normal concrete mixes for structural purposes. Ash only can be mixed with aggregates to produce lean concrete like for blinding purposes to be prepared for foundation activities.

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INTRODUCTION

1. Ash - General

Ash, as a general term, is related to naturally produced material associated with volcanic activities and hence called volcanic ash, or is an industrial by-product of burning coal or sub-bituminous coal at thermal power plants where coal ash or fly ash is produced. In this investigation, ash is referred to the combusted products (waste) of the bituminous limestone deposits from El-Lajjun area, south of the capital Amman in Jordan.

Different types of ash are classified according to their chemical composition into class C, and class F. The main chemical constituents of the various ash types are SiO_2, CaO, Al_2O_3, Fe_2O_3. Minor constituents and trace elements are MgO, MnO, TiO_2, K_2O, P_2O_5, Ni, Cr, Cu, Mo, Zn, V, and Sr.

The chemical composition of ash is characterized by two parts: the major alkaline part which is presented by CaO, and the minor pozzolanic part which is presented by SiO_2, Al_2O_3 and Fe_2O_3. The two parts react with each other by the hydration reactions in the presence of water. Pozzolanic material is only a siliceous or both siliceous and aluminous material, which in itself possesses little or no cementing property, but will chemically react with calcium hydroxide in the presence of water under normal ambient temperature to form a low cost self cementaceous paste.

Ash can be used as a partial substitute of ordinary cement which is used to produce low strength construction materials such as masonry bricks, lightweight concrete, roller compacted concrete in dam’s construction and in many geotechnical engineering aspects as soil improvement and stabilization of problematic soils in roads and highways. Huge quantities of ash, which is a waste material, are expected to be produced as a by-product of retortion processes of bituminous limestone and marl or as a result of the direct combustion of the bituminous rocks in thermal power plants. Minimizing the volume of the solid waste is considered as one of the most important targets to help reduce one of the expected negative environmental impacts.

The self-cementing characteristics of ash have been used in earthwork applications to improve the mechanical properties of soils for more than 20 years (Ferguson, 1993). The potential for using ash in soil stabilization has increased significantly in the world due to its increased availability and the introduction of new environmental regulations. This type of ash provides the opportunity for applications where no other activators would be required and thus it offers more economical alternatives for a wide range of stabilization applications.

Improved engineering properties of fly ash-stabilized soil are also reported by Turner (1997). The experimental works showed that the pozzolanic reactions within the soil-ash mixture or aggregate-ash-cement mixtures in concrete mixes and mortars are highly influenced by curing time, moisture content, and the nature of ash represented by its chemical composition and physical properties.

1.1 Ash Uses

Soil stabilization can be defined as a means of permanently altering soil to increase its strength and bearing capacity, and decrease its water sensitivity and volume change potential. Soil stabilization can eliminate the need for expensive borrow materials, expedite construction by improving wet or unstable soil, or allow reduced pavement thicknesses by improving subgrade conditions. Soil stabilization is based on the treatment of clay soils with a certain material to provide strength and stability.

When the stabilizer is mixed with moist plastic soils, a hydration reaction occurs in which calcium ions are released. As a result, cation exchange and flocculation-agglomeration occurs. This will reduce the moisture content and improves the plasticity characteristics of the amended soil (Brendel et al., 1997). The long-term strength of stabilized soils may increase due to pozzolanic reactions.

In ash stabilized soil, the pozzolanic reaction depends upon the cooperative reaction between the ash alkalis, mainly CaO and soil. Many soils contain silica, alumina, and iron, which will react with lime and develop long-term strength. The expected significant strengthening of the clays is probably due to their relatively high CaO content (Tharpe and Abdulally, 1997).

In addition to the above mentioned uses, ash can be used in the following applications which are related to soil problems:

1.1.1 Drying Agent

One of the early applications of self-cementing ash is its use as a drying agent to facilitate soil compaction. Due to its hydration characteristics, the additions of ash can provide an immediate drying effect and reduce the moisture contents to levels suitable for compaction.

Application of fly ash can reduce moisture content in the soil and make it stable over a wide range, thus the soil will reach the final compaction with less effort and cost.

1.1.2 Reduction of swell potential

Ash soil treatment can be used effectively to reduce the shrink-swell potential of clay in soils instead of the traditional lime and ordinary Portland cement treatment method. Ash can contribute to the reduction of swell potential. Literature review and laboratory tests indicate that fly ash treated soil could significantly reduce the swell potential as compared to the untreated materials (ACAA, 1995).

1.1.3. Stabilization to increase shear strength and subgrade support capacity

Ash can be used in the construction of a stabilized section where an increase in shear strength is needed. Such applications include stabilized base or subgrade for pavements, stabilization of backfill to reduce lateral earth pressure and stabilization of embankments to improve slope stability.

1.1.4. Stabilization of soil

Ash can provide enough array of cations which can enhance the flocculation of fine clay particles. The unstable clay soils therefore can be stabilized effectively by cation exchange inside the soil structure.

On the international scale, different classes of ash (Class C and Class F) are utilized in various engineering aspects and are controlled by the ash class according to its chemical composition. The use of self-cementing fly ash as a soil stabilizing agent was investigated by Ferguson (1993), and Little et al. (2003). Tables 1 and 2 indicate the major differences between the two classes of ash.

Stabilization of soils and pavement based on ash utilization is an increasingly popular option in earthworks concerning highways, roads, airports, structural fills and flowable fills that flow like liquid and set up like a solid. Other fly ash applications that have been reported include use in grouts, fast-track concrete pavements, and as structural fills and backfills as reported by the American Coal Ash Association (ACAA, 1995). Ash stabilization is used to modify the engineering properties of locally available materials and produce a structurally sound construction base.

Table (1) ASTM Standard C 618-99, ash classification

Chemical requirements	ASTM Standard C 618-99	
	Class F ash	Class C ash
$SiO_2+Al_2O_3+Fe_2O_3$ %	70	50
SO_3 , max %	5	5
Moisture content max 5	3	3
Los on Ignition max%	6	6

Table (2) Ferguson et al., 1999, ash classification

Chemical composition	Ferguson classification	
	Class F ash	Class C ash
SiO_2 %	54.9	39.9
Al_2O_3 %	25.8	16.7
Fe_2O_3 %	6.9	5.8
CaO %	8.7	24.3
MgO %	1.8	4.6
SO_3	0.6	3.3

The bituminous limestone from El-Lajjun area is rich in $CaCO_3$, thus the ash produced during combustion is rich in CaO, resulting in the self–cementing characteristics. The activators such as lime or Portland cement are not required. Upon exposure to water, ash hydrates to form self cementaceous materials similar to those produced during the hydration of Portland cement. This is an advantage that makes the self cementing ash a very effective and economical stabilization agent for use in soil stabilization and variety of construction applications.

1.1.5 Purposes of soil stabilization

Pavement design is based on the premise that specified levels of quality will be achieved for the various layers in the pavement system. Each layer must have the target shear strength to resist elastic deflections that may result in fatigue cracking within the layer or the overlying layers and hence excessive permanent deformation through densification. As the quality of the soil layer is improved, then the ability of the stabilized layer to distribute the loads over a larger area is increased and hence the reduction in the required thickness of base and sub-base layers. High calcium ash is considered efficient in the following:

1- Improve quality

The quality of the properly stabilized soil is significantly improved through reduction of plasticity, and swell potential, increase of strength and durability.

2- Reduction of the thicknesses of the pavement layers

The tensile strength and stiffness of a soil layer can be increased through the use of a suitable cementaceous material as ash or any suitable additive.

3- Selection of the stabilization agent

The selection of the stabilizer depends on many factors as the type of soil to be stabilized, the purpose for which the stabilized layer will be used, the required strength, cost and the environmental aspects.

The success of additive stabilization depends, to a large extent, upon attaining complete and uniform distribution of the additive in the soil. This step is most critical, the use of ash with high lime content is considered an effective method by which pulverizing of fine clayey soils became easier to maintain proper distribution of moisture content and homogeneity of the treated soil before compaction. This will lower the compaction effort, saving time and will lower construction costs. For materials stabilized with ash whose strength increases with time of curing, it is essential that the stabilized layer be constructed sufficiently early in the seasons of moderate temperatures to allow the development of adequate strength since the rate of strength gain depends on curing time and temperature.

1.1.6. Fly ash in concrete production

The use of fly ash in concrete has many benefits as indicated by the American Coal Ash Association (ACAA). It enhances workability, reduces bleeding, increases ultimate strength, reduces permeability, and lowers heat of hydration and drying shrinkage (ACAA, 1995).

The main precautions usually associated with the use of fly ash in concrete include somewhat slower early strength development, extended initial setting time, possible difficulty in controlling air content, seasonal limitations during winter months, and quality control of fly ash sources (ACAA, 1995).

Some of the engineering properties of fly ash that are of particular interest (when fly ash is used as an admixture or as cement added to concrete mixes) are fineness, loss on ignition (LOI), chemical composition, moisture content, and pozzolanic activity. Most specifying agencies refer to (ASTM C618) when citing acceptance criteria for the use of fly ash in concrete.

The ideal ash for concrete production must satisfy the following characteristics:

1- Low carbon content

2- High fineness and reactivity

3- Consistency and uniformity

4- Low alkalis

Previous studies of fly ash concrete mixes have generally confirmed that most mixes that contain Class F fly ash can replace Portland cement at a 1:1 (equal weight) ratio. The gain of compressive strength, as well as tensile strength, is slower than conventional concrete mixes for as long as 60 to 90 days.

Beyond 60 to 90 days, Class F fly ash concrete mixes will ultimately exceed the strength of conventional Portland cement concrete mixes, (ACAA, 1995). For mixes with replacement ratios from 1:1 to 1.5: 1 by weight of Class F fly ash to the Portland cement that is being replaced, 28-day strength development is approximately equal to that of conventional concrete.

The initial impetus for using fly ash in concrete is stemmed from the fact that the slower reacting fly ash generates less heat per unit of time than the hydration of the faster reacting Portland cement. Thus, the temperature rise in large masses of concrete (such as dams) can be significantly reduced if fly ash is substituted for cement, since more of the heat can be dissipated as it develops. Not only the risk of thermal cracking can be reduced, but greater ultimate strength is attained in concrete with fly ash because of the pozzolanic reaction, (Halstead, 1986).

Class F fly ashes are generally more effective than Class C fly ashes in reducing the heat of hydration. The percentage of Class F fly ash used as a percent of total cementaceous material in typical highway pavement or structural concrete mixes usually ranges from 15 to 25 percent by weight, (ACAA, 1995). This percentage usually ranges from 20 to 35 percent according to the American Concrete Institute (ACI, 1996).

Ash concrete mixes must be properly cured due to slow strength development, and hence moisture must be retained in the concrete for a longer period of time than what is normally required for conventional concrete. Adequate time must be allowed to reach the desired concrete strength before applying traffic loads to pavement construction.

1.2. Statement of the problem

Large feasible deposits of bituminous limestone or what is called bituminous oil shale is an indigenous source of energy in Jordan. About 52 billion tons of these deposits are widely distributed in the northern and central parts of the Hashemite Kingdom as confirmed by the successive geological surveys carried out by the Natural Resources Authority in Jordan. These deposits are of Cretaceous age, but vary in their organic content, thickness, average oil content, and overburden thicknesses (Abu Ajamieh, 1980).

The expected investment or utilization of these rocks in the future as a source of crude oil extraction by retortion processes, or for use as an energy source for electricity generation by direct combustion, will lead to the production of 52% by weight of the original rock as solid waste (ash). The huge quantities of ash are considered as one of the main obstacles when considering the environment, cost of dumping and site preparation.

Ash could be the solid waste product of possible utilization of the EL-Lajjun bituminous limestone. The following investigation shall concentrate on determining the physical, chemical, and mechanical properties of ash. The possibilities to utilize the solid waste ash material in various geotechnical engineering aspects will be investigated such as the improvement and stabilization of weak materials like soil and marl, flowable and normal backfill in roads and highways. The production of various construction materials as lightweight blocks for masonry works, cement substitute in normal concrete mixes, cement treated base (CTB), roller compacted concrete in dams construction, low cost self cementing mortars for various engineering purposes are among the possible utilizations.

El-Lajjun bituminous limestone has been chosen as a source for the ash to be used in the following research work. The deposit is the shallowest known in Jordan with a thin overburden, easily accessible and characterized by favorable mining conditions and costs. The average thickness of the deposits ranges between 25-30 meters with an estimated reserve of 120-130 million tons of oil in place (Hamarneh, 1995).

The necessary physical and mechanical testing will be carried out as maximum dry density-moisture content relationship, California bearing ratio, unconsolidated undrained triaxial testing, free swelling and unconfined compressive strength. The testing will be carried out at 7 and 28 days for ash-soil mixtures at different percents of ash content.

Compressive strength for concrete-mixes and ash-mortars will be determined for various cement-ash contents at 7 and 28 days. Some samples will be tested after 56 days.

The results are expected to reveal the importance of utilizing the bituminous limestone ash from El-Lajjun area as a self cementing material. The material could be utilized for soil stabilization in roads and highway construction. Different light weight, low cost construction materials and mortars with adequate strength can be produced at ordinary ambient temperature curing conditions, with zero ordinary Portland cement content.

1.3 Objectives

The largest feasible reserves of bituminous limestone and marls in Jordan occur in El-Lajjun area. The retortion of the bituminous rocks, or the generation of electricity based on direct combustion is expected to produce huge amounts of solid waste (ash), which is considered as one of the largest obstacles in the utilization of bituminous rocks

No detailed work was carried out on utilizing the ash from El-Lajjun for construction purposes. Serious problems of handling and removing the solid waste to the disposal sites are expected. The produced ash is expected to be huge and could reach more than (50%) of the exploited bituminous rocks which makes this problem complex.

At present, the huge reserves of the Jordanian bituminous rocks are taken into consideration on the national scale as a potential expected energy source for the future. This is due to the continuously increasing demand of imported crude oil as a result of the high rate of population growth.

Safe disposal of the ash without adversely affecting the environment and the large storage area required are the major concerns considering the solid waste management. Understanding of the physical, chemical and engineering properties of the ash from the proposed location will help find the ways and means of utilizing the ash rather than dumping it.

The proposed study aims at the following:

1. Stabilization and improvement of the physical and mechanical properties of problematic soils concerning embankments in roads and highways construction.
2. Utilizing the ash in the flowable backfill and light weight backfill
3. Production of low strength normal concrete by adding naturally occurring local resources.

4. Production of various construction materials as lightweight masonry blocks.
5. Possible use in roller compacted concrete, which is a recommended technology in dam construction.
6. Production of self cementing mortars for various construction purposes.
7. Production of CaO to be used in cement industry.

1.4 El-Lajjun Deposits

Intensive studies have been carried out by the Natural Resources Authority (NRA) in Jordan since 1968. A detailed work was done by the NRA with consultation provided by the Institute of Geological Science (IGS) in London and the Exploration, and Research Division of British Petroleum (BP) at Sunbury, (Nemri, et al., 1981).

The studies have included core drilling, laboratory analysis for oil yield determination using the Fischer Assay procedure, determination of the inorganic constituents, trace elements and calorimetric value.

In 1986, a proving test was carried out in order to determine the feasibility for processing the El-Lajjun bituminous limestone using the Fushun-type retort. The results were positive (Hamarneh, 1995). The characteristics of some of the bituminous limestone deposits are given in Tables 3 a and b as indicated by Bsieso (2003) in Jordan.

1.4.1 Location of El-Lajjun deposit

The deposits are located in the western part of central Jordan, about 100 kilometers, south of Amman. The highway joining Qatrana and Karak divides the deposit area into two southern and northern parts relative to the highway. The deposits cover an area that extends about 10 kilometers long and 2.5 kilometers wide.

1.4.2 Geology, structure, lithology and petrography

Oil shale deposits which are bituminous limestone and bituminous marl in Jordan belong to the Maestrichtian-Danian age and are believed to be deposited in a shallow marine environment according to paleontological studies (Speers, 1969, Amireh, 1979, Abed and Amireh, 1983). The deposits are covered by thin overburden sediments ranging in thickness from 0–30 meters.

Table 3 Characteristics of some bituminous limestone deposits in Jordan, (Bsieso, 2003)

Table 3a

Location	Surface area (km^2)	Overburden thickness (m)	Oil shale thickness (m)	Organic matter (%)	Average oil content (%)
EL-Lajjun	20	31	29	28	10.5
Sultani	24	69	32	25	9.7
Jurf Ed-Darawish	150	47	68	18	5.7
Attaraat Um-Ghudran	226	47	36	29	11.0
Wadi Maghar	29	40	40	20	6.8

Table 3b

Location	Ash Content (%)	Specific gravity	Density (gr/cm^3)	kcal/kg	Calorimetric Value (kj/kg)
EL-Lajjun	54.7	2.4	1.81	1650	6906
Sultani	55.5	2.4	1.96	1526	6380
Jurf Ed-Darawish	58.4	2.4	2.1	1100	4603
Attaraat Um Ghudran	53.2	2.6	1.8	1730	7.235
Wadi Maghar	57.5	2.6	2.03	1090	4.773

The lower part consists of marlstone with embedded gravels, while the upper part (1-2m) is unconsolidated brownish, calcareous sandy silty clay sediments (Figure 1). The deposits have variable thicknesses and vary from one location to another. The average thicknesses of the bituminous deposits in Jordan range from 30 meters at the El-Lajjun area to 400 meters at the Yarmuk area in the northern parts of Kingdom. The bituminous limestone and marl in Jordan are recognized in the Chalk Marl Unit of Bender (1968) or in the stratigraphic equivalent B3 Muwaqqar Formation of Barper and Parker (1970). The bituminous sequence of the Chalk Marl Unit is divided into smaller subunits (Huffnagel, 1980).

El-Lajjun area is a graben like structure and is restricted between two parallel faults striking north-east. There are three outcrops of bituminous limestone in the vicinity of El-Lajjun area (Abu Ajamieh, 1980):

1- Wadi Arbid outcrop, (south of Karak- Qatrana highway where the middle and upper parts of the bituminous limestone are exposed.

2- Wadi El-Lajjun outcrop in the vicinity of Ain El-Lajjun where the lower part of the bituminous limestone is exposed.

3- Wadi Ramla outcrop where the lower part of the bituminous limestone beds are also exposed.

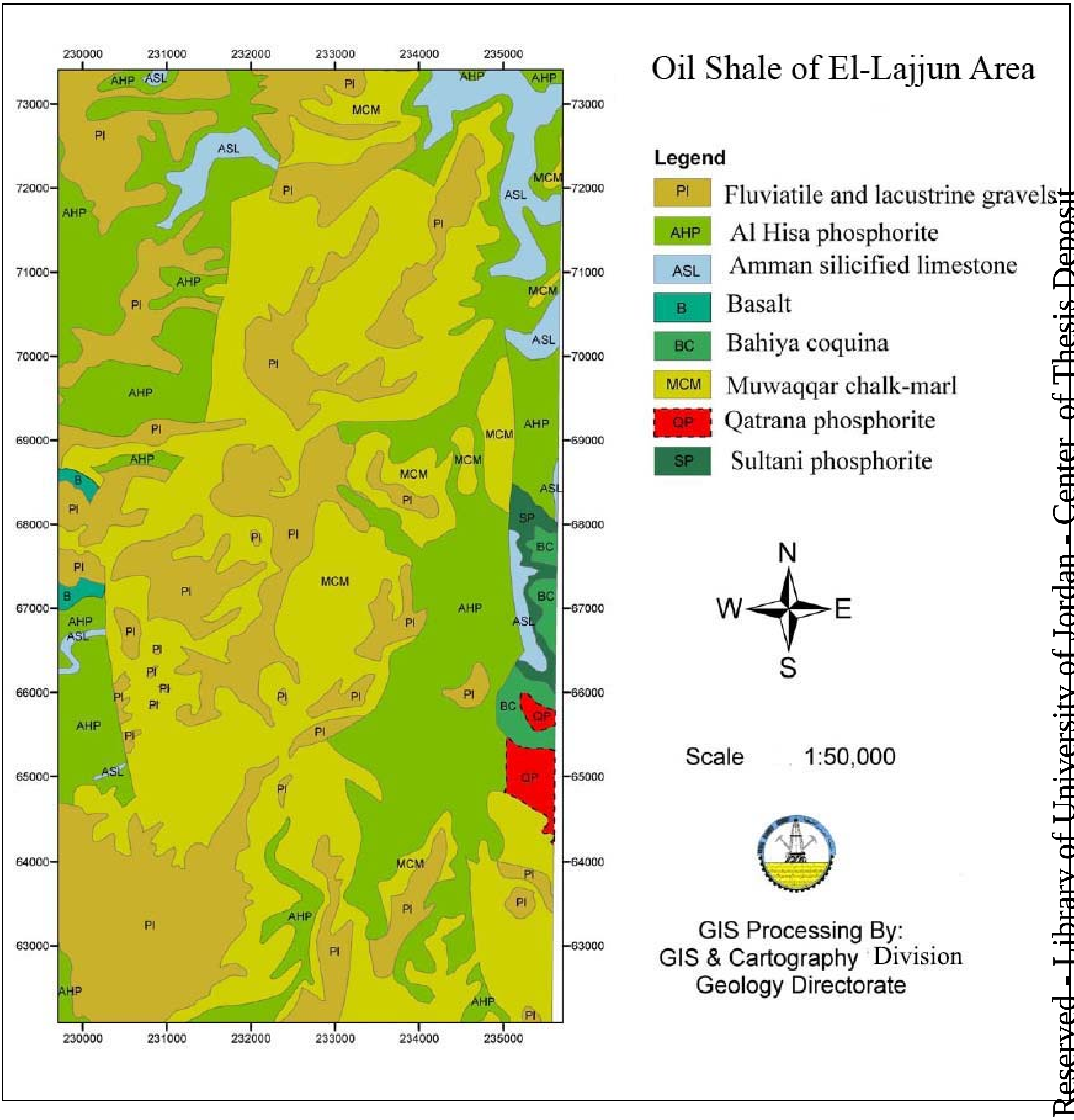

Figure (1) Geologic map of El-Lajjun area (modified after NRA, 2005)

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1.4.3 Mineralogical and chemical composition

Intensive X-ray diffraction and chemical analyses were carried out on the bituminous rocks by the Natural Resources Authority (NRA) for the last forty years. Analytical work was also carried out on core samples obtained from boreholes drilled at variable depths in El-Lajjun deposits (Amireh, 1979, Abu Ajamieh, 1980, Hamarneh, 1995).

The results indicated that the mineral content is mainly dominated by calcite 20%-80%. Quartz 10%-40%, kaolinite 5-10%, apatite 4-14%, and dolomite 2-3.6% are the other constituents. Minor minerals (around 5%), as feldspar, goethite, opal, muscovite. and pyrite. Gypsum is also present as a secondary mineral.

1.4.4 Chemical characteristics of the bituminous rocks

Experimental and retorting facilities which were used to test the bituminous rock samples, the cost of extracting oil, chemical analysis and the environmental impacts are given by Hamarneh (1995. Disturbed and undisturbed samples were collected for the performance of analytical work by Suncor (Al-Kailani, et al., 1998). The chemical composition for some different bituminous limestone deposits from different locations are summarized by Mnezel (2005) and are given in Table (4). The table indicates the chemical characteristics of the Jordanian bituminous rocks which are rich in sulfur and base metals. The possibility of utilizing the expected huge quantities of the solid waste (ash) from EL-Lajjun area was not investigated by the Natural Resources Authority or other sectors.

A limited experiment, concerning the utilization of ash from the northern parts of Jordan as a substitute of ordinary Portland cement in normal concrete mixes was carried out by the National Energy Research Center of Royal Scientific Society (RSS). The report is not available for researchers due to the restrictions and regulations of the

RSS even through an official request by the Department of Geology, University of Jordan.

Table (4) Chemical composition of bituminous limestone at different locations (Mnezel, 2005)

Location / Oxides	El-Lajjun	Sultani	Jurf Ed-Darawish	Attaraat Um-Ghudran	Wadi Maghar
NO. of samples	2200	1400	1200	4000	850
SiO_2 %	16.13	26.3	9.3	21	24.5
Al_2O_3 %	3.8	2.9	3.8	2.7	2.3
Fe_2O_3 %	1.5	1.12	1.55	1.7	1.2
MgO %	0.85	0.95	0.22	1.35	1.5
CaO %	30.43	26.3	38.8	25.7	28.2
P_2O_5 %	3.3	3.5	1.5	2.5	1.2
SO_3 %	4.8	4.4	4.3	5.6	5.6
As ppm	-	17	10	-	-
Cu ppm	92	115	68	100	75
Mo ppm	73	94	20	50	73
Ni ppm	167	139	102	75	75
Sr ppm	1015	707	1187	500	650
U ppm	29	25	17	20	22
Zn ppm	451	649	190	150	250
Ba ppm	113	46	35	60	45
Cr ppm	431	267	226	275	325
V ppm	162	268	101	100	120

METHDOLOGY

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2. METHODOLOGY

To achieve the proposed goals of this research work, the following investigations are performed:

1. Characterization of the parent bituminous limestone bulk sample. A representative bulk sample of around 500 kilograms of ash was prepared by direct combustion of the bulk representative sample from El-Lajjun site.
2. Identification and characterization of two problematic soil samples, brown soil and yellowish clayey marl.
3. Ash stabilization / modification of the problematic soils/marl
4. Determination of the physical, mechanical and chemical characterization of the involved materials as, 19 mm aggregate, wadi sand, tuff, and glass sand.
5. Concrete trial mixes with different percents of ash as Portland cement substituent.
6. Determination of strength parameters for ash-soil mixtures at various ash content
7. Determination of compressive strength at 7, 28, and 56 days of concrete mixes with various ash content.
8. Production of low cost light weight block and similar construction materials..
9. Strength determination of different ash mortars made with various available natural materials such as tuff and glass sand.

The testing and investigation program will include various materials such as ash, soil, aggregate, tuff, glass-sand, ordinary Portland cement. The necessary physical, mechanical and chemical analyses were carried out to characterize the raw materials before and after their use at Toukan and Saket Georesearch office-Amman and the Natural Recourses Authority/ Department of Soil and Rock Mechanics Division. The samples were checked for possible geotechnical improvement or stabilization of the soil by mixing the materials with different proportions of ash. The production of some construction materials in which ash is an important goal of this work as a primary cementaceous material that can substitute for a suitable amount of Ordinary Portland Cement.

The bituminous limestone source rock was investigated through the application of necessary chemical and physical testing techniques. Fresh boulder size samples were quarried from the outcropped layers of the studied area. The whole rock bituminous limestone samples were crushed and mixed and three representative samples were obtained. The samples were labeled as B1, B2, and B3. The chemical composition for each sample was determined using the inductive coupled plasma technique (ICP). The mineralogical composition was also determined for each sample using the X-ray diffraction method. The average bulk density and moisture content was determined for B1, B2, and B3 samples.

2.1 Preparation and characterization of the representative ash samples

A bulk representative sample of about 800 kilograms was collected from various outcrops from El-Lajjun deposits. The sample was crushed using a jaw crusher to obtain bituminous limestone aggregates of 9 mm nominal size particles. The whole aggregates were mixed on a clean plastic sheet to avoid any contamination and were filled in plastic bags. The bulk sample was transferred to the Jordanian Ceramic Industries Company at Zarqa where it was divided into three homogeneous parts labeled as S1, S2, and S3. The samples were combusted separately at temperatures of 1200 ^{0}C, 925 ^{0}C and 525 ^{0}C respectively by using an automatically controlled electrical oven.

Each sample was allowed to cool down to the ambient temperature (30 ^{0}C) and then was ground under dry conditions to obtain the possible minimum grain size. Small ball mills and Los Angles machine were used. The moisture content of the combusted ash samples was checked directly after cooling and was found to be around 0.2%. The fine ash was filled immediately in tight plastic bags and was transported to NRA laboratories.

2.2 Testing program of ash samples

Representative samples of S1, S2, and S3 were packed in tight plastic bags for further analysis. X-ray diffraction (XRD) analysis, X-ray fluorescence (XRF) analysis, loss on ignition, SO_3 content, specific gravity, sieve analysis, and hydrometer analysis were carried out for the three ash samples. All possible physical and chemical properties of designated ash were accomplished before proceeding in any further experimental work.

2.2.1 Mineralogical and chemical composition

The chemical and mineralogical composition of ash is considered as the key factor that will determine the most suitable aspects in which ash can be utilized efficiently. Complete chemical analysis utilizing the induced couple plasma (ICP) technique and XRF in addition to XRD were carried out on S1, S2, and S3 ash samples.

2.2.2 Physical and mechanical properties

The reference representative ash sample was tested to determine its physical properties. Grain size, hydrometer, bulk density, specific gravity, liquid limit, plastic limit, shrinkage limit, moisture content in addition to compressive strength determination at 7, 28, and 56 days for ash containing mixtures were determined. Ash-sand mortar was prepared and tested similar to the same procedure that is used for OPC-sand mortars following (ASTM C109 compressive strength of cement sand mortar). This step was necessary to compare the OPC past strength with that of ash past and hence to predict the strength buildup of ash when it is used as a self cementaceous material.

Various concrete like mixtures, ash-mortars and ash-soil mixtures were prepared and tested according to the related standards in order to check the suitability of these mixtures for various engineering purposes.

2.2.3 Specific gravity

Specific gravity is the ratio of the weight of a given material particle to the weight of the same volume of water at 4 ^{0}C. This test is considered a primary test in most engineering problems as compaction, settlement, and weight–volume relationship in concrete mix designs, in addition to determine other index properties

2.2.4 Grain size analysis

Mechanical analysis (sieving) technique was used to determine the grain size distribution of gravel and sand. Sedimentation test (Hydrometer) technique was used to identify the clay and silt fractions passing # 200 sieve. The hydrometer method is based on Stokes' Law which indicates that a larger grain size of the same material will result in a larger terminal velocity when dropping through a fluid of known density.

Grain size analysis was carried out for the ash samples after grinding the whole sample using Loss Angles abrasion machine. Forty standard balls were put in the drum for 1500 revolutions to grind the ash particles to the minimum possible size. Grinding aims at increasing the surface area of the ash particles to its maximum in order to increase the efficiency of any expected reaction when it is subjected to mixing with other materials in the presence of water.

2.2.5 Consistency limits

Consistency for a particular soil is defined by the water content present when the soil changes its response to stress. This measurement has been further refined by establishing Atterberg Limits. These limits divide four different "states" of consistency. If the tested material is heavily saturated with water and then is dried out, it will move from a liquid state to a plastic state to a semisolid state and then to a solid state. The dividing line between the liquid and plastic states is the liquid limit (LL) following (ASTM D423-66). The dividing line between the plastic and semisolid state is the plastic limit (PL) following (ASTM D424-29). The dividing line between the semisolid and solid state is the shrinkage limit (SL) as shown in figure (2).

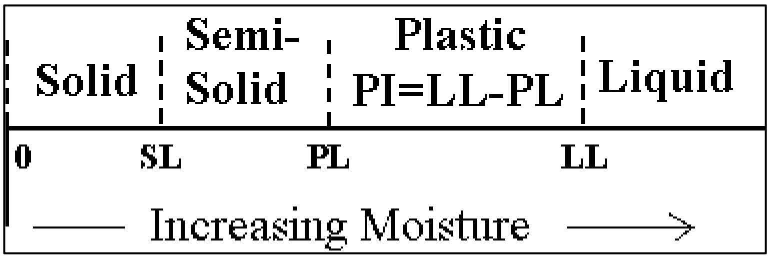

Figure (2) Consistency limits

The plasticity index (PI) is the range of the plastic region. These limits are expressed as a percent of moisture content. The experimental measurement of these limits requires a "liquid limit device". More specifically, it is a device that measures the water content at which the shear strength of the soil becomes so small that the soil "flows" to close a standard groove cut in a sample of soil when it is jarred in a standard manner. The brown clay (BC0) was mixed with various percents of S1, S2 and S3 , the same was carried out for the clayey marl (MC0) to the liquid and plastic limit variation with increasing the ash content. The liquid and plastic limit is determined for all of the ash samples S1, S2 and S3.

2.2.6. Strength behavior

Strength parameters are one of the important properties for utilizing the bituminous limestone ash (BLA) in many geotechnical applications. The production of various construction materials that can fulfill the international standards in which ordinary Portland cement (OPC) can be substituted with proper percent of the self cementation ash was discussed by Leonards and Bailey (1982). The free lime content of fly ash contributes to self-hardening (Yudbir and Honjo, 1991). The expected strength development due to CaO-Pozzolanic hydration reactions was investigated through unconfined compressive strength using standard cubic molds. The samples were prepared following ASTM C109 to determine the compressive strength of the ash samples in the same way that is used to determine the strength of hydraulic cement mortars at 7 and 28 days.

2.2.7 Unconfined compressive strength (ASTM D-2166-66)

An axial load is placed onto a sample, the stress is increased until the soil fails, or 15% strain occurs. This stress is known as the unconfined compressive strength. There is no lateral support on the tested sample for this measurement.

The ash samples S1, S2 and S3 were compacted at the maximum dry density and the optimum moisture content following the standard compaction procedures. Standard soil sampling tools were used to get undisturbed cylindrical samples from S1, S2 and S3 by pressing the tube sampler into the compacted sample while it is in the compaction mold. The trial failed, due to penetration refusal.

The experiment was repeated for S1, S2 and S3 by mixing with water at the optimum moisture content (35%), and was compacted into standard 5cmx5cmx5cm cubes into three layers, as the same process for testing ordinary Portland cement mortar. Nine cubes were cast from S1, the same is done for S2, and S3. The cubic samples were de-molded the next day, and left for an extra 24 hours to dry at ambient temperature and cured directly in water at normal ambient temperature (28 0C) until testing at 7 and 28 days. Uncombusted bituminous limestone sample was ground to the same grain size of the combusted limestone samples (ash). The sample was treated the same as the ash samples to check the possibility of strength buildup with increasing the curing time.

2.2.8 Compaction behavior

The density of ash is an important parameter since it controls the strength, compressibility and permeability. Densification of ash, as any material, will improve the physical, mechanical and engineering properties to the optimum. Compaction is a process by which the density of the unit weight of any lose material can be increased. Compaction efficiency to obtain the maximum density with the minimum applied energy depends on the nature of used material, moisture content at compaction, grain size distribution, in addition to the plasticity characteristics. Modified proctor compaction test was carried out for the ash sample according to ASTM (D1883).

2.3 Parent soil and marl samples

Two representative problematic samples were investigated. One was collected from brown sandy clay at 0.3 meters depth from Khalda, in west of the capital Amman, the other was collected from disturbed yellowish greenish marl at 0.3 meters depth from a toe of a hill, two kilometers to the south of Abu-Nsair. Both soil samples are considered as problematic soils due to their poor strength characteristics and high susceptibility to successive swelling-shrinkage behavior during the rainy winter and arid summer. Differential settlement under structures, damage to infrastructures especially roads, in addition to landslides problems associated with the yellowish marl in various locations. Landslides are common along the old Amman-Jarash road, and at some locations along the newly constructed highway (The Al- Urdun Highway).

The brown sandy silty clay sample was designated as (BC0), the yellowish marl was designated as (MC0). Each sample was mixed, pulverized and split into 10 homogeneous parts. The samples were labeled and saved in plastic bags after measuring the natural moisture content of the brown clay and marl samples.

2.3.1 Mineralogical and chemical composition

Complete chemical analysis was carried out for BC0 and MC0 samples in the Natural Resources Authority, laboratories directorate using the inductive coupled plasma technique and the X-ray fluorescence technique. The mineralogical composition was determined by using the X-ray diffraction technique in the Natural Resources Authority and University of Jordan. Appendices I-VIII show the XRD identification tables of the different mineral phases.

2.3.2 Soil classification

In order to determine the characteristics of the parent soil samples, an extended range of classification tests were carried out. Gradation curves were determined following ASTM D422. The characteristics of both soil samples BC0 and MC0 were determined through necessary classification tests. Hydrometer analysis for the fine fraction passing #200 sieve, liquid limit (ASTM D423-66), plastic limit (ASTM D-424-29, 1979) were determined. The soil samples were classified according to the Unified Soil Classification System (ASTM D2487) and AASHTO, specific gravity was determined according to (ASTM D854).

2.3.3 Proctor compaction test

Standard Proctor tests (ASTM D698) were performed upon natural soils BC0 and MC0 to determine the maximum dry density (MDD), optimum moisture content

(OMC), and MDD-OMC compaction curves. The compaction test elements are shown in table (5).The tests have permitted the analysis for strength–density-moisture relationship. As expected, the soil's strength generally has decreased at moisture levels that have exceeded the optimum value for compaction.

Table (5) Compaction elements

Specifications	Modified compaction
Hummer weight (lb)	10
Drop height (in)	18
Number of blows	25
Number of layers	5

After drying the soil samples, compaction test was started by adding 2% water by weight of the dry sample, mixing to ensure homogeneous distribution of moisture, then compacting the sample into 5 layers in the standard mold by a mechanical compactor as is shown in Figure (3). Successive trials continued by the addition of an extra 2% of water to the soil until the maximum density is achieved. The moisture content was calculated for each trial and then the dry density was calculated from the formula:

Dry density = wet density/ (1+moisture content)

Dry density versus moisture content for each trial is plotted on the compaction curve from which the maximum dry density and optimum moisture content are determined.

2.3.4 Undrained strength tests

Unconfined compression tests were conducted for the natural soils BC0 and MC0 at their maximum dry density and optimum moisture. In this manner, compaction curve for each soil was produced. Standard cylindrical samples of 38 mm diameter, 76 mm length were used to estimate the unconfined compressive strength (ASTM D 166-66, 1980). These samples were prepared by pressing standard cylindrical tubes into the compacted soil sample at its maximum dry density and optimum moisture content, this is because the strength of a given soil is highly affected by its density and moisture content.

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Figure (3) Mechanical compaction machine

2.3.5 Swelling –consolidation phenomenon

This phenomenon is related to volume changes of clayey soils due to variation in moisture content under certain normal stresses. Swelling is the volume increase due to increase of moisture content if the confining pressure is released in the presence of water. Consolidation is the volume decrease associated with decreasing of moisture content when the particles are forced closer together when the applied stress is higher than the swelling pressure. This is one of the serious problems that a rise when clayey soils are encountered in various civil construction works such as highways, roads, airports, etc. Swelling and consolidation were determined using front loading oedometer (ASTM D2435). The soil samples BC0 and MC0 were compacted to their maximum dry density at optimum moisture content. Undisturbed standard samples of the parent soils were prepared by inserting a 75 mm diameter, 20 mm thick standard molding ring into the compacted sample. Consolidation behavior was determined.

The collapse potential (Cp) is defined as dh/h or the change in the initial void ratio e/e_0 may be used to calculate the degree of collapse of the tested sample when water is introduced into the specimen.

The collapse potential (Cp) is calculated from one of the following equations

$Cp = \Delta h/h_0 = (h_0 - h_p)/h_0$

$C_W = \Delta h/h_0 = (h_p - h_w)/h_p$

Where

Δh: The change in thickness

h_0 : The initial specimen thickness

h_p: The specimen thickness after 24 hours under the applied pressure of 200 KN/m^2

h_w: The specimen thickness after 24 hours of saturation

2.3.6 California bearing ratio (CBR).

CBR is the ratio (as percent) of the unit load required to penetrate 2.5 mm into the soil by driving a rounded piston with a standard cross sectional area (19 cm^2) as shown in Figure (4).

CBR is considered as an essential test to determine the subgrade strength and the required thickness of flexible pavements for highways, airfield earth works as aprons, runways and taxiways. The samples were compacted to maximum dry density and optimum moisture in the CBR mold. The samples were soaked in water for 96 hours, under 4.5 kilograms of static pressure. The CBR mold was mounted with a flexed bridge to measure swelling of the sample every 24 hours until testing.

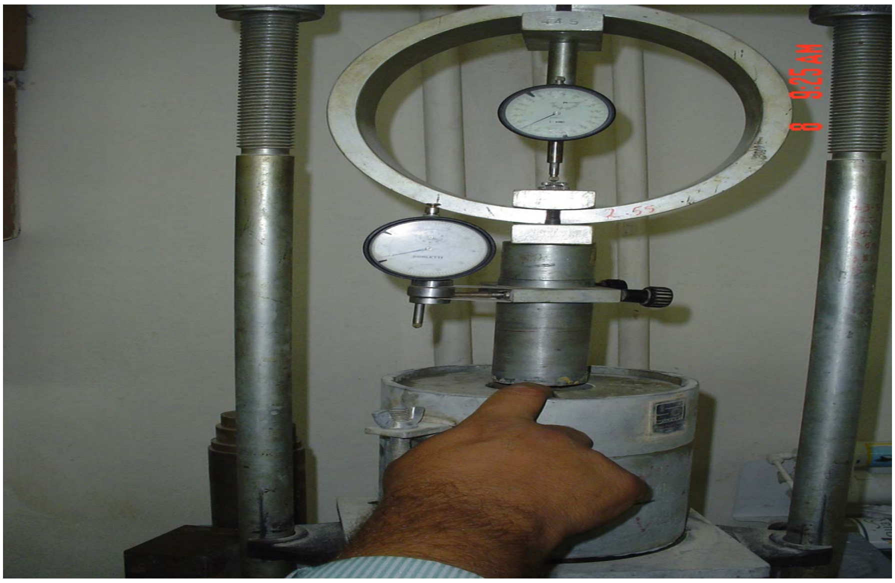

Figure (4) CBR apparatus

2.4. Ash-soil stabilization

The parent soil samples of brown soil and clayey marl where mixed with variable percents of S1, S2, and S3. To avoid confusion, a proper labeling system is followed. The sample label is composed of three parts. The first two letters indicate the soil type as BC (brown clay), or MC (marly clay). The third letter(S) indicates the ash followed by its type as (S1, S2, or S3), the number follows the dash sign indicates the percent weight of ash by the dry weight of the soil sample. The sample designated BCS1-20 indicates that brown clay mixed with 20% of S1 ash, while MCS3-100 indicates clayey marl mixed with 100% of S3 ash by dry weight of the soil sample (1:1). If the soil type is followed by the letter (S) without the number that indicates the ash type, it means that the mixture is repeated for the thee types of ash S1, S2, and S3 separately at the indicated ash %. The sample BCS-50 means that the brown soil is mixed with S1, S2, and S3 separately with 50% of ash content.

An evaluation testing of the S1, S2 and S3 ash potential to improve the engineering properties of brown clay was conducted to review the influence of ash addition on some soil's characteristics like, consistency, CBR, swelling and strength.

2.4.1 Classification of ash-brown soil mixtures

Six portions of the same parent soil (named as BCS-10, BCS-20, BCS-30, BCS-50, BCS-100, were used for various testing purposes.

Liquid limit, plastic limit in addition to plasticity index were determined directly after mixing for BCS-10, BCS-20, BCS-30 and BCS-50. The ash soil mixtures were reclassified according to the Unified Soil Classification System (ASTM D2487) and AASHTO. The effect of ash addition to improve the physical and mechanical properties of the plastic brown clay was inspected.

2.4.2 Compaction tests

The maximum dry density and optimum moisture content were determined for BCS-5, BCS-10, BCS-15, BCS-20, BCS-25, BCS-30 and BCS-100 following (ASTM D 698), compaction was carried out directly after mixing with the assigned ash percent without any delay. Maximum dry density and optimum moisture content were determined for the indicated samples in addition to dry density-moisture relationship curves.

2.4.3 Unconfined compressive strength

Standard cylindrical samples were taken from compacted ash–soil mixtures with defined ash content. The samples BCS-10, BCS-20, BCS-50 and BCS-100 were investigated to determine the undrained unconfined compressive strength of the compacted samples at the maximum dry density and optimum moisture content after 7 and 28 days using a triaxial testing machine by which lateral pressure can be applied on the tested sample if it is needed as it is shown in Figure (5).

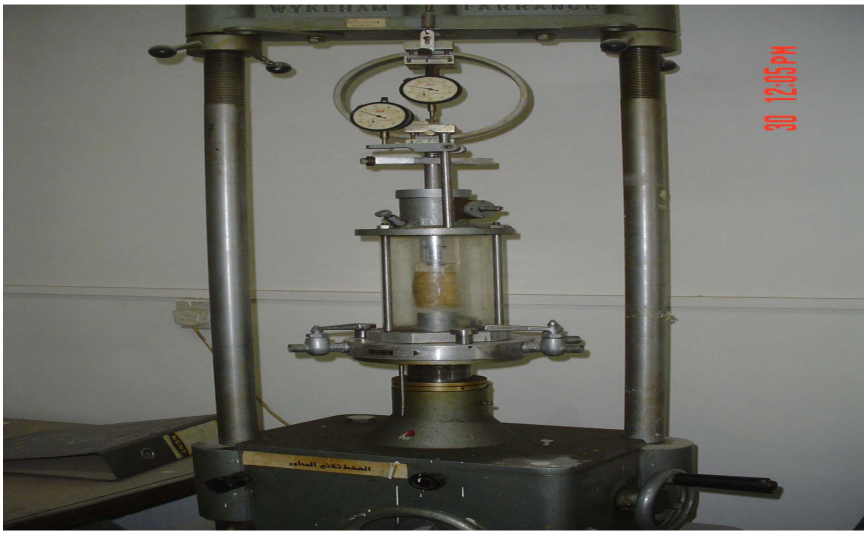

Figure (5) Triaxial testing machine

The compacted samples have shown a brittle structure. Brittleness is proportional to the increase of ash content. This phenomenon has caused unexpected difficulties in obtaining undisturbed samples at the determined optimum moisture content and appeared as an opposite behavior to normal clayey soils. It is considered as a positive result to be added to the geotechnical data. The ash-soil samples were mixed with six to eight percents of extra water above the optimum and compacted at the adjusted moisture content to obtain 38 mm diameter, 76 long standard samples. This experiment was the only possible way to estimate the soil strength at the worst expected compaction conditions. The samples were labeled and kept in tight plastic bags to avoid any variation of moisture content and cured at the dominant ambient temperature until the testing date.

Extra cylindrical samples were prepared by compacting wet ash-soil mixtures in the 38 diameter, 76 mm long sampling mold. The samples were demolded and were kept to cure in dry conditions under the dominant ambient temperature. The samples showed high stability against disintegration when immersed in water after 28 days curing.

2.4.4 California bearing ratio

CBR was carried out for BCS-50 and BCS-100. The samples were prepared at maximum dry density by compacting the samples into the standard CBR mold. The prepared sample was kept in the compaction mold in tight plastic bags. The samples were soaked for 96 hours directly before testing at 28 days. Swelling measurements were obtained every 24 hours during the soaking interval.

2.4.5 Swelling –consolidation behavior

Swelling and consolidation behavior were determined using front loading oedometer (ASTM D2435). Undisturbed standard samples of the compacted ash-soil mixtures were tested by inserting a 75 mm diameter, 20 mm thick standard molding ring into the compacted sample through the standard procedures. The soil samples BCS-15, BCS-50 and BCS-100 were compacted to their maximum dry density at optimum moisture content. Consolidation behavior was determined.

2.4.6 Stabilization of clayey marl (MC0)

The parent clayey marl sample was split into four homogeneous samples. The samples were designated as MCS-10, MCS-15, MCS-30, MCS-50 and MCS-100.

Liquid limit and plastic limit were determined at various ash content directly for soil classification purposes according to the unified soil classification system (USCS).

Maximum dry density and optimum moisture content were determined for MCS-10, MCS-20, MCS-30, MCS-50 and MCS-100 following the standard proctor compaction test. The unconfined undrained compressive strength was determined for MC0, MCS1-50, MCS1-100, MCS2-30, MCS2-50, MCS2-100 and MCS3-50 and MCS3-100 at 7 and 28 days.

California Bearing Ratio was determined at 28 days for compacted samples which were soaked for 96 hours directly before testing. Swelling measurements were carried out every 24 hours.

Standard cylindrical samples were prepared for testing at various ages, some of the samples were prepared at ash–soil maximum dry density and moisture content, the other samples were prepared by adding water to the mixture until the saturation state, then samples were molded into standard cylindrical samples as the undisturbed samples. Both types of samples were investigated. CBR testing was carried out on various ash-soil mixtures at 7 and 28 days using a single cell oedometer set as it is shown in Figure(6).

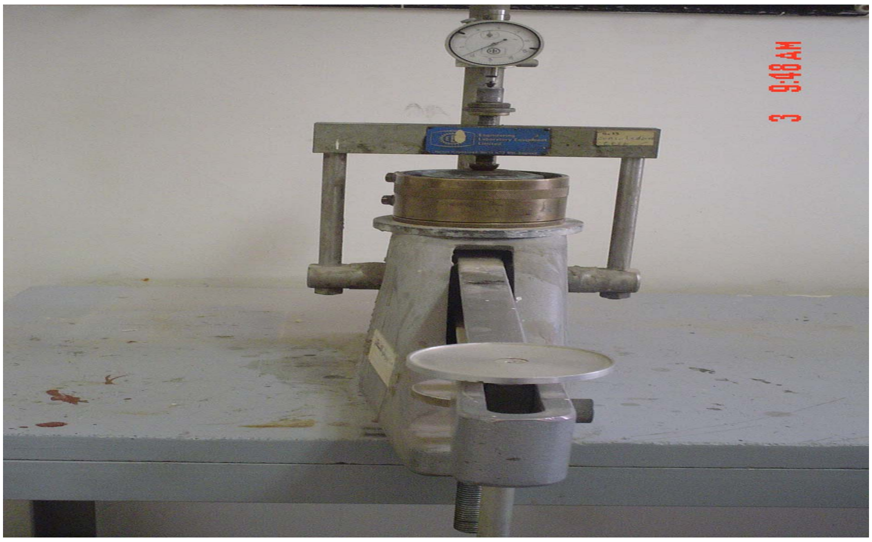

Figure (6) Oedometer set

2.4.7 Utilization of S2 ash as flowable fill

Flowable fill is a type of fill that can be produced from a proper cementaceous material to be utilized in certain engineering purposes. In general this type of fill is considered as an efficient solution for backfill activities in very narrow trenches and unreachable locations that must be filled and compacted using normal soils in various construction works. The cost is considered as a disadvantage when ordinary Portland cement, chemicals or other commercial materials are used.

Utilization of ash as flowable fill is considered as an advantage from the cost point of view as a self cementacious and binding material. Flowable fill which is a liquid like mixture or slurry can be produced by mixing ash with around 70% water by dry ash weight, then it can be poured easily in narrow service trenches that include pipes and cables. This fill can be re-excavated for maintenance purposes if needed in the future because it's low strength as a result of mixing ash with a very high water/ash ratio.

A flowable mix trial was carried out in the lab, the mix was designated as K1, samples were taken to check strength at 7, and 28 days. The samples were poured into standard 10x10x10 cm cubes and demolded the next day and left to air dry for another day. The samples were completely disintegrated when immersed in water. The trial was repeated again, and labeled as K2, but after demolding, the samples were kept in fully dried conditions until testing date. The compressive strength results were determined at 7 and 28 days.

2.5 Methodology concerning ash-concrete work

2.5.1 Utilization of ash in normal concrete mixtures

Concrete is a mixture of cement, aggregate, and water that possess strength over a certain period of time. The strength, durability, and other physical properties of concrete are controlled by many factors. The cement content and, water cement ratio (W/C) governs the strength of the cementaceous paste. The quality of coarse aggregate as type, hardness, nominal size, grain size distribution and chemical compositing must be investigated before use to check their compliance with national and international specifications.

All in aggregate gradation, passing # 200 sieve, percent of clay lumps and friable particles, quality of mixing water, total dissolved salts in sand fraction, and soundness characteristics of both coarse and fine fractions of the used aggregate must be carefully determined to get durable, strong and high quality concrete.

Two types of ash S1, and S2 were prepared and investigated as a partial substituent of ordinary Portland cement in normal concrete mixes. The total weight of cement and ash are equal to the weight of OPC content in the reference mix. XRF analysis for S1, S2, and S3 was carried out to characterize the samples chemically.

2.5.2 Aggregates

Partially crushed natural wadi coarse aggregate with nominal size of 19 mm, together with natural wadi sand bulk samples were collected separately from an operating quarry at Swema, east of the Dead Sea. The glass sand was collected from one sand quarry at Quira in south of Jordan. The proper percent of each ingredient was selected to fulfill the standard fuller curve and hence to obtain a well graded combined aggregate mixture

The physical properties of coarse aggregate, as absorption, specific gravity, abrasion, and passing #200 sieve were determined, while absorption, specific gravity, passing #200 clay lumps and friable particles, total dissolved salts were determined for the wadi sand and glass sand samples.

A reference concrete mix which is designated as (T0) was prepared following the standard mix design calculations to produce one cubic meter of concrete. The trial mixes are based on the proper proportions, water/cement ratio of 0.48 and cement content of 350 Kg of OPC per cubic meter of concrete. A high water reducing agent superplastisizer with normal dose was utilized. The reference mix components are summarized in Table (6).

Table (6) Ingredients by weight to produce 1m^3 of concrete.

Material	Weight kg	S.G	Volume(m^3)
Cement	350	3.15	0.111
Water	168	1	0.168
Voids	0.01	0	0.01
Ash	0.0	3.15	0.000
Coarse aggregate	908	--	0.289
Wadi aggregate	545	--	0.711
Glass sand	363	--	1.0
Total	2334	--	1.0

The required quantities of aggregate, cement and water were weighed accurately to mix 0.04 m^3 of concrete for sampling and testing purposes. Mixing was carried out using a small electric mixer. The slump test was measured directly after mixing, and nine standard 15x15x15cm concrete samples were prepared by filling each cubic mold with three layers and tamping each layer with 25 blows using a standard steel rod.

The concrete samples were labeled, and demolded the next day; the samples were directly cured in a water tank under normal ambient temperature until testing dates. Compressive strength results were determined at 7, 28, and 56 days using a high capacity digital compression testing machine as is shown in Figure (7).

Figure (7) Concrete compressive strength machine

2.5.3 Utilization of S1 ash type in normal concrete mixes

The S1 ash sample was prepared by direct combustion of the bituminous limestone sample at 1200 ^{0}C. A total of five trial mixes were carried out using the same materials, weights, proportions, conditions as in the reference mix, but with various ash and cement content taking into consideration that the total weight of cement plus ash is constant as 350 Kg per 1 m^3 of concrete.

These mixes were designated as T1-10, T1-20, T1-30, T1-40, and T1-50, with ash content of 10%, 20%, 30%, 40%, and 50% by weight of OPC respectively. The added ash replaces the same portion of the ordinary Portland cement quantity in the reference mix. The water/cement ratio was fixed for all trials, but the total water was different from one trial to another due to its ash content as shown in Table (7).

Table (7) Summary of T1 trial mixes with different S1 ash content

Ingredients	T0	T1-10	T1-20	T1-30-	T1-40	T1-50
Cement k/m^3	350	315	280	245	210	175
Water/Cement ratio	0.48	0.48	0.48	0.48	0.48	0.48
Ash S1 kg/m^3	0.0	35	70	105	140	175
Coarse aggregate kg/m^3	908	908	908	908	908	908
Wadi sand kg/m^3	545	545	545	545	545	545
Glass sand kg/m^3	363	363	363	363	363	363
Slump (mm)	10	11	10	9	12	10

The ingredients to prepare nine standard 15 x 15 x15cm cubes for each of the trial mixes were accurately weighed and mixed in a small electric concrete mixer for about 10 minutes. Slump was measured directly.

The same procedures for sampling were followed as in T0; samples were kept away from any disturbance until demolding the next day. The samples were cured at normal ambient temperature (around 28 ^{0}C) and the compressive strength results were obtained for mixes T1-10, T1-20, T1-30, T1-40, and T1-50 at 7, 28, and 56 days.

2.5.4 Utilization of S2 ash type in normal concrete mixes

The S2 ash sample was prepared by direct combustion of the bituminous limestone base sample at 925 ^{0}C. The same ingredients of aggregate, ordinary Portland cement, proportions were used. Three trial mixes, T2-10, T2-20, and T2-30 with 10%, 20% and 30% of S2 ash respectively were mixed.

Nine concrete samples were taken from each mix and cured for 7, 28 and 56 days to determine their compressive strength, and to compare these obtained results with the S1 compressive strength results using the same ash content. The trial mixes concerning S2 ash-concrete mixes are given Table (8).

Table (8) Summary of T2 trial mixes with different S2 ash content

Ingredients	T0	T2-10	T2-20	T2-30-
Cement kg/m^3	350	315	280	245
Water/Cement ratio	0.48	0.48	0.48	0.48
Ash S2 kg/m^3	0.0	35	70	105
Coarse aggregate kg/m^3	908	908	908	908
Wadi sand kg/m^3	545	545	545	545
Glass sand kg/m^3	363	363	363	363
Slump (mm)	10	12	11	9

2.5.5 Utilization of S2 in OPC free concrete

Utilizing the S2 ash and normal aggregate trial mix was carried out without the addition of ordinary Portland cement. The reference mix T0 was repeated but with 350 kilograms of S2 ash. The mix was designated as AG (Ash and Gravels), and thus replacing the OPC completely. Sampling was carried out using six standard 10x10x10cm cubes, each cube was filled in three layers, and each layer was tamped with 25 blows using the standard compaction rod. Samples were demolded the next day and were left for an extra 24 hours of drying at the ambient temperature, and then the samples were cured as conventional concrete until testing at 7, and 28 days.

Dry and wet density, in addition to compressive strength were determined at 7 and 28 days. The mix proportions by weight are given in Table (9).

Table (9) Ash-aggregates mix (AG).

Ingredients	Weight kg/m^3
OPC	0.0
Water/Ash ratio	0.6
Ash S2	350
Coarse aggregate	908
Wadi sand	545
Glass sand	363

2.5.6 Utilization of S2 in light weight ash-polyester morters

This trial was carried out utilizing S2 ash and polyester particles to produce very low density concrete like those made of self cementaceous material that can reach a reasonable strength when cured under normal ambient temperature. The bulk density of polyester particles was measured by calculating the weight of particles needed to fill 1000 cm^3, the measured density was 0.02 g/cm^3. The S2 ash was mixed with enough quantity of water until reasonable workability was achieved, the water ash ratio was 0.65 of dry weight of ash. The polyester particles were added and mixed with the wet ash until homogeneous ash polyester past is prepared, the mix was sampled in the same way that followed for normal concrete samples by filling 10x10x10cm cubic molds, each cube was filled in three layers, each layer was tamped by 25 blows using a standard tamping rod. The samples were demolded the second day and left until complete drying for an extra 48 hours under normal ambient temperatures, this is followed by wet curing as normal concrete samples since the prepared samples showed a high stability against disintegration. Wet density and dry density in addition to compressive strength were determined at 7 and 28 days. The required quantities to produce one cubic meter of light weight concrete are indicated in Table (10).

Table (10**)** Quantities of ash and polyester to produce 1m^{3}of light weight concrete

Ingredients	Weight kg/m^3
S2 ash	800
Polyester	20
Water	480

2.5.7 Self cementing mortars

Many trials were carried out to produce suitable mortars which can be used in different construction activities without the need of ordinary Portland cement. The S2 ash was investigated to produce such mortars by mixing with naturally occurring resources as tuff, glass sand and Sweileh (Kurnub, Lower Cretaceous) sand. These mortars can be utilized in plastering works, tiles and masonry block fixing, decorative and cladding works. It has been found that ash can be used alone as a self cementing material or can be mixed with glass sand, tuff, or both to produce workable, strong, durable low cost self cemented materials when cured at normal ambient temperature.

The raw materials were tested to determine their physical and chemical properties, sieve analysis, specific gravity, absorption and passing #200 were determined for tuff, glass sand, and Sweileh sand. The chemical composition of the tuff sample was determined by ICP technique.

2.5.8 Ash-tuff mortars

Tuff is a natural volcanic material that is characterized by very high porosity, low density, rich in SiO_2, and has a considerable content of Al_2O_3 in addition to Fe_2O_3. Huge quantities of this material are available in the eastern and north eastern provinces of the Kingdom, these natural recourses are utilized in various engineering and agricultural aspects

About 200 kilograms of reddish crushed sand size was sampled from Amani quarry at Tall Hassan in the vicinity of Al-Azraq area. The sample was ground to get the maximum possible passing #200 sieve fraction, the Loss Angles abrasion machine was used for this purpose to produce a bulk fine tuff sample that will be used later to investigate the ash-tuff mortars. Four mixes of ash-tuff mortar were prepared, S1-T-25, S1-T-50 samples that have 25% and 50% of tuff by weight of dry S1ash, and S2-T-25, S2-T-50 samples that were composed of S2 ash, 25% and 50% of tuff respectively. Standard cubic samples from each mix were prepared, labeled and cured in water until testing date. Compressive strength was determined at 7 and 28 days. The trial mixes of S1 and S2 ash-tuff mortars are summarized in Table (11).

Table (11) Summary of ash-tuff mortar

Mix designation	Ash ratio (weight %)	Tuff ratio (weight %)	Ash/water ratio
S1-T-25	1	25	0.83
S1-T-50	1	50	0.83
S2-T-25	1	25	0.83
S2-T-50	1	50	0.83

2.5.9 S1, S2 and S3-glass sand mortar

About 250 kilograms of white natural glass sand were sampled from one of the working quarries at Al-Qwera area. This sand was proposed to be used in various mortar mixes to provide an extra amount of SiO_2 and hence to get the maximum cementacious paste as a result of ash-alkali sand-silica reactions. Gradation, specific gravity, absorption and passing #200 sieves were determined. S1, S2 and S3 ash were mixed with glass sand as shown in Table (12). The compressive strength results at 7, 28 and 56 days were determined.

Table (12) Ash-glass sand mixtures

Mix designation	S1 (gr)	S2 (gr)	S3 (gr)	Glass sand (gr)	Water (gr)
S1-1	500			1375	540
S1-1	500			1375	540
S1-1	500			1375	540
S2-1		500		1375	560
S2-1		500		1375	560
S2-1		500		1375	560
S2-2		1375		500	
S3-1			500	1375	420
S3-1			500	1375	420
S3-1			500	1375	420
S3-A		250	250	1375	400
S3-B		166	334	1375	390
S3-C			1375	500	720

2.5.10 Ash-Sweileh sand

A bulk sample was collected from Hisban quarries. Hisban sand will be utilized in some ash-mortar mixes as an alternative of the glass sand.

A mixture of 500 grams of S2 ash and 1375 grams of Sweileh sand of multiple weights were mixed with a suitable amount of water until a homogeneous, workable mixture was achieved as the same proportions and procedure that was followed in Ash-glass sand Standard cubic samples of 5x5x5cm were prepared. The samples were cured and tested to determine compressive strength at 7 and 28 days.

2.5.11 Ash-OPC mortars

Several mixes of ash-OPC cement mortars were investigated. The purpose of these trials was to check the strength behavior of ordinary cement mortars when mixed with different percents of S1 and S2 ash at constant cement content in the mix. The base mix was designated as M0 which is composed of 1 part of OPC to 2.75 parts of glass sand at water cement ratio of 0.48 following ASTM C 910. Six trial mixes were carried out and designated as S1-3, S1-6, S1-9, S1-12, S1-15, S1-18 and S1-27 by adding 3, 6, 9, 12, 15, 18 and 27% of S1 ash by weight of OPC in the base mix M0 which is ash free. Additional trial mixes using S2 ash were carried out and designated as S2-15, S2-18 and S2-27 which contains 15, 18 and 27% of S2 ash in the mix. Extra mixes of S3-9, S3-15 and S3-27 were carried out also. Standard samples were prepared from each trial, cured and tested at 7 and 28 days. A summary of S1-OPC and S2-OPC prepared mortars are shown in Table (13).

Table (13) Summary of S1, S2 and S3 mortar mixes

Mix designation	Glass sand (gr)	S1 (gr)	S2 (gr)	S3 (g)	OPC (gr)	Water (gr)	Density gr/cm^3
M0	1375	00	-	-	500	242	2.06
MS1-3	1375	15	-	-	500	243	2.16
MS1-6	1375	30	-	-	500	243	2.18
MS1-9	1375	45	-	-	500	250	2.17
MS1-12	1375	60	-	-	500	250	2.21
MS1-15	1375	75	-	-	500	290	2.22
MS1-18	1375	90	-	-	500	310	2.16
MS1- 27	1375	135			500	330	2.0
MS2-15	1375	-	75	-	500	300	2.21
MS2-18	1375	-	90	-	500	320	2.22
MS2-27	1375	-	135	-	500	350	2.16
MS3-9	1375	-		75	500	400	2.21
MS3-15	1375	-		90	500	330	2.22
MS3-27	1375	-		135	500	420	2.06

Tables 14-18, summarize the laboratory techniques used to carry out in this work. The tables demonstrate the detailed tests that were performed on the bituminous limestone, ash, soil, marl, aggregates, mortar mixes and mixtures.

Table (14) Laboratory techniques used to characterize bituminous limestone, ash, brown soil, clayey marl and tuff

Sample type	Sample number	XRD	XRF	ICP	Density	S.G	Moisture content	Thin section
Bitum. Ls	1	X		X	X	X	X	X
	2	X		X	X	X	X	X
	3	X		X	X	X	X	X
	4	X		X	X	X	X	X
	5	X		X	X	X	X	X
Ash	S1	X	X	X	X	X	X	
	S2	X	X	X	X	X	X	
	S3	X	X		X	X	X	
Brown soil	BC0	X		X	X	X	X	
Clayey marl	MC0	X		X	X	X	X	
Tuff	T			X	X			

Table (15) Laboratory techniques used for mechanical testing for coarse and fine aggregates

Material	Gradation	S.G	Passing 200	Absorption	Abrasion
19 mm	X	X	X	X	X
9mm	X	X	X	X	X
Wadi sand	X	X	X	X	
Glass sand	X	X	X	X	
Swieleh sand	X	X	X	X	
Tuff sand	X	X	X	X	

Table (16) Testing procedures used for mortar trial mixes

Mortar mixes	Mix	Density	Ash			Compressive strength		
			S1	S2	S3	7 days	28 days	56 days
Cement sand mortar	M0	X				X	X	X
	MS1	X	X			X	X	X
	MS2	X		X		X	X	X
	MS3	X			X			
Ash-tuff	S1- T	X	X			X	X	X
	S2-A	X		X		X	X	X
	S2-B	X		X		X	X	X
Ash-polyester	AP	X	X	X		X	X	X
Ash only	S1	X	X			X	X	X
	S2	X		X		X	X	X
Glass sand	S1-1	X	X			X	X	X
Glass sand	S2-1	X		X		X	X	X
Glass sand	S3-1	X		X	X	X	X	X

Table (17) Testing procedure for ash, brown soil, marl and soil- ash mixtures

Material	Sample No.	Atterberg limits	Gradation	MDD-OMC	CBR (28 days)	U.C. Strength		Consolidation
						7 days	28 days	
Ash	S1, S2, S3	X	X	X	X	X	X	X
Brown clay	BC0	X	X	X	X	X	X	X
Ash-brown clay	BCS	X	X	X	X	X	X	X
Clayey marl	MC0	X	X	X	X	X	X	X
Ash-Clayey marl	MCS	X	X	X	X	X	X	X

Table (18) Summary of concrete trial mixes and related tests.

Concrete trial mixes	Mix designation	Ash free	With variable ash content	Density	Compressive strength		
					7 days	28 days	56 days
Ash-aggregate	AG			X	X	X	X
Normal concrete	T0	X	X	X	X	X	X
Concrete-ash	T1		X	X	X	X	X
Concrete-ash	T2		X	X	X	X	X

3. RESULTS

3. RESULTS

3.1. Chemical and mineralogical characterization of the bituminous limestone.

The concentrations of the major oxides and trace elements of the three representative samples of the parent bituminous limestone are given in Table (19).

Table (19) Chemical composition of the bituminous limestone from El-Lajjun *

	Sample 1	Sample 2	Sample 3	Average
SiO_2 %	12.2	12.6	13.6	
Al_2O_3 %	2.5	2.7	2.9	
Fe_2O_3 %	1.3	1.8	1.5	1.5
CaO %	25.7	26.9	28.4	27.0
MgO %	0.46	0.45	0.47	0.46
MnO %	0.005	0.005	0.003	0.004
TiO_2 %	0.13	0.14	0.14	0.14
K_2O %	0.5	0.47	0.44	0.47
P_2O_5 %	2.40	2.96	2.42	2.59
Li ppm	5	6	6	6
B ppm	22	22	26	23
V ppm	133	174	154	154
Cr ppm	474	467	534	492
Co ppm	46	48	56	50
Ni ppm	268	241	285	265
Cu ppm	110	124	118	117
Zn ppm	472	507	539	506
As ppm	91	117	87	98
Sr ppm	796	900	817	838
Y ppm	22	20	23	22
Nb ppm	16	20	19	18
Mo ppm	122	160	155	146
Ag ppm	3	4	4	4
Sn ppm	90	100	113	101
La ppm	8	5	5	6
Ce ppm	44	52	57	51
Pb ppm	34	38	41	38
W ppm	23	28	33	28
Zr ppm	661	722	415	599

*Analysis was accomplished by using the coupled plasma technique (ICP).
* Loss on ignition is 51.98 %.

The results in Table (19) reveal the following:

1- LOI forms 51.98 % and oxides form 47.66% of the whole rock composition.
2- The trace elements form 0.359% of the rock constituents and are considered relatively high.

It is clear that the parent bituminous limestone is composed of around 48% oxides which represent the composition of the ash (solid waste). The ash is composed of CaO, SiO_2, Al_2O_3, P_2O_5, and Al_2O_3 with average values of 27, 12.8, 2.7, 2.59, and 1.5 respectively.

Trace elements are presented in relatively high concentrations. Strontium (Sr) shows the highest concentration with an average value of 838 ppm, followed by zirconium (Zr) 599 ppm, while the concentrations of zinc (Zn), chromium (Cr) and nickel (Ni) are 506, 492 and 265 ppm respectively. Vanadium (V), copper (Cu), and molybdenum (Mo) values are also considered relatively high. Other trace elements are available in concentrations less than 100 ppm.

The X- ray diffraction (XRD) results of the bulk bituminous limestone sample has shown that calcite ($CaCO_3$) and quartz (SiO_2) are the major constituents. The XRD difractograme of the studied sample is represented in Appendix (IX). Fluroapatite $Ca_5(PO_4)_3F$ is found in one sample.

Thin section study of the bituminous limestone using the polarized microscope has indicated that the rock is biomicrite. The brown organic matter is intermixed with the micrite and is present filling the cavities of the microfossils. Figure (8) is a typical biomicrite sample representing El-Lajjun bituminous limestone deposits. The thin section illustrates that foraminifera and bioclasts are embedded in the organic rich micrite.

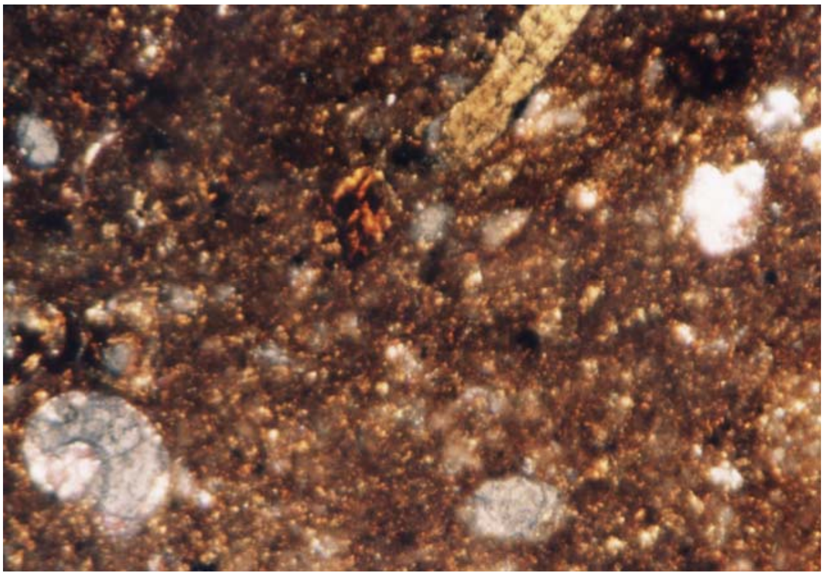

Figure (8) bituminous biomicrite from El-Lajjun under the polarized microscope (10x-X nicols)

The physical properties of the representative samples of the bituminous limestone from El-Lajjun are given in Table (20).

Table (20) Physical characteristics of El-Lajjun bituminous limestone

Physical property	Sample (1)	Sample (2)	Sample (3)
Bulk density g/cm^3	1.96	1.94	1.95
Specific gravity	2.39	2.38	2.41
Moisture content %	0.9	0.75	1.05
Color	Dark gray to black	Dark gray to black	Dark gray to black

The bulk density of the tested samples is lower than the bulk density of the ordinary limestone. This is due to the high organic matter content of El-Lajjun bituminous limestone deposits. The natural moisture content of the bituminous limestone is very low, and this is related to the dominant arid to semi arid environment of the study area. The tested representative samples are dark gray and black in color because of their high content of organic matter.

3.2 Ash characterization

The chemical composition of the ash samples is clearly dependent on the original composition of the parent bituminous limestone and the temperature of combustion.

The chemical composition of S1, S2, and S3 ash samples are given in Table (21). The samples are compared with the Ordinary Portland cement. The chemical composition in addition to the mechanical and physical properties of S1, S2 and S3 ash samples have revealed that the ash potential as a self cementing material.

Table (21) Chemical composition of S1, S2 and S3 ash samples as compared with standard OPC.*

Oxide Wt. %	S1 ash 1200 ^{0}C	S2 ash 925 ^{0}C	S3 ash 525 ^{0}C	OPC
SiO_2	33.80	25.30	21.01	23
Al_20_3	3.25	2.35	2.73	4
Fe_2O_3	1.44	1.37	1.16	2
CaO	47.84	45.21	43.52	64
MgO	2.47	1.63	2.44	2
P_2O_5	5.88	5.47	4.75	---
Na_2O	1.17	0.85	1.36	---
TiO_2	0.16	0.14	0.18	---
MnO	0.02	0.02	0.02	---

***** Analysis was done using XRF technique.

The SO_3, carbon content and the loss on ignition results for the ash samples are given in Table (22). The table shows a comparison with OPC. The OPC is completely free of any organic matter.

Table (22) SO_3, loss on ignition and carbon content of ash.

%	S1	S2	S3	OPC
SO_3	5.71	6.67	6.0	2
Loss on ignition	0.40	3.20	21.90	---
C	0.19	0.63	4.5	----

The chemical composition of S1, S2, and S3 shows a general proportional increase in the weight percent of all oxides. This is a result of the increase of the combustion temperature and the decrease of LOI as the temperature decreases. Figure (9) shows a variation diagram of the chemical composition of the ash with the increase of oxides as a result of increasing the temperature of combustion. The diagram shows that CaO and SiO_2 are the major constituents.

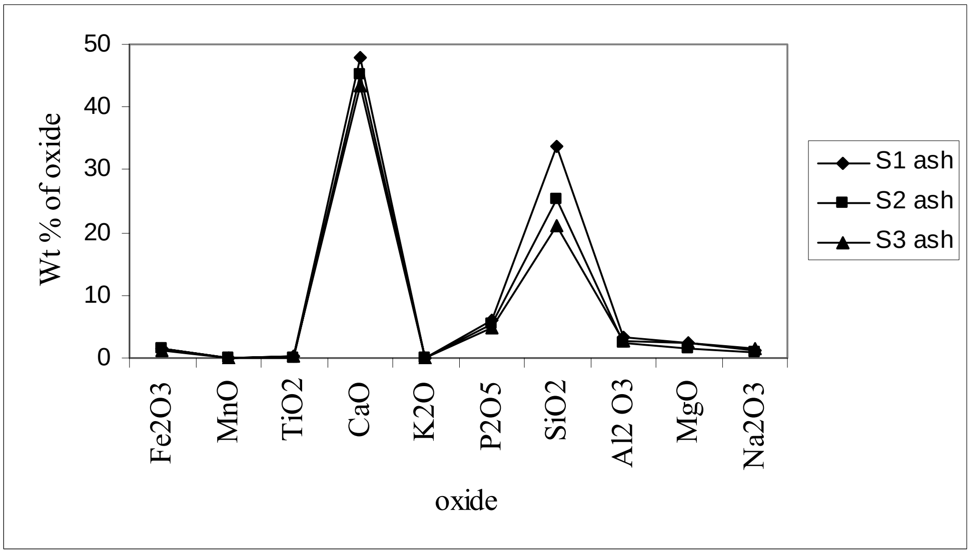

Figure (9) Chemical variation diagram of ash

The chemical composition of ash is typically made up of silicon, calcium, aluminum, iron, magnesium, and sulfur oxides, along with carbon and various trace elements. These components are probably found in the ash because of their high melting point and the lack of fusion during combustion. Quartz (SiO_2) sustains the combustion process as a result of its high melting point. Other parent minerals are not stable at high temperatures and are decomposed and/ or recrystallized to form new phases. Al, Fe, Ca, and Mg are usually present in the form of oxides Al_2O_3, hematite (Fe_2O_3), lime (CaO), and percales (MgO).

The SO_3 content of S1, S2 and S3 ash samples are considered very high when compared with OPC. The loss on ignition value increases with the increase of the temperature of combustion.

The chemical composition is considered the controlling factor that determines the physical and mechanical properties of the resulted self cementaceous material; this is due to the variation of the pozzolanic and alkaline content at different combustion temperatures.

The calcium oxide (CaO) is considered as the major component since it represents more than 45% by weight of the ash sample, followed by the silicon oxide SiO_2 (25 %). The aluminum oxide Al_2O_3 (2.5 %), iron oxide Fe_2O_3 (1.5 %) and other components are considered as minor components. It is clear that the concentrations of all the oxides in the ash samples S1, S2, and S3 are higher than the concentrations of the same constituents in the original bituminous samples. This is related to the removal of volatiles as CO_2 as a result of the oxidation of the organic matter and decarbonation of calcite during the direct combustion process.

It is clear from Table (21) that the ash is composed of two main parts, the pozzolanic part ($SiO_2 + Al_2O_3 + Fe_2O_3$), and the alkali (CaO) part. The composition is similar to the OPC composition. The comparison between the ash samples and OPC is summarized as follows:

1- The pozzolanic content of S1, S2 and S3 samples are 38.49, and 29.02% respectively. These values are higher or equal the pozzolanic content of OPC. The pozzolanic content of S3 (24.9 %) is lower than that of OPC (29%).

2- The CaO content of S1, S2, and S3 is 47.8, 45.2 and 43.5% respectively which is less than the CaO content in OPC (64%).

3- The SO_3 content of S1, S2 and S3 samples is 5.71% and 6.67% and 6.0% respectively which is too high compared with SO_3 content of OPC (maximum 2%).

4- OPC is carbon (C) free, while S1, S2 samples have very low content of carbon. S3 has the highest carbon content (4.5%).

Accordingly, the mechanical properties of S1, S2 and S3 are different from that of OPC due to the difference in the pozzolanic and alkaline content and the manufacturing processes concerning OPC production. OPC is manufactured through complex process by heating the raw ingredients up to the melting point at 1450 ^{0}C, followed by slow cooling to form a homogeneous clinker through melting, cooling and re-crystallization.

The ash samples S1, S2, and S3 are compared with class C and class F ash according to their chemical composition following ASTM standards C618-99 and Ferguson as given in Table (23) and Table (24).

Table (23) ASTM classification of class C, F compared with EL-Lajjun ash

	ASTM Standard C 618-99		El-Lajjun Ash		
Chemical characteristics	Class F ash	Class C ash	S1	S2	S3
$SiO_2+Al_2O_3+Fe_2O_3$ %	70	50	38.5	29.5	24.9
SO_3 , max %	5	5	5.71	6.67	6
Moisture content max 5	3	3	<2	<2	<2
Los on Ignition max%	6	6	0.4	3.2	21.9

Table (24) Ferguson et al., 1999, classification of class C and F compared with EL-Lajjun ash

	Ferguson et al. (1999)		EL-Lajjun Ash		
Oxide%	Class F ash	Class C ash	S1	S2	S3
SiO_2 %	54.9	39.9	33.82	25.3	21.01
Al_2O_3 %	25.8	16.7	3.25	2.35	2.73
Fe_2O_3 %	6.9	5.8	1.44	1.37	1.16
CaO %	8.7	24.3	45.84	53.61	43.52
MgO %	1.8	4.6	2.47	1.63	2.44
SO_3	0.6	3.3	5.71	6.67	6.0

According to ASTM standards, the S1, S2 and S3 ash samples have a lower pozzolanic content than both classes F and class C ash. The El-Lajjun ash (S1, S2, and S3) has a very high CaO content relative to the ASTM classification. This indicates that the El-Lajjun ash can be used efficiently in some proper aspects other than that indicated for both fly ashes of type F or type C in the international standards. The El-Lajjun ash is not complying with the classification proposed by Ferguson et al., 1999 as given in Table (24). This leads to the decision that S1, S2, and S3 samples have a unique chemical composition which is characterized by a very high CaO. Lime can be utilized to produce a self cementaceous material through the reaction with the pozzolanic portion of the ash or any added addition of naturally occurring substance. Materials rich in SiO_2 as silica sand, Swieleh sand (Lower Cretaceous, Kurnub Sandstone) or any tuffaceous material rich in SiO_2, Al_2O_3 and Fe_2O_3 could react easily with CaO to form hydrated products.

3.3 Physical and mechanical properties of ash

The initial set of tests regarding the gradation, Atterberg limits, and specific gravity have revealed that the average density of the tested samples is 1.15 g/cm^3, moisture content is 0.99 %, specific gravity is 2.5gr/cm^3. The results are summarized in Table (25). The effect of water content on compressive strength is significant and therefore the water content of the sample has to be carefully controlled.

Table (25) Physical properties of S1, S2 and S3

Ash type	Specific gravity	Moisture content %	Bulk density g/cm^3
S1	2.5	0.6	1.14
S2	2.48	0.9	1.13
S3	2.45	1.1	1.18

The bulk density of S3 ash sample is slightly more than the S1 and S2 samples because CO_2 incorporated in calcite structure of the S3 sample is not removed as the combustion temperature 525 ^{0}C is below the calcination temperature. The moisture content of S1, S2 and S3 after combustion is related to direct absorption of humidity from the surrounding environment during the grinding and packing processes.

3.3.1 Grain size distribution of ash

Dry and wet sieve analysis results of the ground ash samples are given in Table (26). The results show that 66% percent by weight of the S1, S2 and S3 samples is passing # 200 sieve. The grain size distribution is illustrated in Figure (10).

Fineness is the reflection of the specific surface area that reflects the ash particle size which has a definite weight given in cm^2/gr. The prepared ash samples do not comply with the grain size requirements of Ordinary Portland Cement.

Table (26) Dry and wet Sieve analysis of S1, S2 and S3 ash samples

Sieve No.	Weight retained	Weight passing	Percent Passing
8	0.4	99.6	99.60
16	1.4	98.6	98.60
30	6.0	94	94.00
50	15	85	85.00
100	21.2	78.8	78.80
200	33.4	66.6	66.60

The fineness is a reflection of the grain size and considered as the primary factor that controls the intensity of hydration reactions of ash alkalis with the surrounding media. The finenesses test was not done because the testing apparatus is not available.

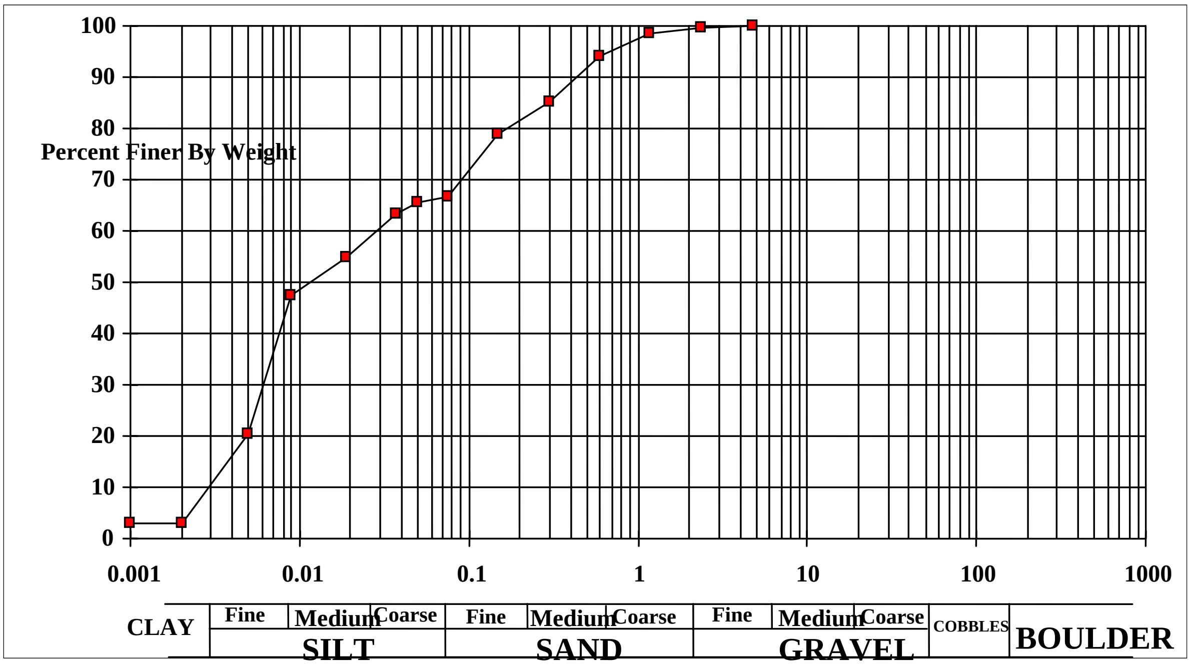

Figure (10) Grain size distribution of ground ash.

3.3.2 Consistency limits

The plastic limit results of the S1, S2 and S3 ash samples have shown that these samples do not reveal any plasticity. The determined plastic limit is 25 according to ASTM D424-24 (1974) and accordingly the ash is classified as a non plastic material.

3.3.3 Compressive strength determination

Sampling and testing procedures are carried out following (ASTM C109) for testing compressive strength of hydraulic cement sand mortar. All standard testing for strength determination of ash as a self cementing material has shown a high stability. No minor features of disintegration or disturbance of the prepared samples during the curing stage in water at normal ambient temperature (24 ^{0}C) are observed. This reflects the opposite behavior of normal soils that could not sustain under saturation conditions. On the other hand, ash behaved as cementaceous material that has the possibility to gain strength under normal curing time and conditions as the other different types of cements. This is confirmed through the compressive strength results of standard 5x5x5 cm cubic samples. Compressive strength results at 7, 28, and 56 days for the S1, S2 and S3 samples are shown in Table (27).

Table (27) Compressive strength results of S1, S2 and S3 ash samples.

Mix designation	Water/ash ratio	density gr/cm^3	7 days strength kg/cm^2	28 days strength kg/cm^2	56 days strength kg/cm^2
S1-1	0.65	I.87	11.7	41.1	52
S2-1	0.85	1.88	20.4	55.2	68.3
S3-1	0.85	1.73	7.3	18.6	19.1

The average strength build up at 7, 28 and 56 days for the S1 ash sample has shown an increasing trend while increasing the curing period. The same trend is detected for the S2 and S3 ash samples.

The compressive strength results of the S2 ash has shown higher compressive strength results at the same testing ages as compared with the S1 and S3 samples. The strength of S1, S2 and S3 is illustrated in Figure (11). The strength build up variation of S1, S2 and S3 is related to the variation of their chemical composition at the indicated combustion temperatures. This is due to the different crystalline structure that is related to phase temperatures at which S1, S2 and S3 were prepared.

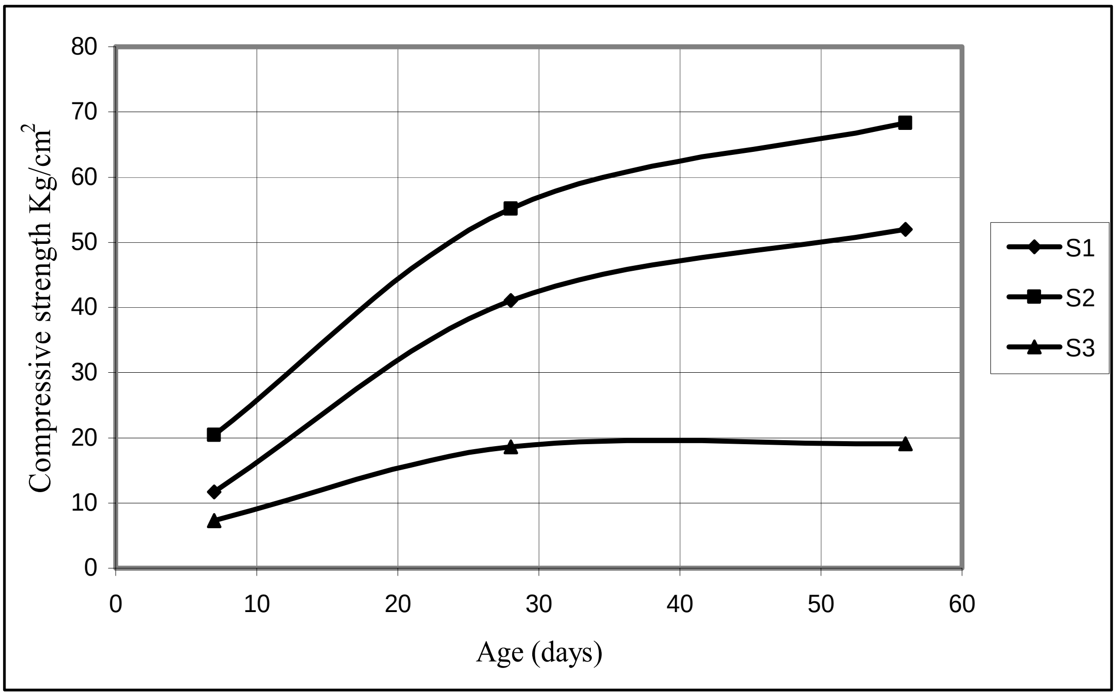

Figure (11) Compressive strength of S1, S2 and S3 ash

The bulk densities of both ash-sand mortars made from both S1, S2 and S3 samples are very close. The mixing water/ash ratio was raised from 0.65 in S1 mortar to 0.85 in S2 and S3 mortar. This addition of water is related to the exothermic behavior of S2 and hence the uncontrolled evaporation during mixing. The S1 and S3 samples have indicated no exothermic behavior. The S3 ash compressive strength results for the various mortar mixes is shown in Table (28).

Table (28) Compressive strength of various S3 mortar mixes

Mix designation	S3 Weight (gr)	S2 weight (gr)	Glass sand weight (gr)	Density (gr/cm^3)	7 days Comp. strength kg/cm^2	28 days comp. Strength kg/cm^2	56 days comp. Strength kg/cm^2
S3-A	250	250	1375	1.92	7.6	35.4	42.2
S3-B	334	166	1375	1.87	6.2	31.1	38.1
S3-C	1375	-	500	1.73	21.5	76.2	92.2

The S3-C mortar has the highest compressive strength results at 28 and 56 days and the lowest density when compared with the S3-A and S3-B mortar. The S3-S mortar contains a higher S3 ash content and less silica. The higher strength gain of S3-C is related to ettringite and to Ca-Al-Si, calcium aluminum silicate hydrate, (CSH) and calcium silicate hydrate, (CSH) as revealed by the XRD pattern (Appendix A). The strength buildup in S1, S2 and S3 ash mixtures is due to the chemical reactions that took place under the dominant ambient temperature in the presence of water to produce CAS and CSH.

The addition of water under surface normal conditions would lead to the following hydration reactions:

$CaO + H_2O \rightarrow Ca(OH)_2$ (Portlandite) (1)

$SiO_2 + Ca(OH)_2 + H_2O \rightarrow Ca.SiO_2.2H_2O$ (CSH) (2)

$Al_2O_3 + Ca(OH)_2 + H_2O \rightarrow CaO.Al_2O_3.2H_2O$ (3)

$SiO_2 + Al_2O_3 + Ca(OH)_2 + H_2O \rightarrow Ca.SiO_2.Al_2O_3.H_2O$ (CAS) (4)

In ash-soil mixtures and after initial mixing of ash and soil, controlled quantities of water are added, the pH of the ash-soil mixture will increases to above 10.5, which enables the destroyed clay particles to react. Aluminum and silicon oxides from the ash it self (the pozzolanic part) and the soil will react with the ash alkalis (CaO) in the presence of water to form calcium silicate hydrate(CSH) as in equation (2), and calcium aluminum silicate hydrate (CSH) as in equation (4). As these compounds form, a self cementing matrix will be created and will contribute to the strength gain of the ash-stabilized soil.

The above pozzolanic reactions have led to self cementing properties of ash and hence to strength gain. The pozzolanic and lime content in addition to the prevailing temperature would affect the type of the reaction. The P_2O_5 is considered as a deleterious substance in concrete mixes, because its responsibility for decreasing the compressive strength and the durability of the concrete.

3.3.4 California Bearing Ratio (CBR), ASTM D-1883-73

The achieved high compressive strength results and the obtained physical properties of the ash samples, California bearing capacity test for pure S1, S2 and S3 samples that were compacted to maximum dry density and optimum moisture content have shown a penetration refusal by the standard plunger under the maximum applied load. The test was carried out under applied pressure during the standard CBR testing procedure after 28 days curing at ambient temperature and soaking for 96 hours directly before testing. The CBR results revealed that S1, S2 or S3 ash can be used without any extra admixtures to provide highly stable embankments. Low cost pavements in which high quality base course can be replaced completely by a high strength and low cost material. Low quality base course with 45% abrasion value and 80% CBR has changed to strong stable material with CBR of 150% when it was mixed with 100% (1:1) by weight with S2 or S3 ash.

Extra strength gain is related to the action of ash as a non plastic filler material in base course and hence decreasing the required compaction energy. The efficient action of the ash as a self cementaceous material is the bonding of the base course grains in a strong cemented matrix and is characterized by a high shear resistance.

3.3.5 Compaction behavior

The ash samples S1, S2 and S3 are characterized as non plastic material; the percent passing # 4 sieve is 100%. The maximum dry density- moisture relationship of the ash samples S1, S2 or S3 revealed that the average maximum density is 1.31 gr/cm^3 at optimum moisture content of 34.7%. Figure (12) shows the average maximum dry density relationship with the optimum moisture content.

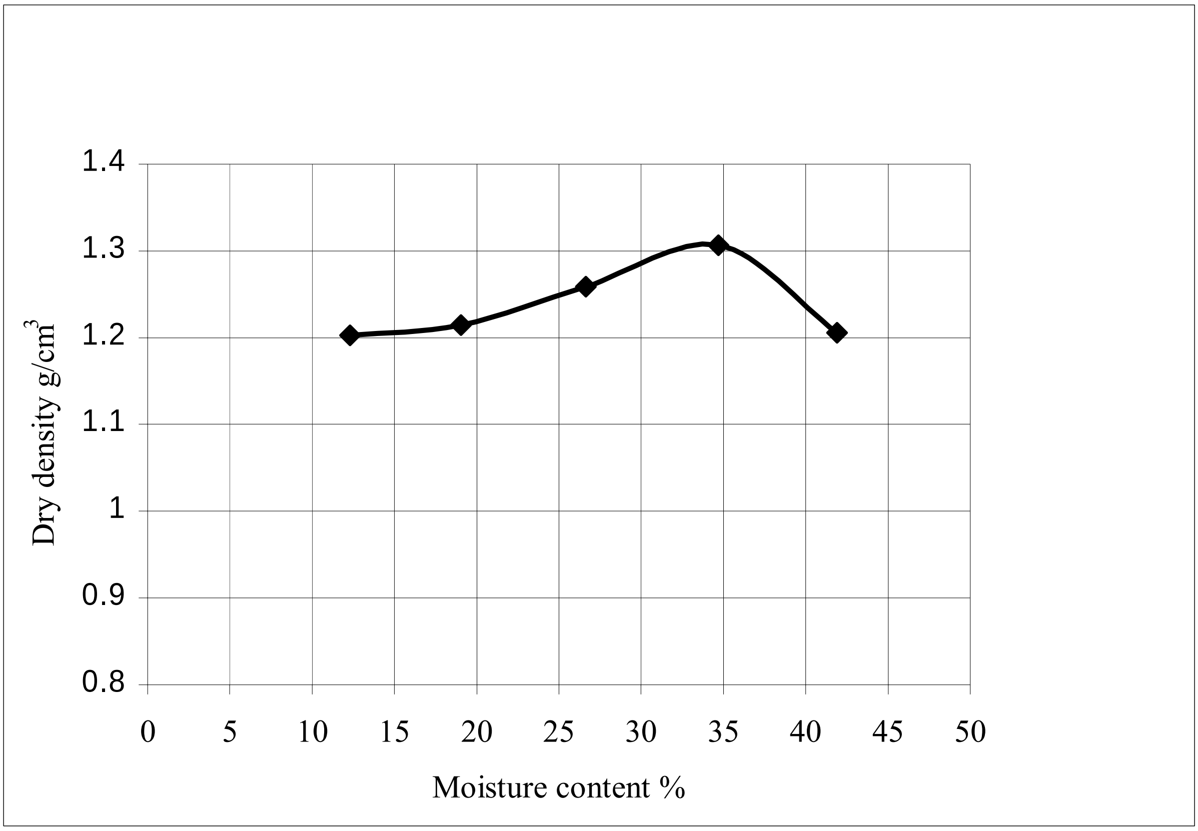

Figure (12) Maximum dry density-Optimum moisture content of ash (S1, S2 or S3).

The following results were obtained during the ash compaction test:

1. Dry density of S1, S2, and S3 ranges from 1.2 gr /cm^3 to 1.31 gr/cm^3 which is considered as a low density material compared with many types of soil as clay, marl, sand and other detrital materials, this can be considered as an advantage

to use ash as compacted backfill material behind retaining walls, slope stabilization, backfill on buried box culverts and similar structures.

2. The ash-soil sample has shown a stable behavior over a very wide range of moisture content which ranges from 19% to 30%.
3. The tested ash has shown a spongy like feature and pumping when moisture content is more than the optimum (wet of optimum). Excessive water is noticed to seep out the bottom of the compaction mold during compaction due to semi squeezing process. This phenomenon is referred to the high air void content of the ash which is replaced by water before compaction and squeezing out under compaction energy.
4. The higher void content of ash tends to limit the build up of pore water pressure during compaction, thus allowing the ash to be compacted over a larger range of water content (Toth. et al., 1988). The compaction behavior of S1, S2, and S3 is very close in maximum dry density and optimum moisture content.

3.4 Characterization and stabilization of brown soil

3.4.1 Characterization of brown soil

The complete chemical analyses (major oxides and trace elements) were determined using the ICP. XRD analysis revealed that quartz and smectites (Montmorillonite) are the major components of the parent brown soil (Appendix X). The major chemical constituents are given Table (29). The table indicates that SiO_2 is the major constituent, while Al_2O_3 and Fe_2O_3 are present in considerable amounts as compared with the very low CaO content. P_2O_5 is present in relatively low concentration and the organic matter content is measured as (3%).

The physical and mechanical properties are determined. Atterberg limits have revealed that the soil is medium to high plastic clay. The maximum dry unit weight is 1.81 g/cm^3 and the optimum water content is 16% at standard Proctor effort (ASTM D 698)

Table (29) Chemical composition of the brown soil*

Oxide	Wt %
SiO_2	67.1
Al_2O_3	12.4
Fe_2O_3	7.1
CaO	1.9
MgO	0.8
MnO	0.16
TiO_2	1.7
K_2O	0.8
P_2O_5	0.11

L.O.I = 7.5%, ICP technique

The particle size, passing #200, hydrometer analysis, specific gravity calculations are given in appendix A. As presented in Table (30), the clay content of the brown soil sample is 56.6% and passing #200 is 93 %. Gradation curve for this soil is presented in Figure (13). The curve shows that the soil is poorly graded. Atterberg limits showed that the plasticity index for the considered brown clay has PI >20 and liquid limit >45. The physical properties results for brown clay are summarized in Table (30). The high liquid limit indicates a high exposure to moisture sensitivity due to the presence of high percentage of the clay fraction. This result reveals the cause of problems exhibited by this soil as a subgrade material.

Table (30) physical and mechanical properties of brown soil (BC0)

Parameter	BC0
Liquid limit (LL)	46.3
Plastic limit (PL)	23.5
Shrinkage limit(SL)	11.7
Plasticity index	22.8
Passing #200%	93.65
Clay fraction %	56.6
Specific gravity	2.64
Unconfined compressive	6.5
Maximum dry density	1. 88
California Bearing Ratio	4.2

The soil is classified into A-7-6 according to the American Association of Highway and Testing Organization (AASHTO), and classified as silty clay (CL) according to the Unified Soil Classification System (USCS).

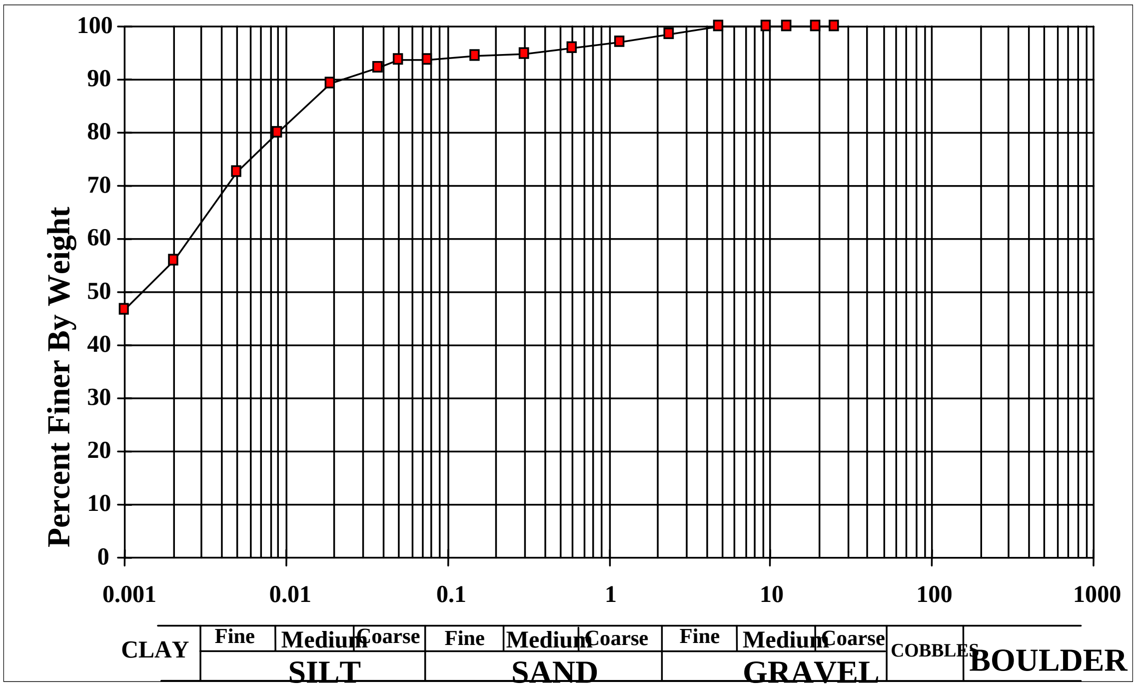

Figure (13) Gradation curve of the brown soil

The CBR value which is around 4.5 as indicated in Figure (14) reveals the normal values for the brown soil in the western parts of Amman. The result shows that this soil is not suitable to be used in its natural state as a structural fill or embankments in highways, roads or similar civil works. Many roads and yards which are constructed on the brown clay soil in the western parts of the capital Amman are subjected to continuous costly maintenance works.

Stabilization of brown soil with ash will improve the physical and mechanical properties of the poor soil and decreasing the maintenance costs to the minimum on the long run. The thickness of base course and sub base are reduced when the subgrade is characterized by higher CBR value. This is reflected on the cost, effort and quality of construction.

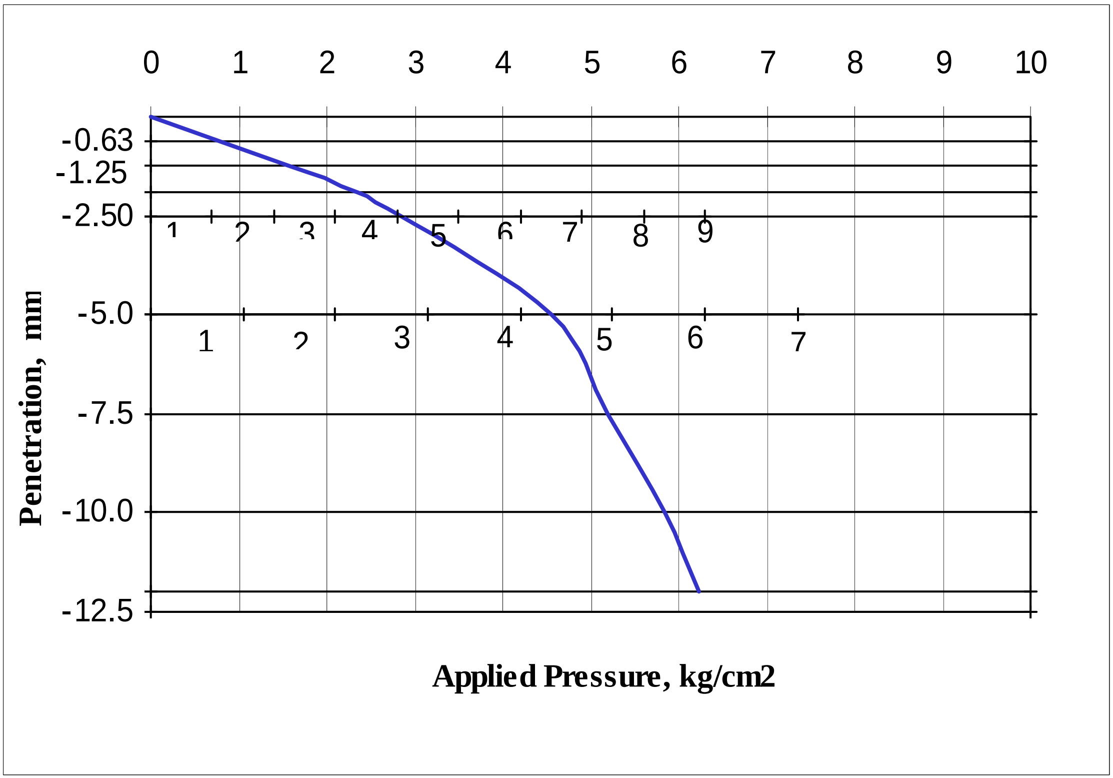

Figure (14) CBR result for the untreated brown soil

The maximum dry density is 1.81 g/cm^{3}at an optimum moisture content of 16.5% as shown from the Proctor compaction test curve in Figure (15). Brown soil as other clays is characterized by high porosity, low permeability, and compressibility. This soil is susceptible to slow changes in volume and unit weight when subjected to variation of moisture content under the applied loads.

The compaction of brown clay is difficult when the moisture content is above the wet of optimum due to the build up of pore water pressure which acts against the applied compaction effort. When necessary, it is compacted by special type of rollers as sheep-foot rollers.

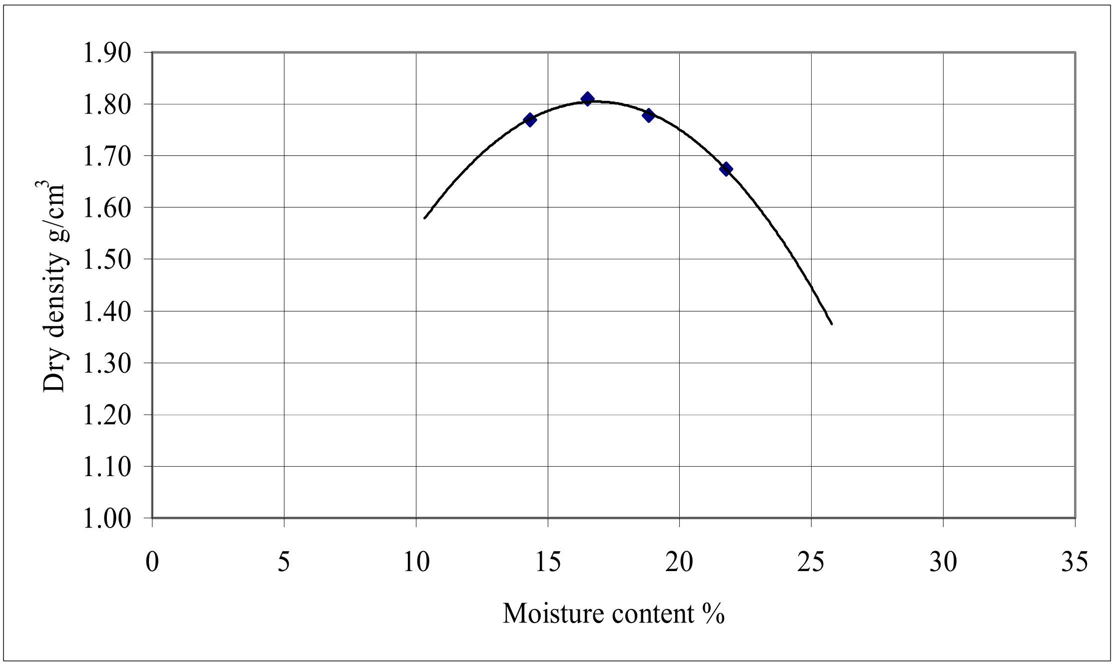

Figure (15) Density moisture content relationship of brown soil

The undrained unconfined compressive strength results have shown that the compacted sample has an average of 6 kg/cm^2 as shown in Figure (16). The results are normal for this type of soil at its maximum dry density and optimum moisture content. The unconfined compressive strength will be highly affected and decreased while increasing the moisture content in expansive clays in the western parts of Amman. This conclusion is built on personal experience upon site investigation of various tested locations for normal construction purposes.

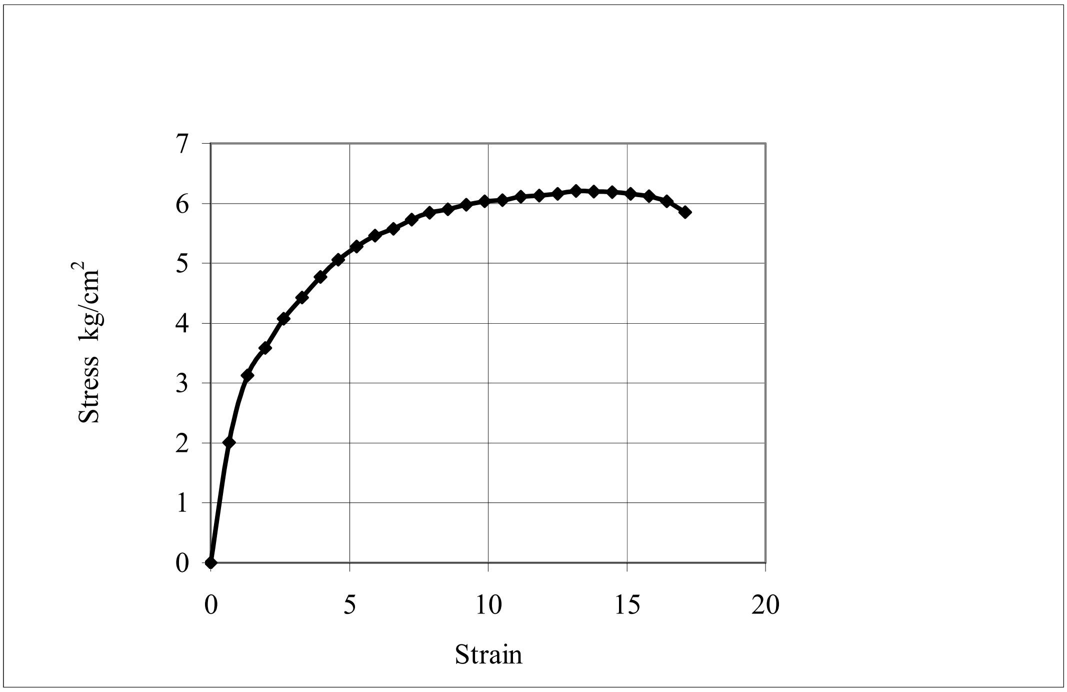

Figure (16) Unconfined compressive strength of the brown soil

3.4.2 Stabilization of the brown soil sample (BC0) with ash

The results of physical and mechanical testing of ash–brown soil mixtures have revealed that the parent brown soil sample (BC0) is changed to hard stable cemented soil. The results of ash-soil mixtures have shown a high cohesive properties and no disintegration signs as confirmed by the slaking test. The results of the various tests carried out on BCS-10, BCS-20, BCS-30, BCS-50 and BCS-100 follow.

3.4.2.1 Consistency limits

Atterberg tests are conducted on BCS-10, BCS-20, BCS-30, and BCS-50. The results have revealed that the consistency limits have changed when compared with the untreated sample. The results have shown that the liquid limit has decreased and the plastic limit has increased when the brown soil is mixed with S1, S2 or S3. The variation of the consistency limits is very close at the same ash content regardless of the ash type since all of the S1, S2 and S3 ash samples are not plastic. The results are shown in Table (31).

The group index for the same soil has also changed with increasing the ash content

Table (31) Summary of consistency limits for ash (S1, S2, S3) soil mixtures

Sample No.	BC0	BCS-10	BCS-20	BCS-30	BCS-50
Ash content %	0.0	10	20	30	50
Liquid limit	46	42	42	40	Non plastic
Plastic limit	23	25	26	28	
Shrinkage limit	----	------	26	20	
Plasticity index	23	17	16	12	
Group index (AASHTO)	A-7-5	A-7-6	A-7-6	A-6	
Group index (Unified classification)	MH	CH	CL	CL	

The liquid limit variation with ash content (S1, S2 or S3) is illustrated in Figure (17). The figure indicates that the LL decreases with the increase of the ash content regardless of the ash type.

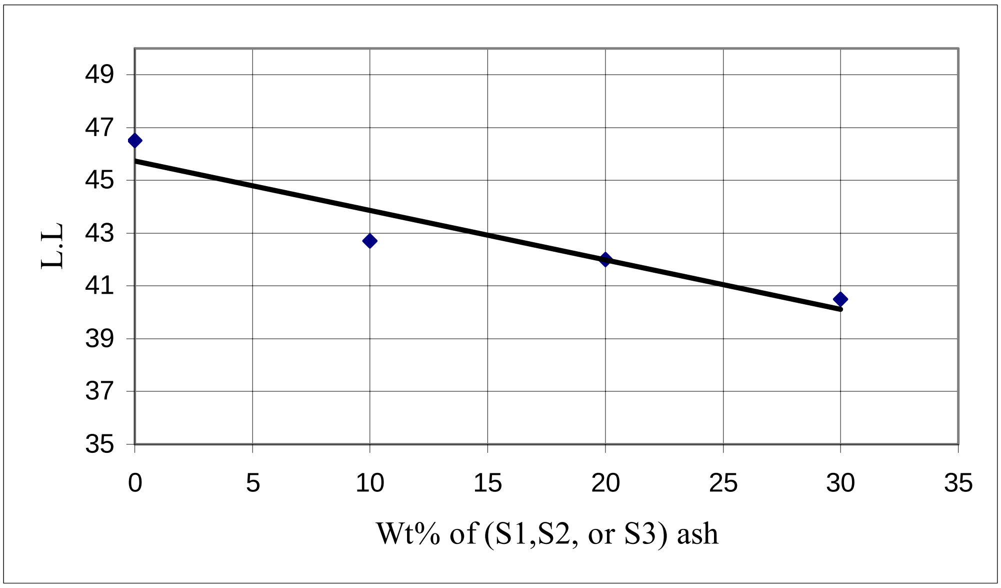

Figure (17) Liquid limit (L.L) variation with variable ash content in brown soil

The plastic limit shows increasing values with increasing the ash content as illustrated in Figure (18). The soil plasticity has decreased until the non plastic behavior has appeared when it is mixed with the proper percent of ash. This is referred to the proportional increase of the non plastic ash portion in the tested samples.

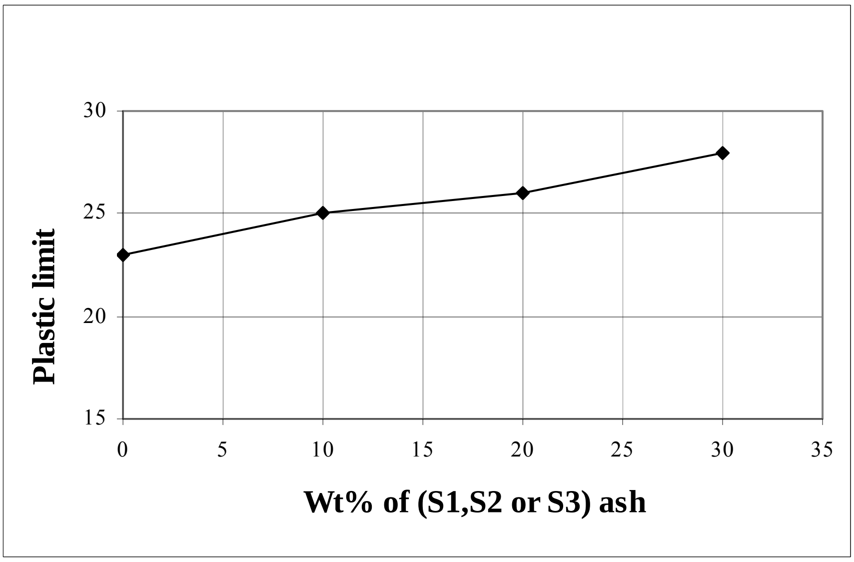

Figure (18) Plastic limit (P.L) variation with increasing ash content for brown soil

The plasticity index (PI) which is defined as LL-PL is decreasing with increasing the ash content as shown in Figure (19). This result means that the soil has improved by mixing with a proper amount of the non plastic ash.

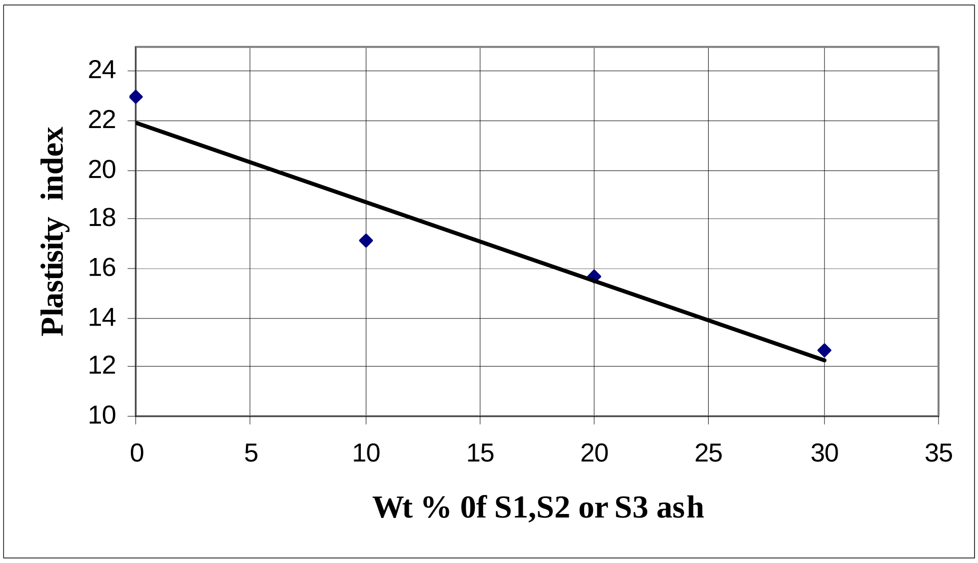

Figure (19) Plasticity index (P.I) variations for ash-brown soil

3.4.2.2 Compaction characteristics

The results of compaction tests that are carried out for ash-brown soil mixtures are summarized in Table (32).

Table (32) Summary of compaction results for ash-brown soil

Sample No.	BC0	BCS-5	BCS-10	BCS-15	BCS-20	BCS-25	BCS-30	BCS-50	BCS-100
% of Ash	0.0	5	10	15	20	25	30	50	100
MDD	1.81	1.8	1.79	1.78	1.78	1.77	1.76	1.57	1.34
OMC	16.5	16.72	17.14	17.24	17.37	8.16	18.56	27	33

The maximum dry density decreases with the increase of moisture content. The normal behavior of ash treated soil is illustrated in Figure (20).

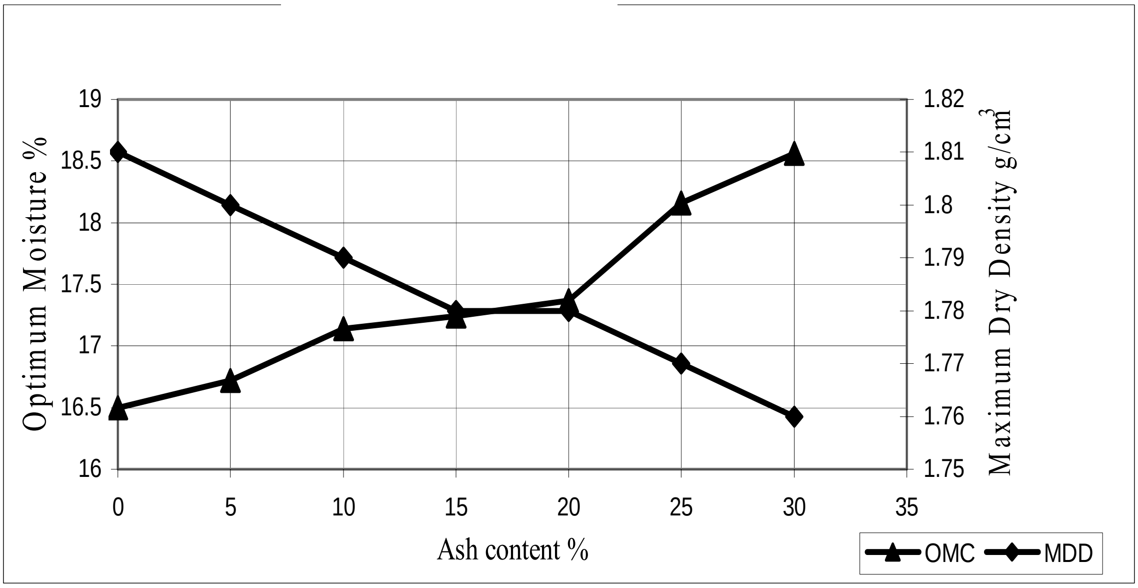

Figure (20) Density-moisture variations for ash-brown soil

The results of compaction behavior following ASTM D 698 for the ash-brown soil mixtures have revealed that the soil mixture has a wider range of moisture content at which the soil is compacted with lower effort. This is due to the decrease of plasticity index as a result of the action of the non plastic behavior of the ash soil mixture. This is illustrated in Figure (21). The maximum dry density of ash-brown soil mixtures has decreased at small values when the mixtures are mixed with up to 30% of ash. The limited decrease of dry density is related to the low ash content in all the compaction trials due to the high plasticity of the treated samples. The ash-soil mixtures (BCS-50) with 50% of ash and more have shown a non plastic mixture with a considerable decrease of dry density (1.57 g/cm^3) and a wider range of stability at higher moisture content when it is compared with the parent untreated sample. The MDD of the ash-brown soil (BCS-100) has decreased to 1.39 g/cm^3 when it is compared with 1.81 g/cm^3 of the original untreated brown soil.

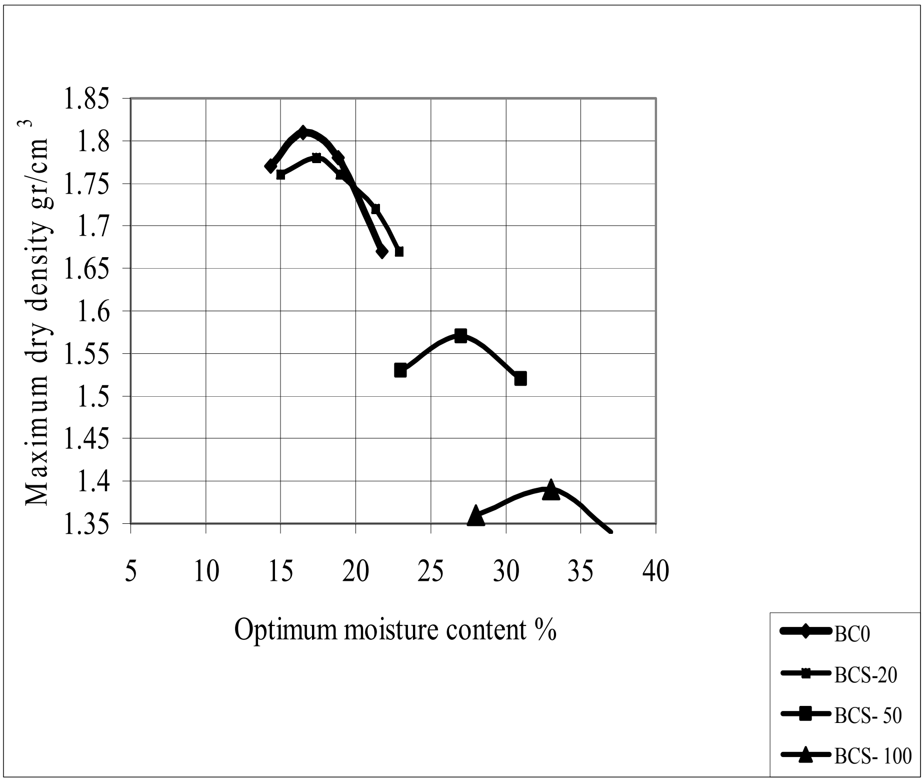

Figure (21) Compaction behavior of ash-brown soil mixtures

3.4.2.3 Unconfined compressive strength

Unconfined compressive strength is considered as one of the most important parameters that reflect the efficiency of introducing the EL-Lajjun ash for stabilizing problematic clays. The improved unconfined compressive strength results will be reflected on the other mechanical and engineering properties as shear strength, consolidation and swelling in addition to California bearing ratio of the ash treated soil. The results of unconfined compressive strength for S1, S2 and S3 ash-brown clay samples at different densities, moisture content and various curing periods are included in Table (33).

Table (33) Unconfined compressive strength result of ash-brown soil mixtures

Sample designation	Ash type	Compressive strength(kg/cm^2)	
		7 Days	28 Days
BC0		6.8	6.88
BCS1-30	S1	6.8	9.2
BCS1-50	S1	7.0	36.8
BCS1-100	S1	8.1	40.2
BCS2-15	S2	7.3	10.2
BCS2-30	S2	7.9	17.3
BCS2-50	S2	8.4	46.5
BCS2-100	S2	9.1	50.1
BCS3-50	S3	6.2	25.3
BCS3-100	S3	6.9	32.8

The buildup of unconfined compressive strength is not instantaneous and increases with the increase of time and ash content as shown in Fig (22). This is due to the exchange of calcium cations supplied from lime, (CaO). Hydrated lime reacts with clay mineral surfaces in the high pH environment creating cementaceous products (Qubain et al., 2000). Carbonation of portlandite $Ca(OH)_2$ is responsible for the precipitation of calcite which acts as a cementing material.

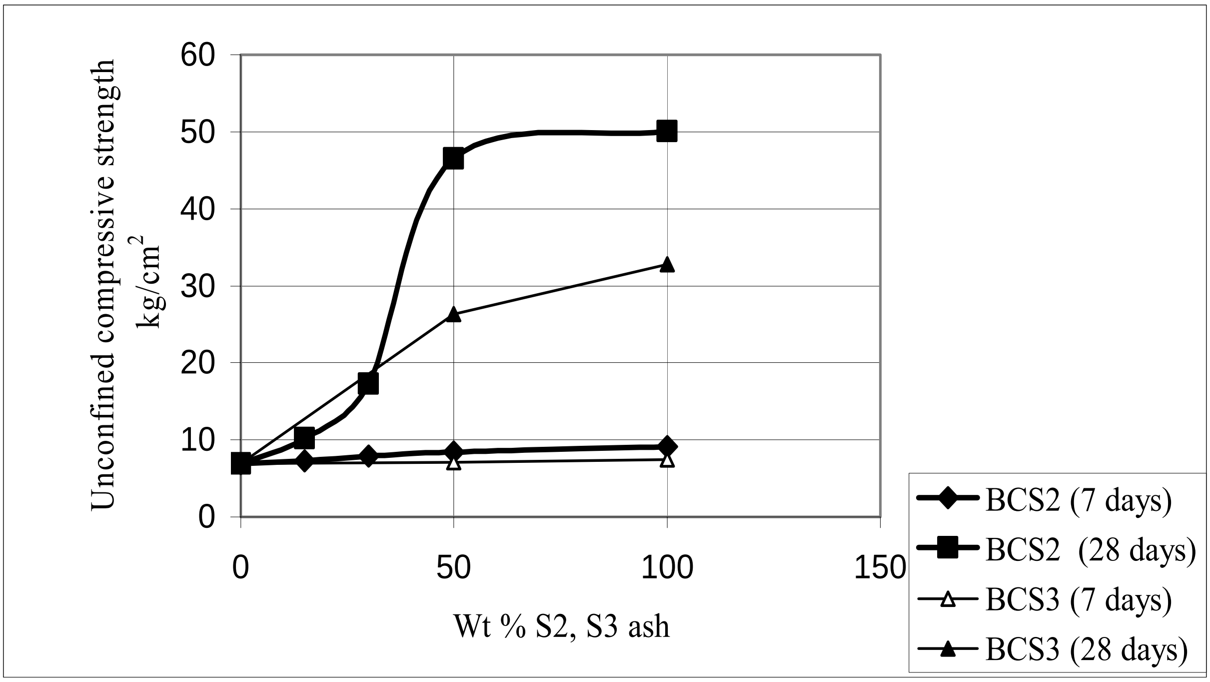

Figure (22) Compressive strength results of ash-brown soil mixtures

In this situation a significant level of strength gain is developed through a long term pozzolanic reactions. Calcium from the ash reacts with alumina and silica released from the clay mineral surfaces, leading to the formation of calcium silicate hydrates and calcium aluminates hydrates as it is revealed in XRD diffractogrames (Appendix XII.).

The pozzolanic reaction can continue for a long period of time, as long as the pH remains high (above 10). As a result of this reaction, the ash treated soil exhibits substantial strength gain, improvements in shear strength and long term durability. Field demonstrations have indicated that the improved soil properties are maintained over 20 to 40 years (Little, 1999 and Qubain et al., 2000.). Scanning electron microscope (SEM) images of the brown clay-S2 ash mixtures are shown in Figure (23).The images show an intensive build up of calcium silica hydrate (CSH) and calcium aluminum silicates (CAS) as a fibrous lattice in random directions. The bonding action of the reinforcement strong fibers is reflected in the strength parameters of the stabilized brown soil sample.

The presence of CSH and CASH phases are confirmed by the XRD results (Appendix XII).

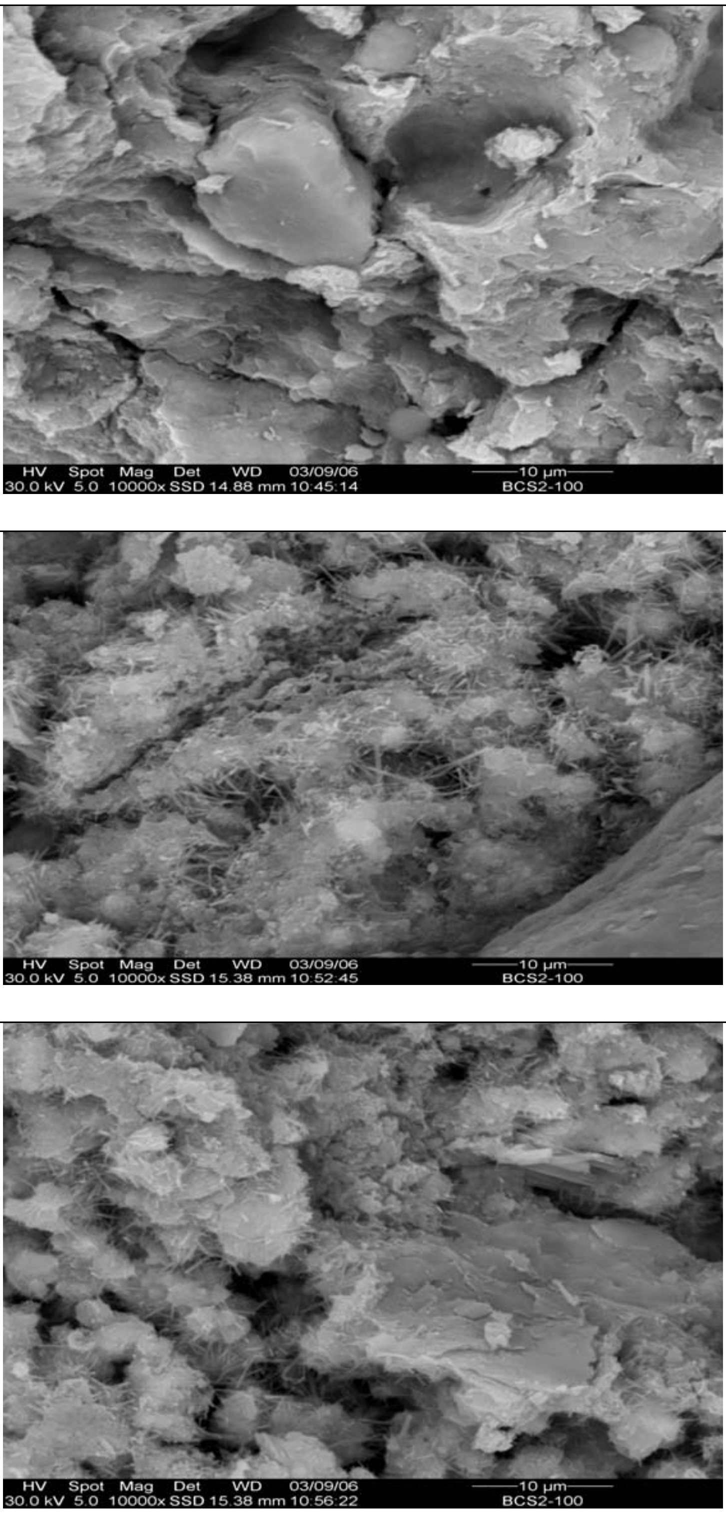

Figure (23) Scanning electron photomicrographs of brown soil-S2 ash mixture showing the growth of fibrous calcium aluminum silicate hydrates (CASH) and calcium silicate hydrates (CSH).

The mechanisms of lime–clay reactions are presented furthermore by (Boardman et. al., 2001). The physicochemical properties of soils are transformed by the addition of ash. When this stabilizer is added to a soil-water system, the formation of $Ca(OH)_2$ improves the soils workability because of the dewatering process. The modification reactions are generated by the exchange between Ca^{2+} ions of and hydroxide ions (OH) and the negative charges on the clay mineral lattice. This process is complex and is influenced by the location of the charge on the mineral structure. The ionic potential and the relative ion concentration, i.e. the concentration of the replacing ion in relation to the concentration of ions at the exchange sites, (Boardman et al., 2001).

The time factor is important for the ash alkalis to react with the pozzolanic part of the brown clay to change the poorly cemented soil into a stable cemented material. The compressive strength results after 7 days are variable due to the variation of ash content and are distinguished by poor gain of strength. The reaction will continue until the pH of the mixture is deceased by the alkali–silica reaction.

The long term strength buildup continues through the continuous ash- alkalis reaction with soil-pozzolanic constituent and or the addition of CO_2 to the ash brown clay mixtures. The CO_2 source is expected to be derived from the dissolved CO_2 gas in water or from the organisms that live in the media as bacteria or from the atmosphere. The long term reaction will continue as a result of the uptake of atmospheric CO_2 according to the following reaction.

$$CaO + CO_2 \rightarrow CaCO_3$$

The strength buildup of the S3 ash mixtures in spite of the law CaO content is related to the variable content of ettringite as revealed by the XRD results.

The stress strain results after 7 and 28 days for S2 ash-brown soil mixtures are shown in Figure (24).

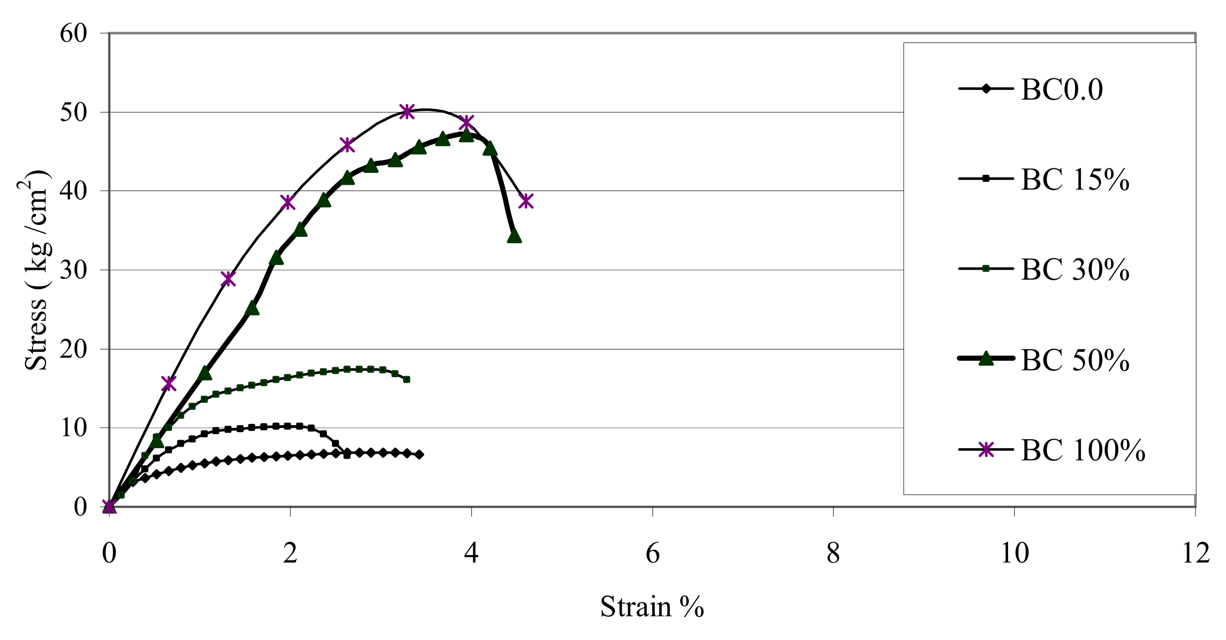

Figure (24) Stress-strain of S2 ash-brown soil mixtures at 7 and 28 days

The unconfined compressive strength for the samples BCS2-50 and BCS2-100 has increased from 8.4 and 9 kg/cm^2 at 7 days to 46 and 50 kg/cm^2 respectively at 28 days. This confirms that the time factor plays an important role in strength development on long term basis. The stress strain analysis of BCS3-50 and BCS3-100 is shown in Figure (25).

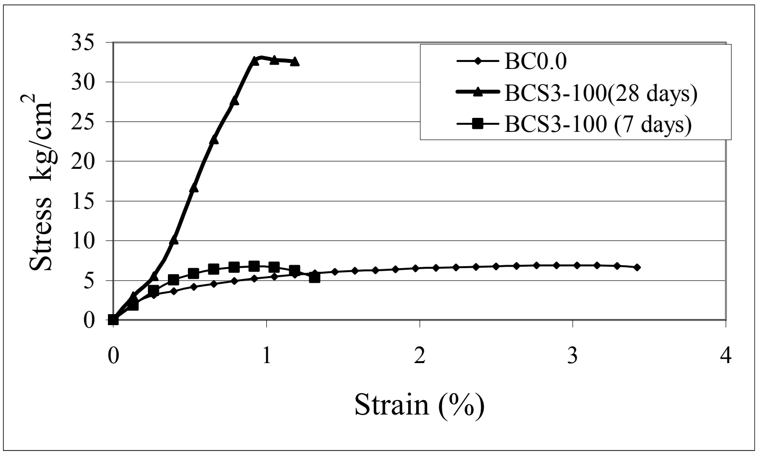

Figure (25) Stress strain of brown soil-S3 ash mixtures at 7 and 28 days

SEM images of the BCS3-100 are shown in Figure (26). The fine fibrous texture is unclear as in BCS2-100, and is replaced by coarser fibrous lattice of sulfate minerals which represent the binding material in the brown soil-S3 ash mixtures. CSH and CASH phases are also present as indicated by XRD results (Appendix XIII).

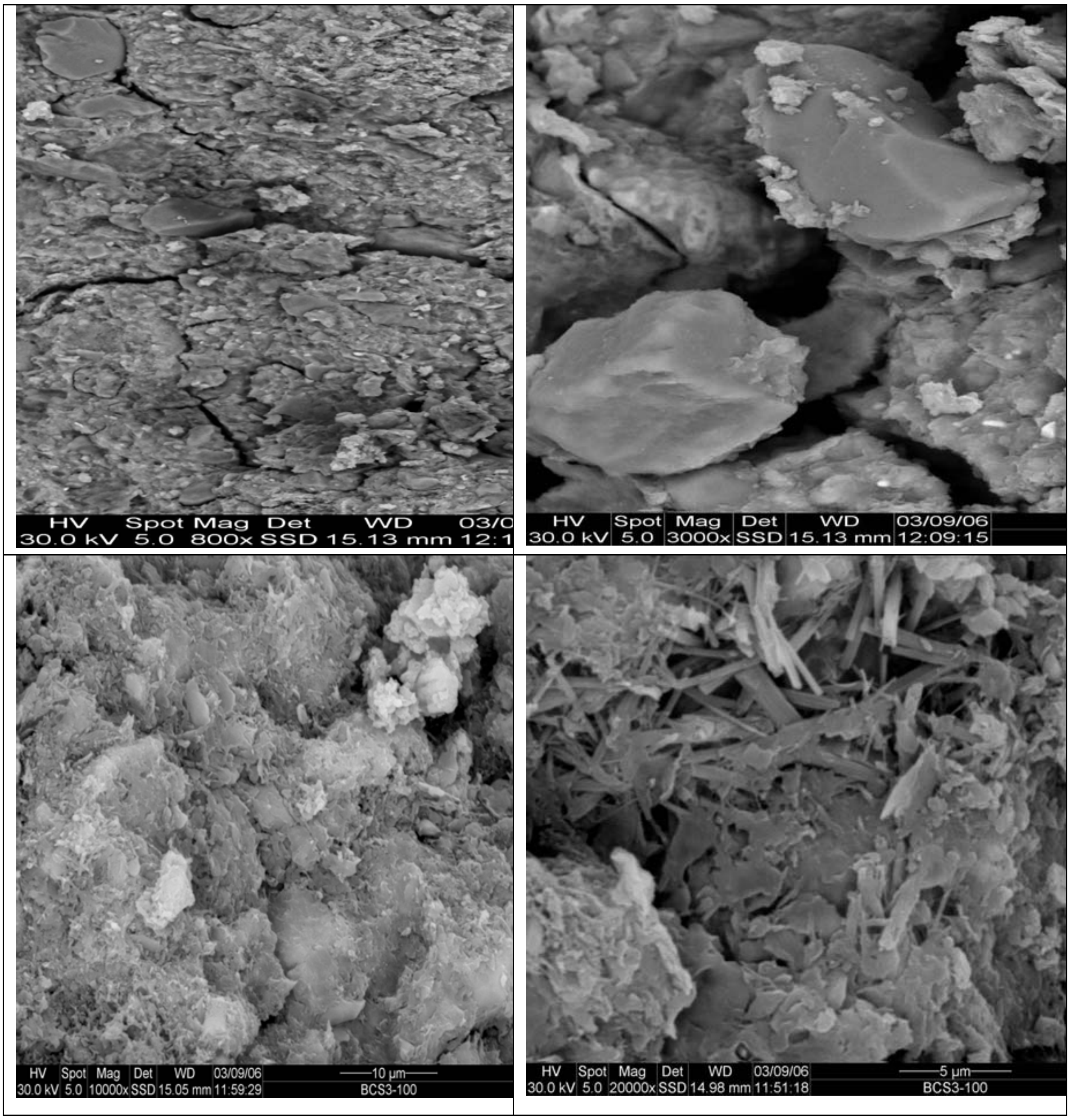

Figure (26) Scanning electron photomicrographs (SEM) of brown soil–S3 mixtures showing platy ettringite and fibrous calcium silicate hydrate

The difference in strength of BCS2-100 and BCS3-100 is related to the difference in binding mineral assemblages in both samples. This is revealed by the lower unconfined compressive strength and CBR results of BCS3-100 when compared with BCS2-100 testing results under the same conditions.

3.4.2.4 California Bearing Ratio

The California Bearing Ratio of the compacted S1, S2 and S3 ash-brown soil mixtures are cured for 28 days in tight plastic bags to maintain the optimum moisture content. Curing was at normal ambient temperature. The samples are soaked for 96 hours directly before testing; the results are shown in Table (34).

Table (34) CBR values of ash-brown soil mixtures after 28 days

Ash- soil mixture	CBR %
BC0	4
BCS1-50	85
BSC1-100	125
BCS2-50	100
BCS2-100	150
BCS3-50	100
BCS3 100	145

The test results have shown an increasing CBR values while increasing the ash content of the same ash type and vary with the ash type itself. The S1, S2 and S3 ash-soil mixtures have revealed very high CBR values when compared with the unstabilized clay as shown in Figure (27). The increased strength is mainly due to the increased cementaceous content of the ash-soil mixtures, which is related to the ash alkalis reaction with the ash pozzolanic content and ash alkalis reaction with the pozzolanic content of the soil itself along a proper curing time to give a highly cohesive mixture.

It is clear that the CBR of the ash treated soil (BCS2-50 and BCS2-100) has increased to 35 times the CBR of the untreated soil. The CBR results of the S3 ash-brown clay mixtures BCS3-50 and BCS3-100 have shown an increasing trend similar to the S2- soil mixtures but with very minor difference as shown in Figure (27).

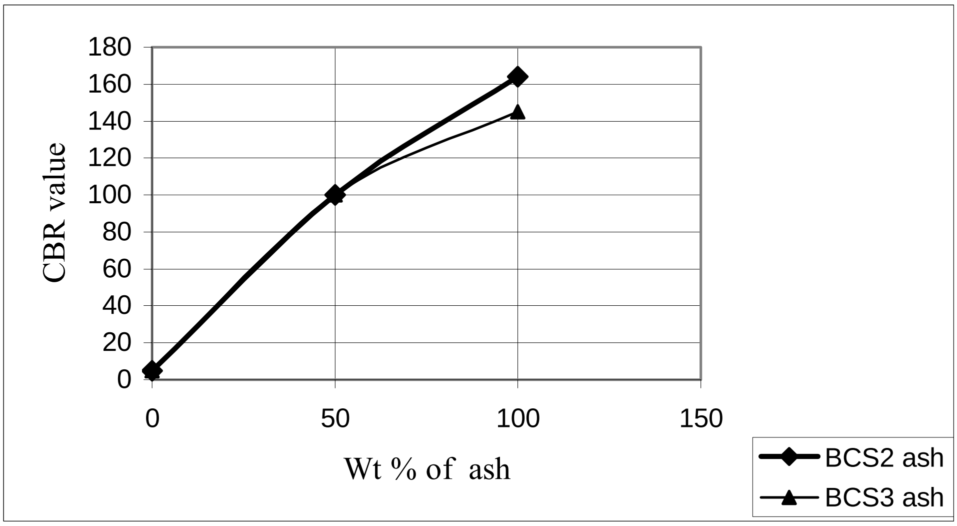

Figure (27) Increasing of CBR with the increase of ash content at 28 days

3.4.2.5 Consolidation-swelling behavior

The addition of ash to brown soil revealed an efficient action in improving the consolidation-swelling properties. The collapse potential has decreased to zero at 28 days when the soil sample is treated with 15% of S1, S2 or S3 ash samples as it is shown in Table (35). The reduction of consolidation-swelling potential is considered an important factor to prevent excessive water absorption of sandy silty clayey soils subgrade and hence reducing the pumping phenomenon under rigid pavements and reducing the subgrade stress by increasing the stiffness of the base course material instead of increasing its thickness.

Table (35) Collapse potential of ash-brown soil mixtures.

Ash- soil mixture	Collapse potential	
	7days	28 days
0	3.9	3.6
BCS-15	0 .05	0
BCS-30	0 .04	0
BCS-50	0 .05	0

However, the addition of extra quantities of ash above the optimum quantity has no effect on consolidation-swelling behavior as shown in Figure (28)

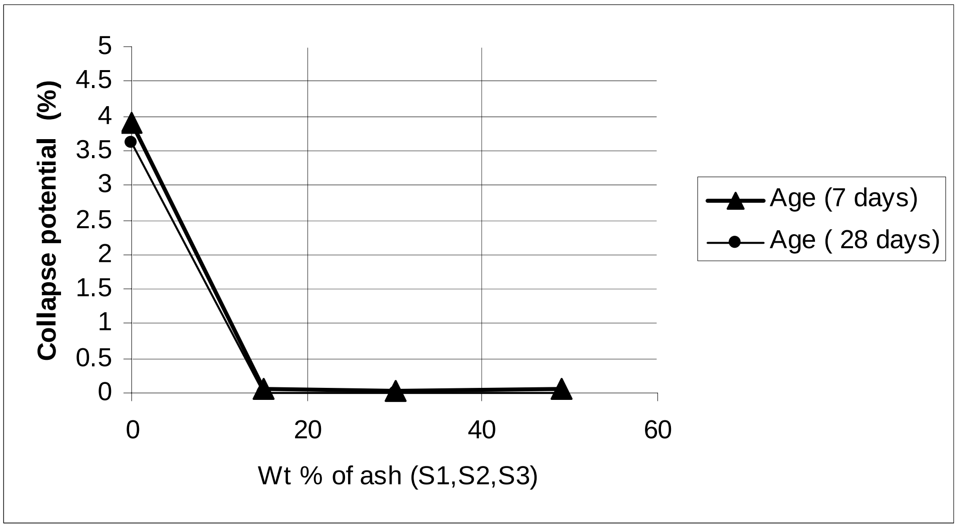

Figure (28) Collapse potential of ash-brown soil mixtures

3.5 Characterization and stabilization of the clayey marl (MC)

3.5.1 Characterization of the marl

The chemical analysis results concerning the oxides content of the marl sample is given in Table (36). The results have revealed that the SiO_2 content in the marl sample is lower than that of the brown clay. The total pozzolanic part of the marl sample is 25.3 % which is very low when compared with the pozzolanic content of the brown clay which is 86.6%.

The SiO_2 content of the considered marl is advantageous to increase the total available pozzolanic content of ash-soil mixture when the ash is added, and more self cementaceous material is expected to form.

The CaO content in the brown soil sample is (1.3%) and is considered as very low when referring to the marl sample.

Table (36) chemical analysis of the clayey marl*

Oxide	%
SiO_2	17.71
Al_2O_3	5.9
Fe_2O_3	1.7
CaO	32.2
MgO	3.1
MnO	0 .02
TiO_2	0.3
K_2O	0.9
P_2O_5	0.12

* Inductive coupled Plasma (ICP) technique.

* L.O.I. = 37.67.

The variation in the chemical composition will be reflected in the strength build up in both brown clay and marl when the high alkaline ash is added to both samples with the same ratios. The XRD analyses of the marl sample (Appendix IX) have shown that quartz (SiO_2), and calcite ($CaCO_3$) are the major components. Dolomite, kaolinite and smectites are also present.

The grain size distribution has revealed that 89.55% by weight of the marl sample is passing #200 sieve as shown in Figure (29). The specific gravity is 2.64 and the clay fraction is 47.5 as determined by the hydrometer analysis. The soil is susceptible to moisture content changes.

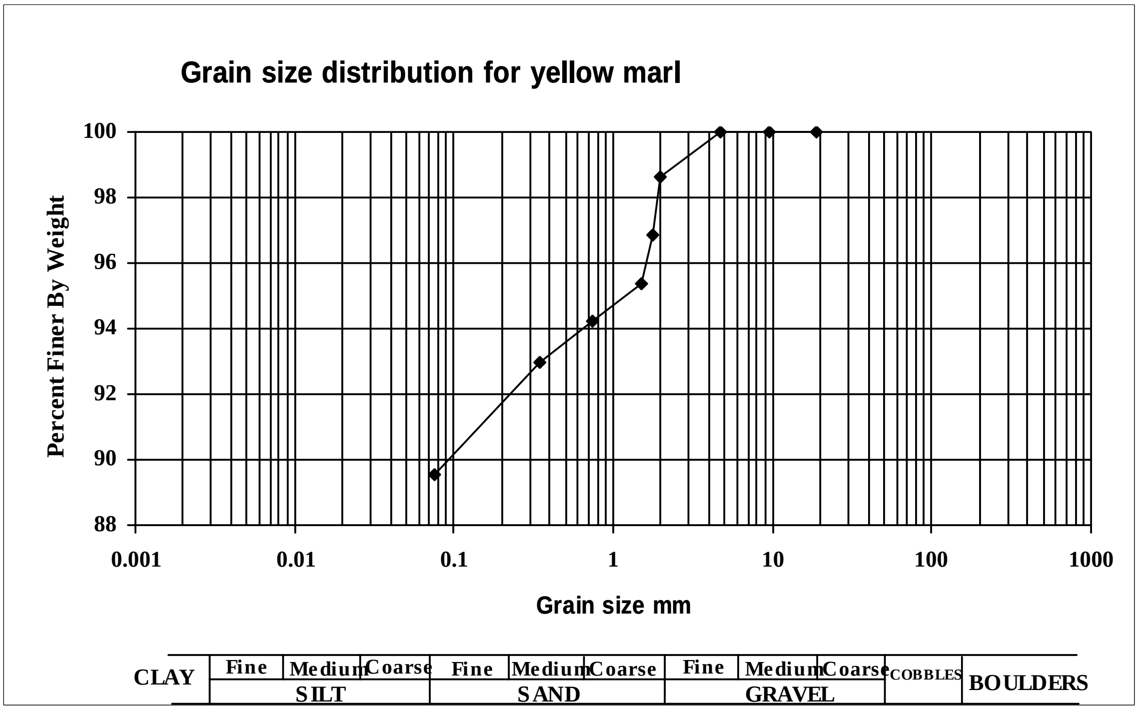

Figure (29) Grain size analysis for the clayey marl

The consistency limits indicated that the marl is medium to high plastic. The liquid limit is 42, plastic limit is 19, and the plasticity index is 23. The shrinkage limit is 14.34 and the shrinkage ratio is 1.91. The soil is classified as A-7-6 according to AASHTO system and as silty clay (CL) according to the Unified Soil Classification System (USCS). The activity of the soil is defined as:

Activity = Plasticity Index (PI) / clay fraction

The calculated value is about 0.5 which means that the soil has a medium degree of expansiveness.

A small increase in moisture content (3-4 percent) will transfer the soil from the plastic state to a state where it flows like a viscous liquid when conducting the liquid limit test. Many highways which were constructed on this type of soil in the vicinity of the capital Amman and northern districts suffered successive land slides and damage in spite of continuous stabilization and maintenance measures. Naur, old Irbid and Al-Urdun highways are examples.

The maximum dry density is 1.92 g/cm^3 with optimum water content of 10.78 at standard Proctor effort (ASTM D 698) as illustrated in Figure (30).

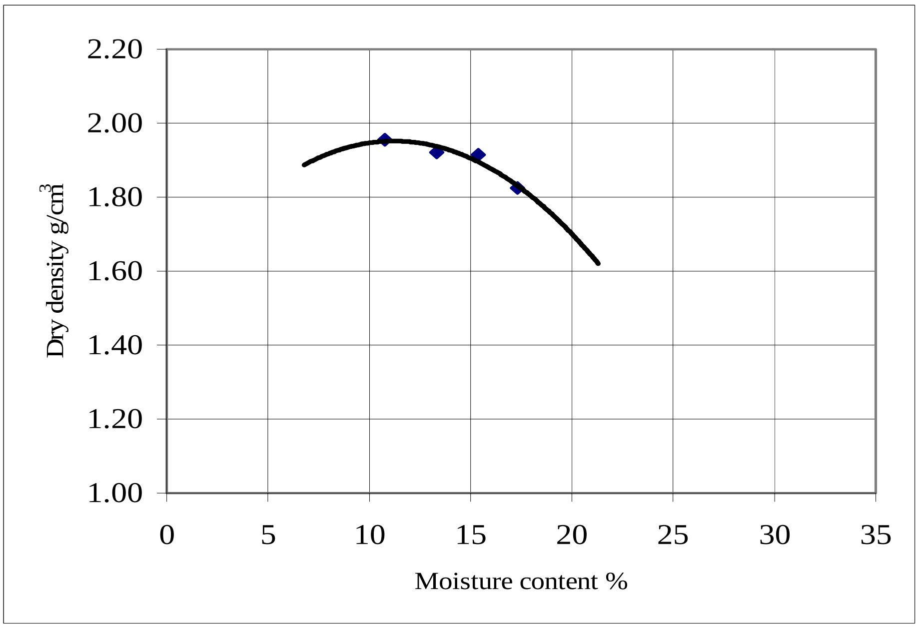

Figure (30) MDD-OMC for the clayey marl sample

The control of the moisture content of the soil sample is very important in order to meet the specifications regarding maximum dry unit weight required for road bases.

An excess of compaction effort will not have a significant effect on this parameter and for moisture levels higher than optimum, the effect on densification is null.

The unconfined compressive strength is determined by the uniaxial untrained test. The result has reached 4.7 Kg/cm^2 as indicated in Figure (31). The marl is highly susceptible to the change of moisture content during compaction. It has rapidly changed from the friable texture at the dry of optimum to sticky paste when the moisture content is increased with extra 2% above the wet of optimum and becomes difficult to compact. So this type of soil is considered stable at a very narrow range of moisture content range. Strength parameters as unconfined compressive strength, cohesion, CBR, and shear strength will be adversely affected when the soil is not protected against different water sources in the wet seasons.

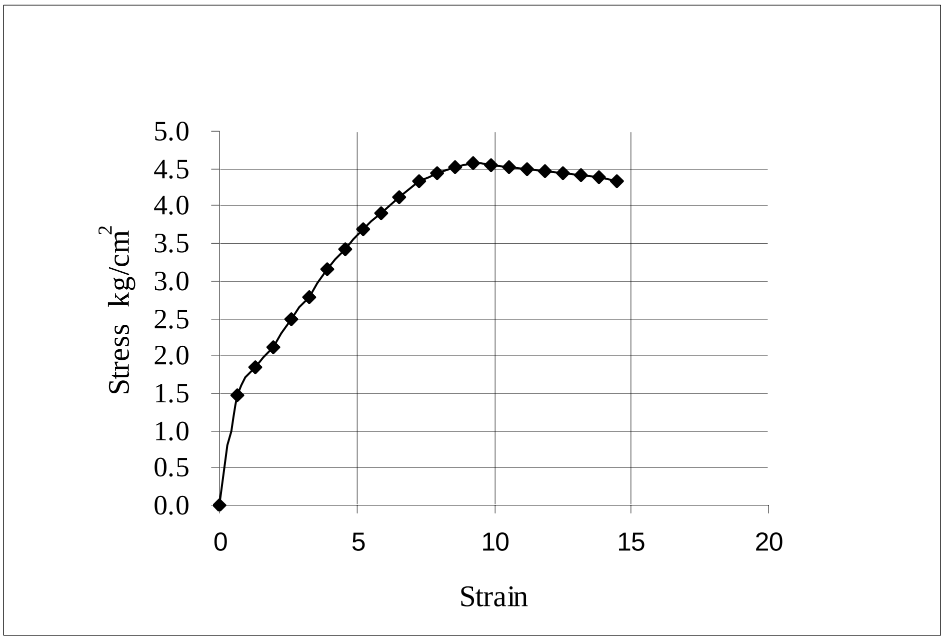

Figure (31) Unconfined compressive strength for clayey marl

The California Bearing Ratio is 4.2 and 6 at 2.5mm and 5mm penetration respectively as illustrated in Figure (32).

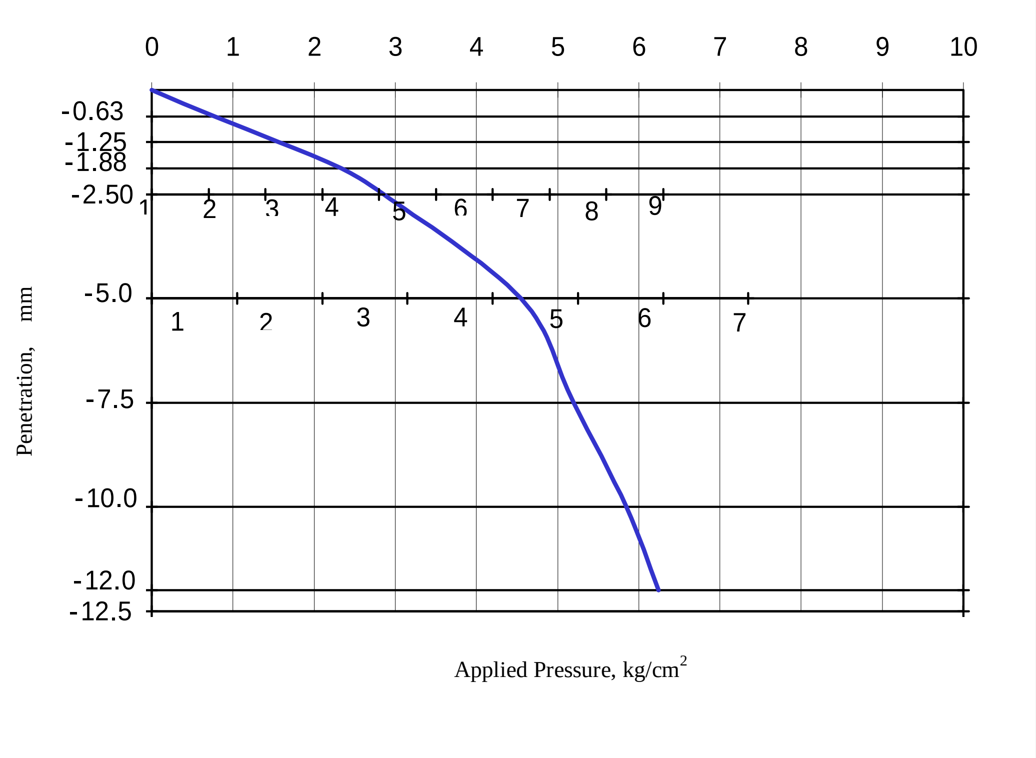

Figure (32) CBR for untreated clayey marl

The results have indicated that the soil is very weak to be used as fill material in earthworks such as embankments and subgrade material in roads and highways without improvement or stabilization. The physical properties of the marl sample are summarized in Table (37).

Table (37) Physical properties of the clayey marl sample.

Parameter	MC0
Liquid limit (LL)	42.3
Plastic limit (PL)	19.2
Shrinkage limit(SL)	14.34
Plasticity index	23.1
Passing #200%	89.55
Clay fraction %	87.58
Specific gravity	2.64
Unconfined compressive strength	4.7
Maximum dry density	1.96
California Bearing Ratio	4.5

3.5.2 Stabilization of marl

The results have shown that the consistency limits are improved as shown in Table (38).The ash-marl mixture has changed to non plastic at 20% ash content. The soil classification is changed from A-7-6 to A-4 as per AASHTO, and has changed from (CL) to (ML) at 15% ash content and to (CL) at 20% ash content. The soil has changed from medium-high plastic soil into non plastic soil.

Table (38) Summary of consistency limits for ash-marl mixtures

Sample No.	MC0	MCS-10	MCS-15	MCS-20
Ash content %	0	10	15	20
Liquid limit	42	38	34	**Non plastic**
Plastic limit	19	25	31	
Shrinkage limit	14.4	12.3	12.3	
Plasticity index	23	15	3	

The variation of consistency limits with ash content shows that both the liquid limit and plasticity index have decreased with the increase of the ash content as shown in Figure (33).

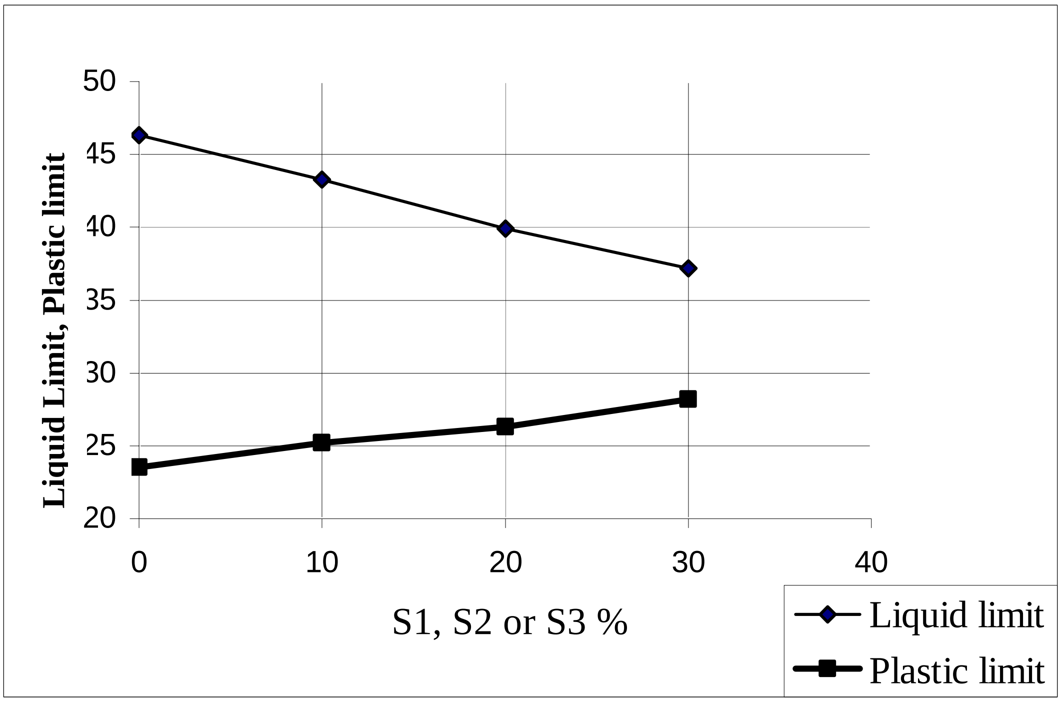

Figure (33) Variation of consistency for ash-clayey marl mixtures

Ash can absorb considerable amounts of water in its porous lattice. The liquid and plastic limits of the treated soil have decreased due to the following:

1- Dewatering of excess water from the soil into the pore spaces of ash
2- Ash is a non plastic material. Increasing the content of the non plastic ash in the plastic soil will decrease its plasticity.

Figures (34) and (35) show the soils distribution of BC0, MC0 and the various ash-brown clay and marl mixtures on the plasticity chart and on the ASTM D 3282 chart for liquid limit and index ranges for silt-clay materials.

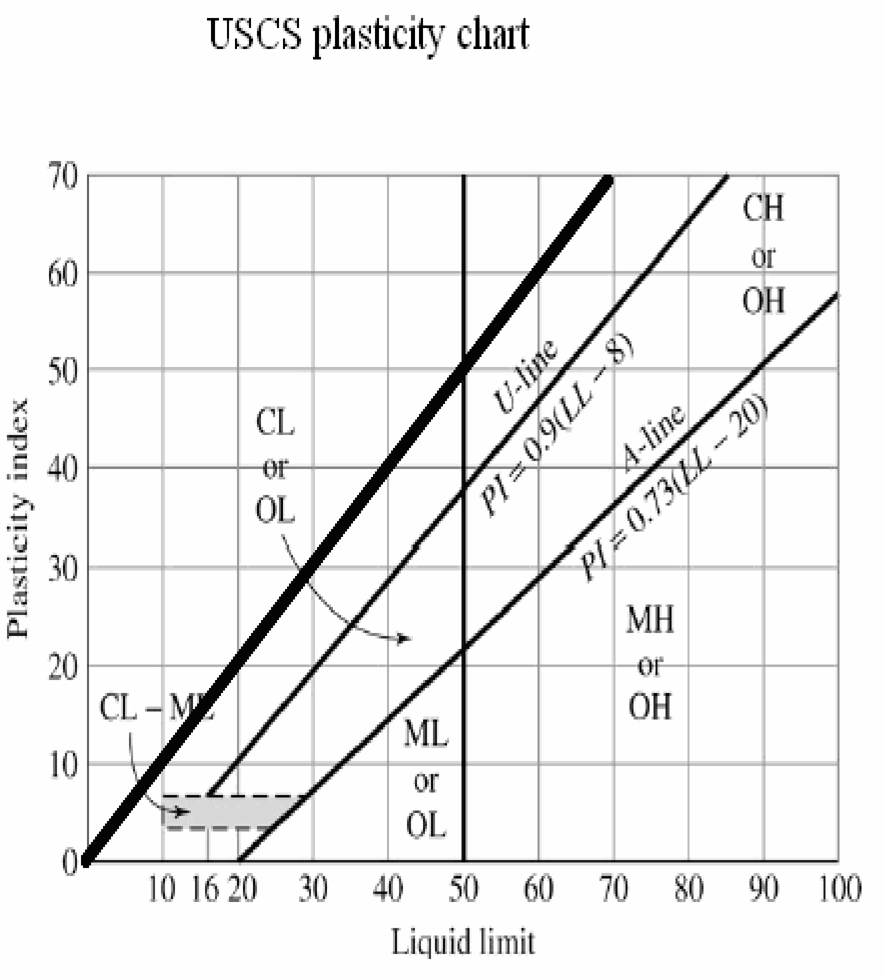

Figure (34) AASHTO soil classification

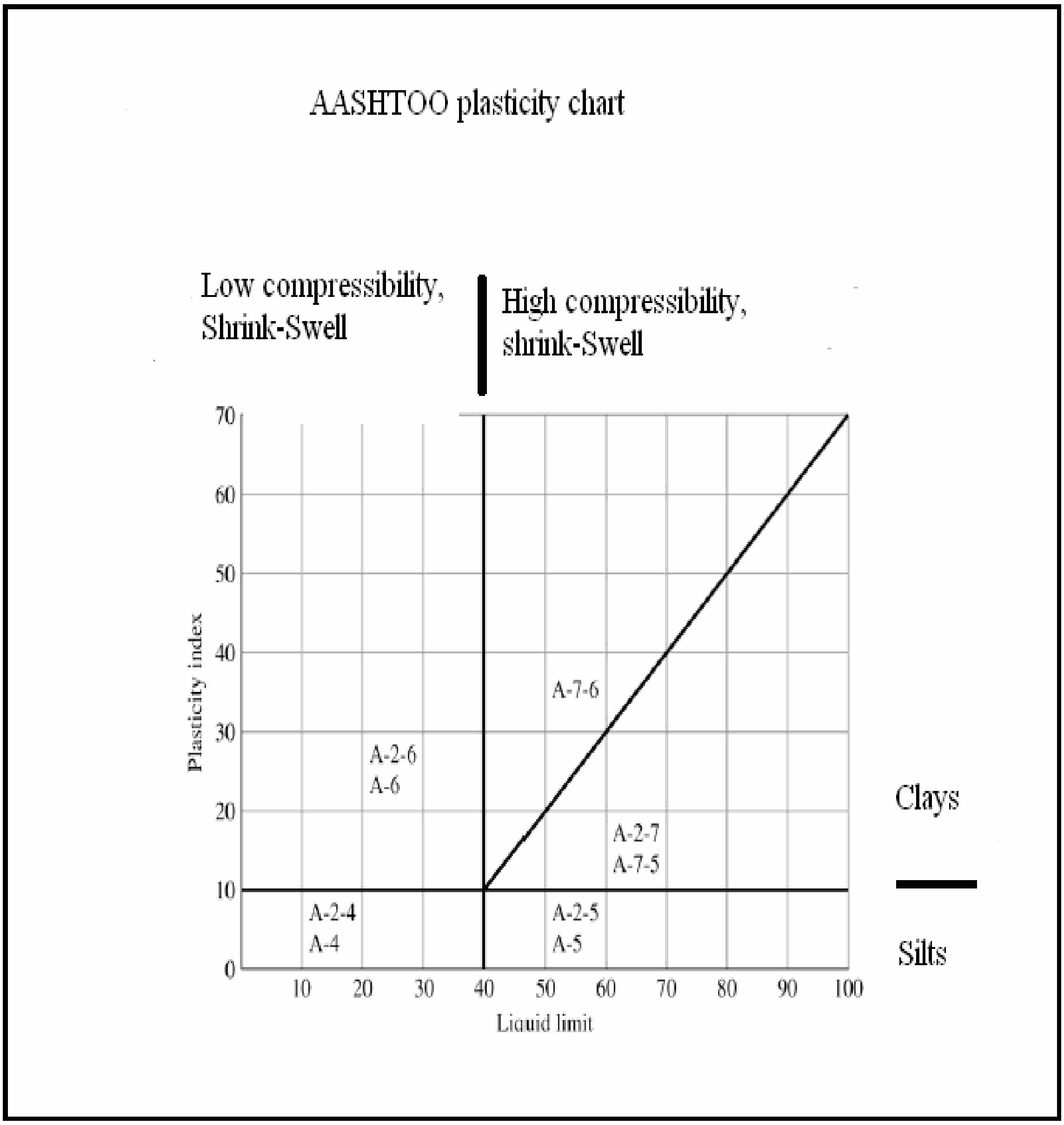

Figure (35) Unified Soil Classification System

3.5.2.1 Compaction of ash-marl

The results of the standard compaction tests carried out on four ash-marl samples named MCS-10, MCS-20, MCS-30 and MC100 have shown that the maximum dry density of the ash-marl mixtures are decreasing with increasing the ash content. The results are summarized in Table (39).

Table (39) Summary of compaction results for ash-clayey marl

Sample No.	MC0	MCS-10	MCS-15	MCS-30	MCS-50	MCS-100
Wt % of S1, S2, S3 Ash	0	10	15	30	50	100
MDD	1.96	1.91	1.88	1.85	1.57	1.45
OMC	11.7	13.33	14.53	15.17	27	29.2

The dry density of ash-marl mixtures however is decreasing with increasing the ash content due to mixing the marl sample with a lower density ash material which has a maximum dry density of 1.35g/cm$^{3.}$ The moisture content increases with the increase of ash content due to the high absorption capacity of the ash as a result of porosity.

This result is considered as an advantage in absorption of excess water from the soil and hence the ash acts as a drying agent and makes the soil more stable over a wide range of moisture content. The maximum dry density–optimum moisture variation with increasing ash content is shown in Figure (36

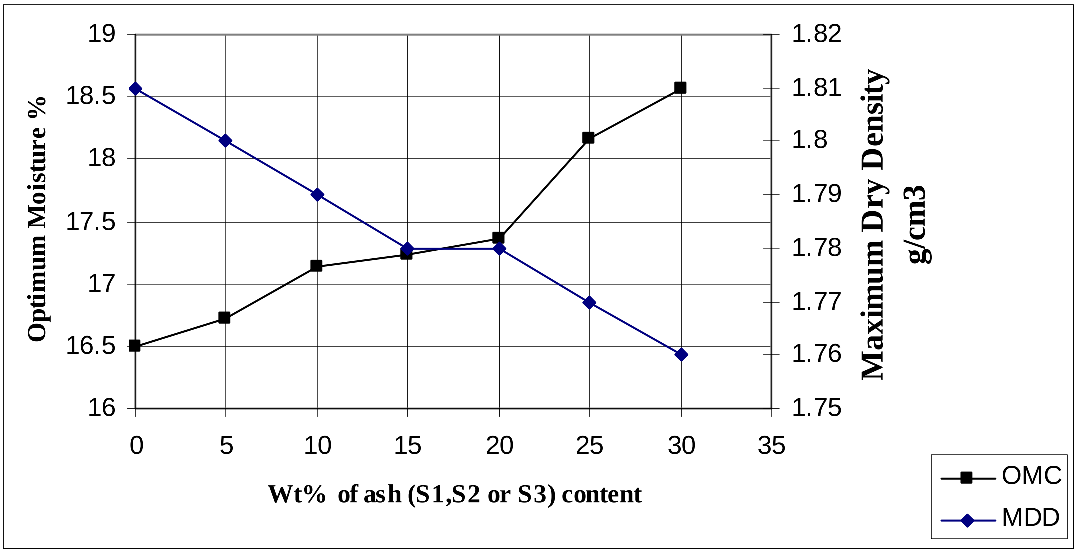

Figure (36) MDD-OMC variation with increasing ash content of ash-clayey marl

The results are considered as advantageous from the practical point of view. The change of high density problematic marl to low density stable soil can be utilized as a stable backfilling material behind high retaining walls. The slope stability problems that are encountered in marly clay areas can be solved by scarifying the upper parts of the soil, followed by mixing and pulverizing the soil to form a stable matrix with minimum cracks.

This result will help to minimize the moisture content variations from dry to rainy seasons. The slaking test results of the clayey marl samples showed no disintegration signs when the treated samples were cured for 28 days. The stabilized samples have shown an excellent integrity under complete saturation conditions for long periods (more than 20 days). The maximum dry density curves for marl with different ash content are illustrated in Figure (37).

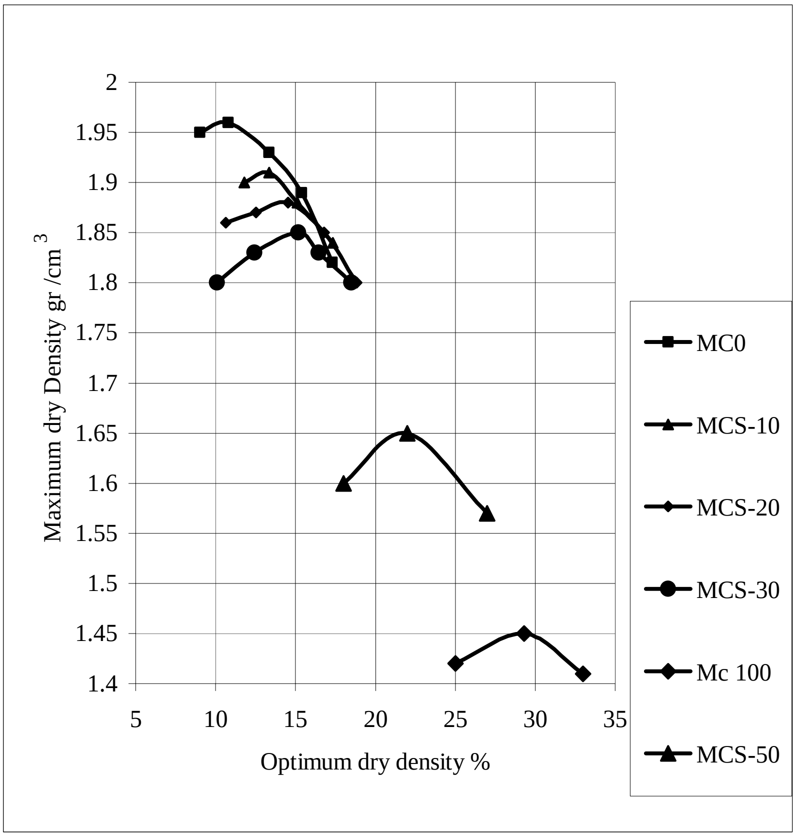

Figure (37) MDD-OMC curves of ash-clayey marl

3.5.2.2 Unconfined compressive strength

The unconfined compressive strength results are obtained for two types of ash-marl samples. The first type of samples is prepared as undisturbed samples at the maximum dry density and optimum moisture content. The disturbed samples are prepared by mixing the marl sample with 6 to 8% of moisture above the optimum due to brittleness behavior of ash-marl mixtures. The brittle texture of the soil samples is related to the over drying of the soil particles which are in close contact with ash particles. The unconfined compressive strength results are shown in Table (40). The results revealed that the unconfined compressive strength of the clayey marl is increasing with increasing the ash content and the curing period as shown in Figure (38). As in brown clay, strength buildup is related to the ash alkalis-soil pozzolanic part to form a cementaceous material that possesses higher strength parameters than the original soil sample.

Table (40) Unconfined compressive strength of clayey marl-ash mixtures

Ash type	Wt % by total wt of dry soil	Unconfined compressive strength kg/cm^2)	
		7 Days	28 Days
S1	50	6.8	14.3
S1	100	7.1	21.2
S2	50	7. 2	16.2
S2	100	8.1	25
S3	50	9.2	23
S3	100	18.8	30.1

The time factor plays an important role for the ash alkalis-soil silica reaction to continue under normal ambient temperature. Figure (38) shows that the unconfined compressive strength of ash-marl mixtures is slightly increased with increased ash content at 7 days in comparison with 28 days compressive strength results of the same samples.

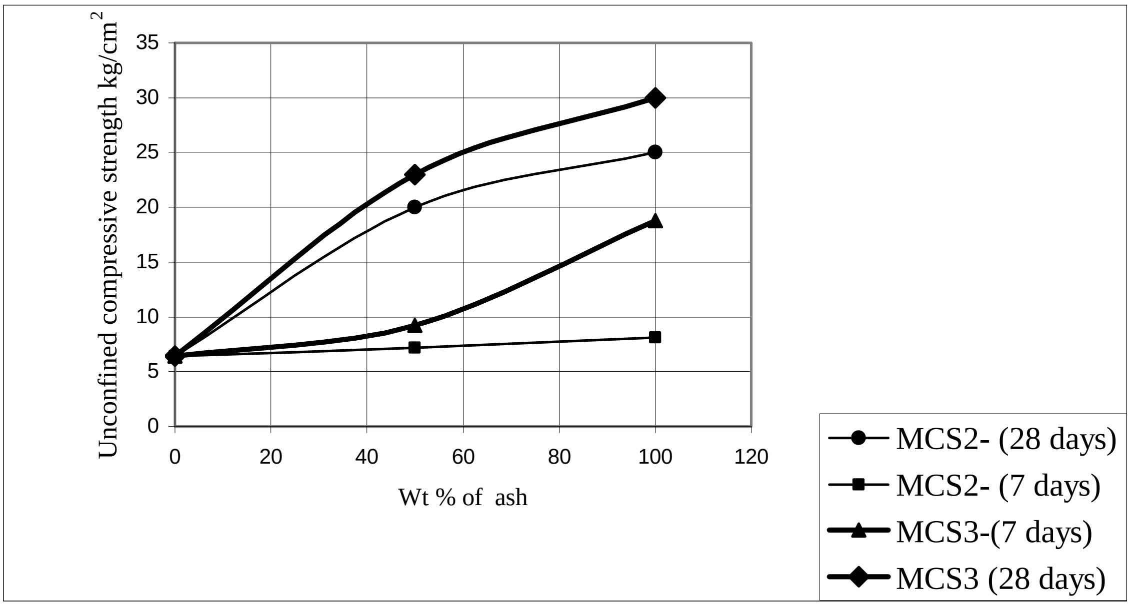

Figure (38) Unconfined compressive strength of clayey marl-ash mixtures

The alkali-pozzolanic reaction is slow and has its own characteristic which is different from Ordinary Portland Cement and other types of cementing materials. Necessary curing time must be determined when utilizing ash as a self cementaceous material in the various construction activities.

The stress-strain relationship of the various ash-marl mixtures is shown in Figure (39). However, the unconfined compressive strength of brown clay and clayey marl–ash mixtures is summarized in the following points:

1-The 7 and 28 day unconfined compressive strength at normal ambient curing conditions for the ash stabilized brown soil and marly clay samples show satisfactory results, but the ash-brown soil samples show higher strength results than the ash-clayey marly samples .

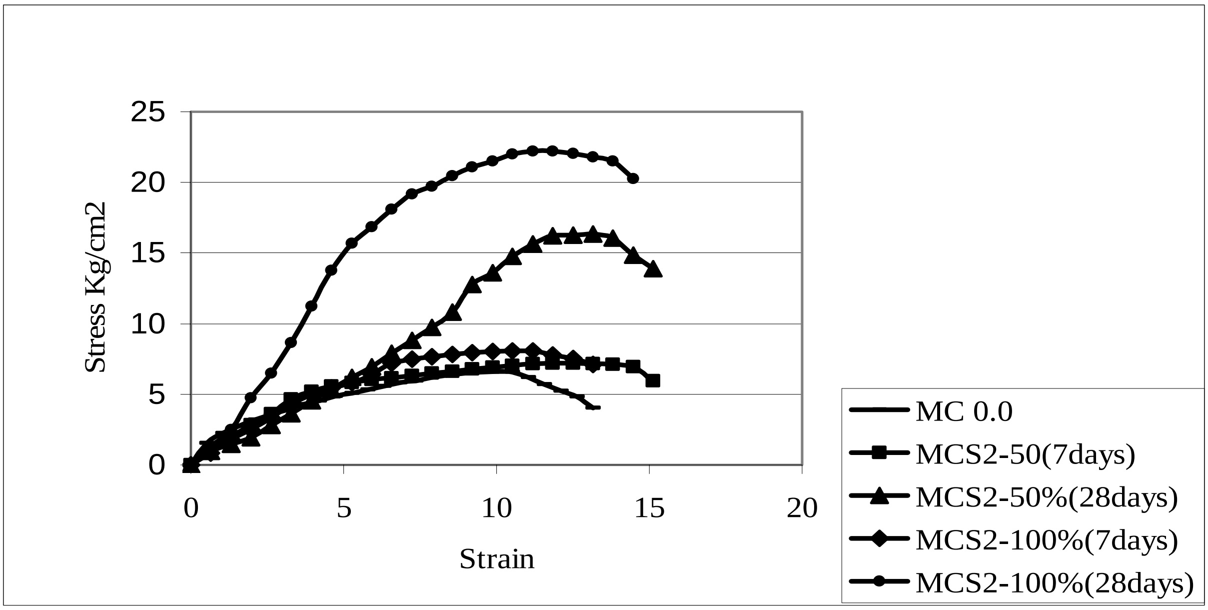

Figure (39) Stress-strain of S2 ash-clayey marl mixtures at different ages

2- All samples have continued to gain strength from 7 days to 28 days. Additional strength gain beyond 28 days is also expected.

3- The addition of ash to both brown clay and marl has increased the unconfined compressive strength.

4- The S3 ash-soil mixtures have revealed lower stiffness than the S2 soil mixtures as shown in Fig (40)

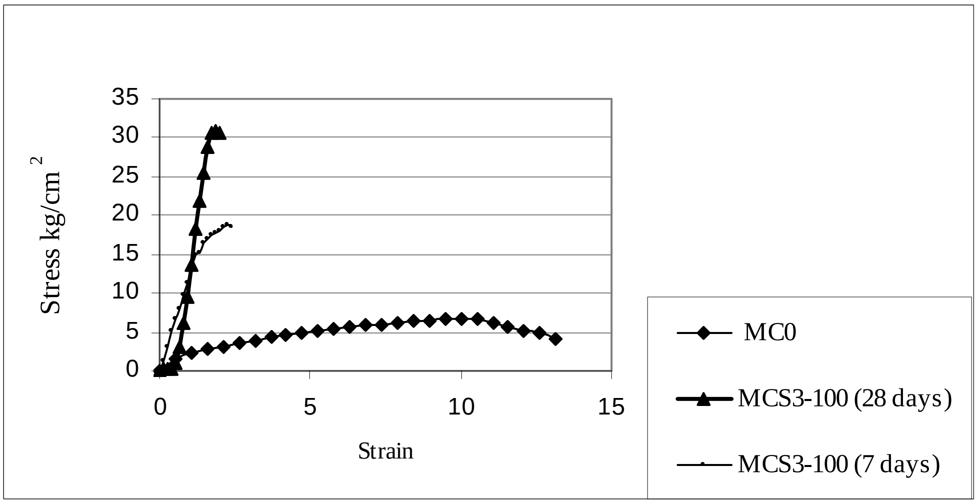

Figure (40) Stress strain of S3 ash-clayey marl mixtures

The SEM images of the MCS2-100 are shown in Figure (41). The images have shown a clear buildup of CSH and CAS in the fibrous texture which are responsible for the strength buildup in a three dimensional matrix. The density of the fibrous minerals in the MCS2-100 is less than that appeared in BCS2-100, this is related to the higher silica content in the BCS2-100 which is reflected in the CAS and CSH content in the BCS2-100 sample. CAS and CSH are indicated by XRD difractograme (Appendix XIV).

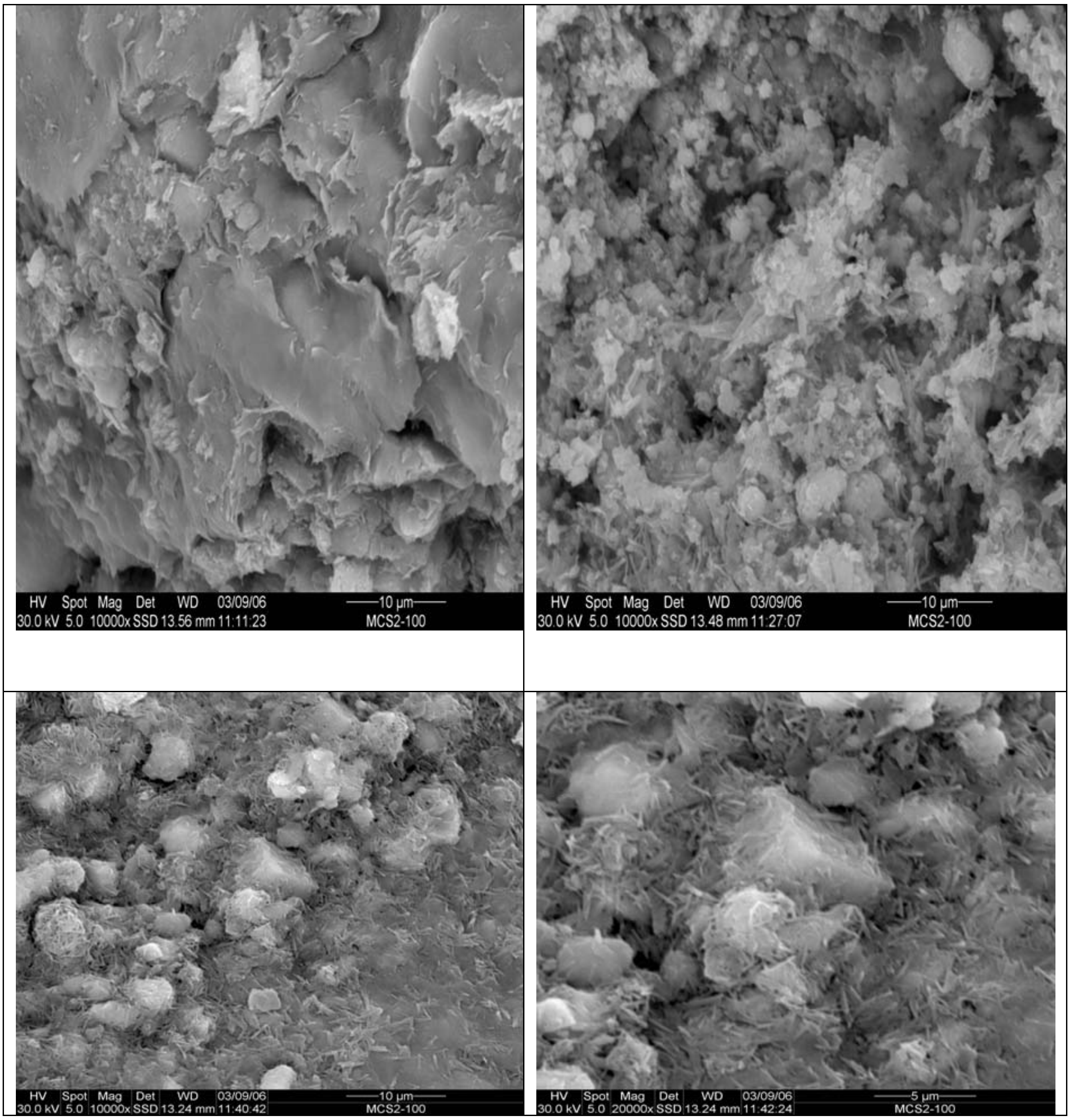

Figure (41) Scanning electron photomicrographs of clayey marl–S2 mixture showing fibrous CAS and CSH

As in the S3-brown clay soil mixtures, the strength buildup of the MCS3-100 is related to the presence of the sulfate minerals. The fibrous texture is not obvious as in the S2 ash–soil mixtures due to the very low CaO content of the S3 ash, minor CAS and CSH minerals are present while major sulfate minerals are formed as the binding matrix as shown in SEM images of Figure (42). CAS and CSH are indicated by XRD difractograme (Appendix XV).

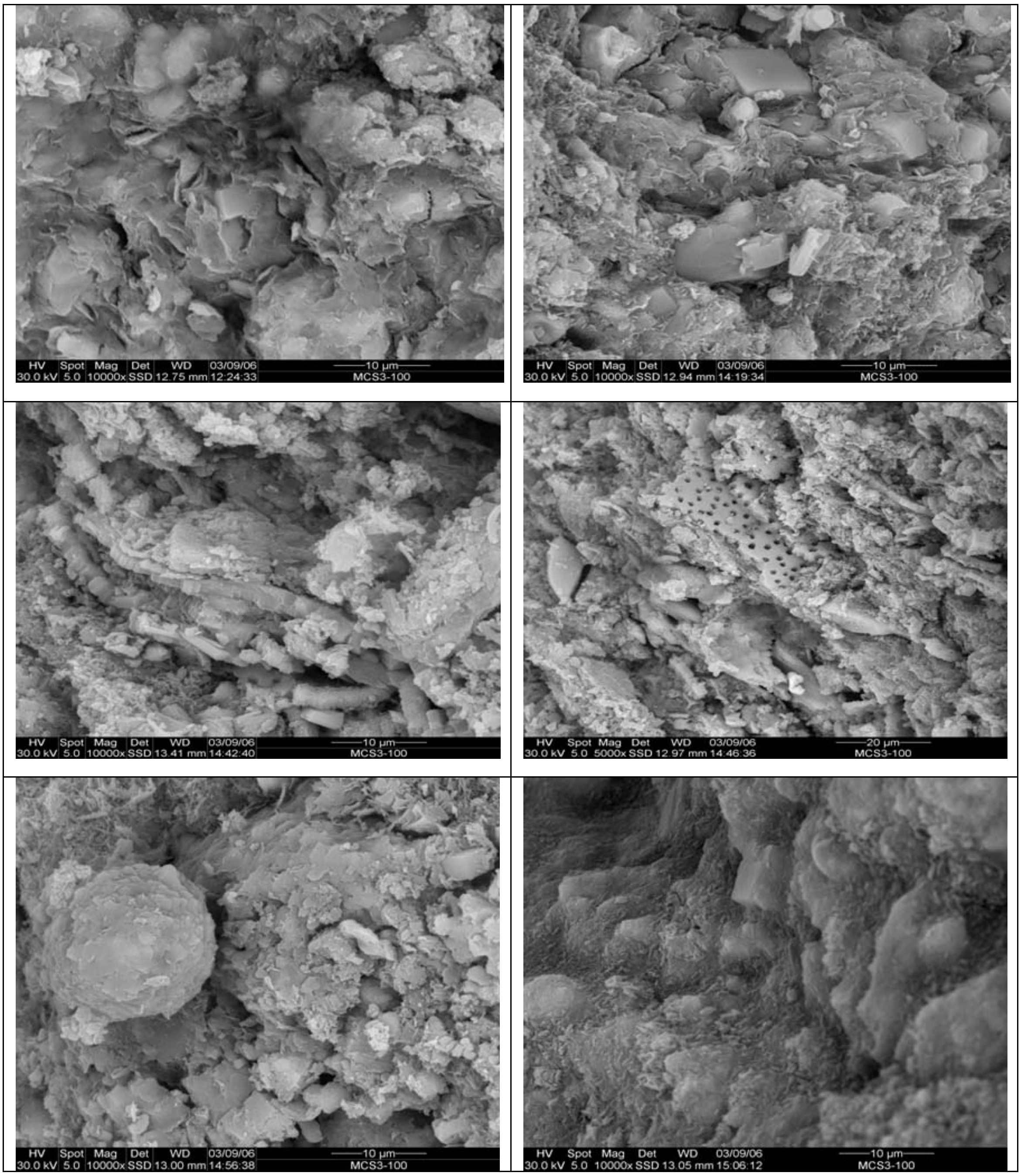

Figure (42) Scanning electron photomicrographs of clayey marl–S3 mixture showing platy ettringite and short fibrous CASH and CSH phases

3.5.2.3 California Bearing Ratio

The California Bearing Ratio test results of clayey marl-ash mixtures are shown in Table (41). The results that are obtained at 28 days under soaking conditions for 96 hours directly before testing show that CBR is increasing while increasing the ash content and curing period as it is revealed in Figure (43). CBR specimens are cured for 28 days at normal ambient temperature in summer and then soaked under water for 96 hours. Both brown soil and clayey marl have shown that the ash has improved the strength of the stabilized soil but with higher CBR values for brown clay samples when compared with the clayey marl samples that have the same ash content.

Table (41) CBR values of clayey marl-ash mixtures

Sample	Wt % by total weight of dry soil	Age Days	CBR %
MC0	0	28	5
MCS1-50	50	28	82
MCS1-100	100	28	122
MCS2-50	50	28	85
MCS2-100	100	28	125
MCS3- 50	50	28	80
MCS3-100	100	28	120

The difference in strength between brown soil and clayey marl samples is due to the different silica content, soil texture and grain size distribution of the parent soil samples.

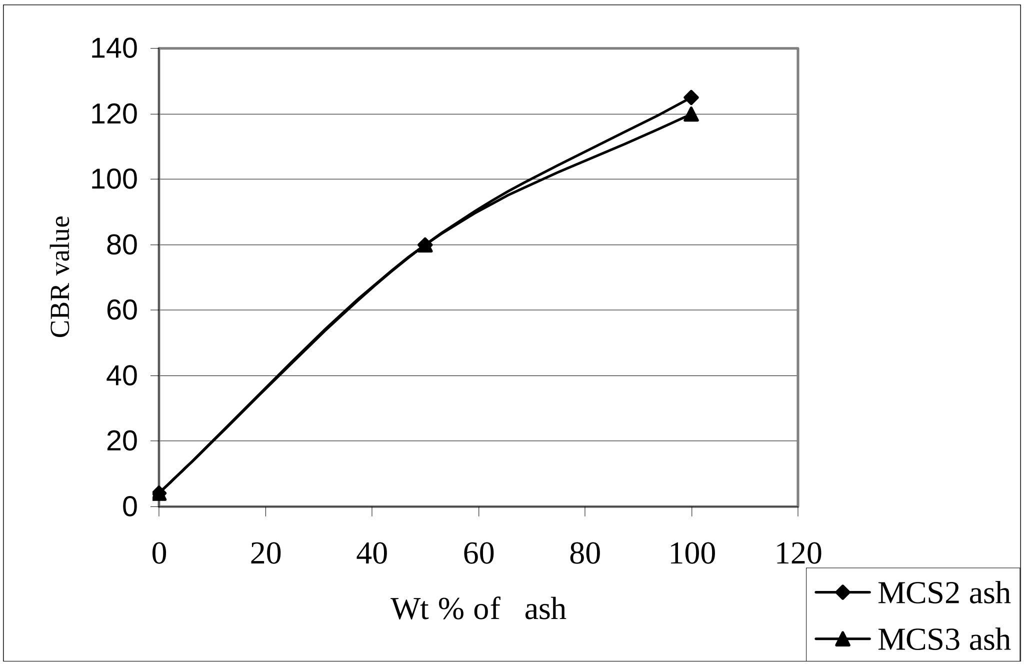

Figure (43) California Bearing Ratio for clayey marl-ash mixtures

3.5.2.4 Consolidation swelling behavior

The collapse potential results are given in Table (42). The collapse potential has decreased to 0.01 mm for the tested clayey marl sample with 15% of ash content. The collapse potential has decreased to zero when the ash content is adjusted to 30%. This is related to the increased cementaceous matrix that increased the soil stability against the internal swelling forces which is acting on the opposite direction of the internal bonding forces.

It is clear that increasing the ash content beyond 30% has no effect on the consolidation behavior but has significant effect on the other strength parameters.

Table (42) Collapse potential of S1, S2, S3 ash-clayey marl mixtures.

Ash % (by weight of dry soil)	Collapse potential	
	7days	28 days
0	3.9	2.8
15	0.04	0.01
30	0.04	0.0
50	0.003	0.0

The decrease of the collapse potential at 7 and 28 days while increasing the ash content is shown in Figure (44)

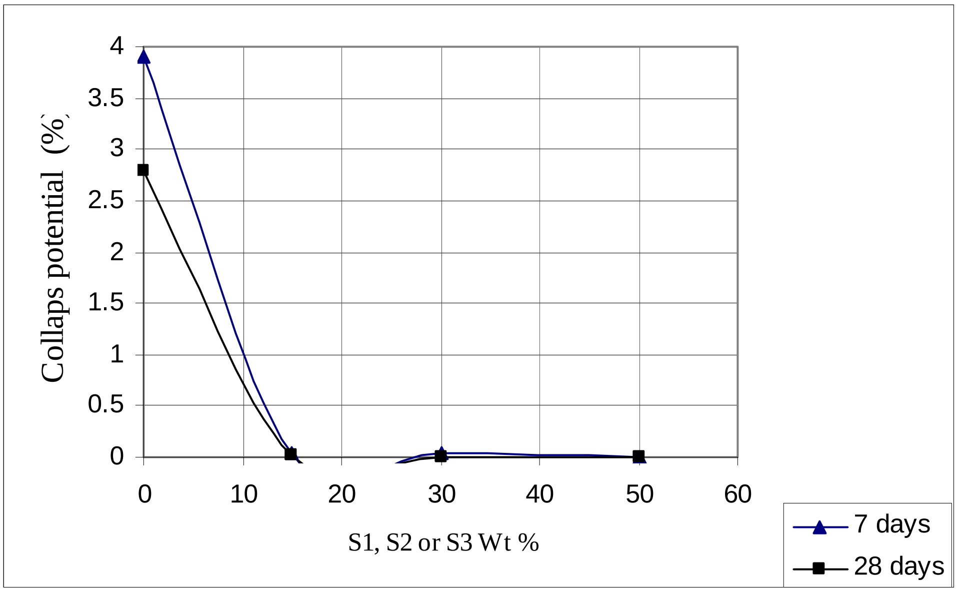

Figure (44) Collapse potential of clayey marl-S1, S2 ash mixtures

3.6 Utilization of ash as Flowable fills

Natural soil borrows, granular fill, and other embankment or structural fill materials are typically tested to determine their shear strength for structural fill purposes. The general strength requirements for different construction uses as indicated by Brendal et al., 1997 and the unconfined compressive strength results after 7 and 28 days of the K2 designated samples that are prepared utilizing the S2 ash, with water/ash mixing ratio of 70%. The flowable and structural fill requirements are summarized in Table (43).

Table (43) Flowable and structural fill requirements. (after Brendal et al, 1997).

Compressive strength	Flowable structural fill	Compacted Flowable fill	Road base	Road base	K2 flowable mix
7 days kg/cm^2	--	--	>28	>35	10
28 days kg/cm^2	3.5- 10.5	>70	>42		17

The compressive strength results have shown that the S1, S2 and S3 ash can be utilized as a low cost self cementing material instead of the Ordinary Portland Cement or the other synthesized chemicals for soil stabilization purposes. Table (44) summarizes the physical and mechanical properties of the stabilized ash-soil mixtures.

Table (44) Summary of physical and mechanical properties of the stabilized soil

Property of ash-soil mixtures	Description of the stabilized soil
Plasticity	The plasticity index decreases, with increasing ash content in all circumstances. This is due to decreasing the liquid limit and the increasing of the plastic limit
Moisture density relationship	The result of immediate reactions between ash and the clay soil causes substantial change in the moisture density relationship. The moisture density changes reflect the new nature of the soil and are evidence of the physical property changes occurring in the soil upon lime treatment.
Swell potential	Soil swell potential and swelling pressures are normally significantly reduced by ash treatment.
Drying	Ash aids the immediate drying of wet clay soils. This allows compaction to proceed more quickly.
Strength properties	Both the Unconfined Compressive Strength (UCS) and CBR increase considerably with the addition of ash. These values can be increased by increasing the curing time. Experience has shown increases of CBR from 4 up to 150 with ash only. This gain in strength is often used in the design of pavement in order to reduce the depth of pavement material required.
Water resistance	The ash stabilized soil forms a water resistant barrier by impeding the soil disintegration under saturation or semi saturation conditions. Treatment with high ash content is required to achieve long-term stabilized soil.
Flowability	Ash can be used as a flowable fill without compaction.

3.7 Results of utilizing S1, S2 and S3 in concrete

The physical properties of coarse and fine aggregates that are used in the reference mix (T0) and other ash–concrete mixes are shown in Table (45).

Table (45) Physical properties of aggregate

Sample (Aggregate)	Specific gravity GS	Absorption %	Pass # 200 sieve%	Abrasion %	Clay lumps &friable particles
19mm aggregate	2.57	1.3	0	22	0
Wadi sand	2.55	1.5	1.8	-----	0
Glass sand	2.53	1.8	4	----	0
Ground tuff	2.6	4	50	-----	0
Sweileh sand	2.54	1.95	5.5	----	0

The fraction passing #200 sieve is very low. Wadi sand is free from clay lumps and friable particles. Sieve analyses results are summarized in Table (46)

Table (46) Grain size analysis of coarse and fine aggregates

Sieve number	19 mm aggregate pass %	Wadi sand pass %	Glass sand pass %
3/4	100	100	100
3/8	47.8	100	100
4	0.8	69.6	99
8	0.0	41.7	96.5
16	0.0	25.9	94.7
30	0.0	14.6	89.7
50	0.0	8.1	48.7
100	0.0	4.5	13.2
200	0.0	0.5	1.5

Coarse aggregates have revealed that the aggregates are of low abrasion value and low absorption. The glass sand has a very low fraction passing #200 sieve. All ingredients are accepted for concrete production. Definite proportions of 19 mm, wadi sand, and glass sand are used. The mix proportions were 50% 19 mm coarse aggregate, 30% wadi sand and 20% of glass sand. These proportions are obtained from the sieve analysis results to get a well graded combined aggregate mixture. The same proportions of the same ingredients were used in all concrete trial mixes that were labeled as T0 (ash free), T1-10, T1-20, T1-30, T1-40 and T1-50 where S1 ash was used. The S2 ash was used in the trial mixes labeled as T2-10, T2-20 and T2-30. The grain size distribution of coarse and fine aggregates is shown in Figure (45).

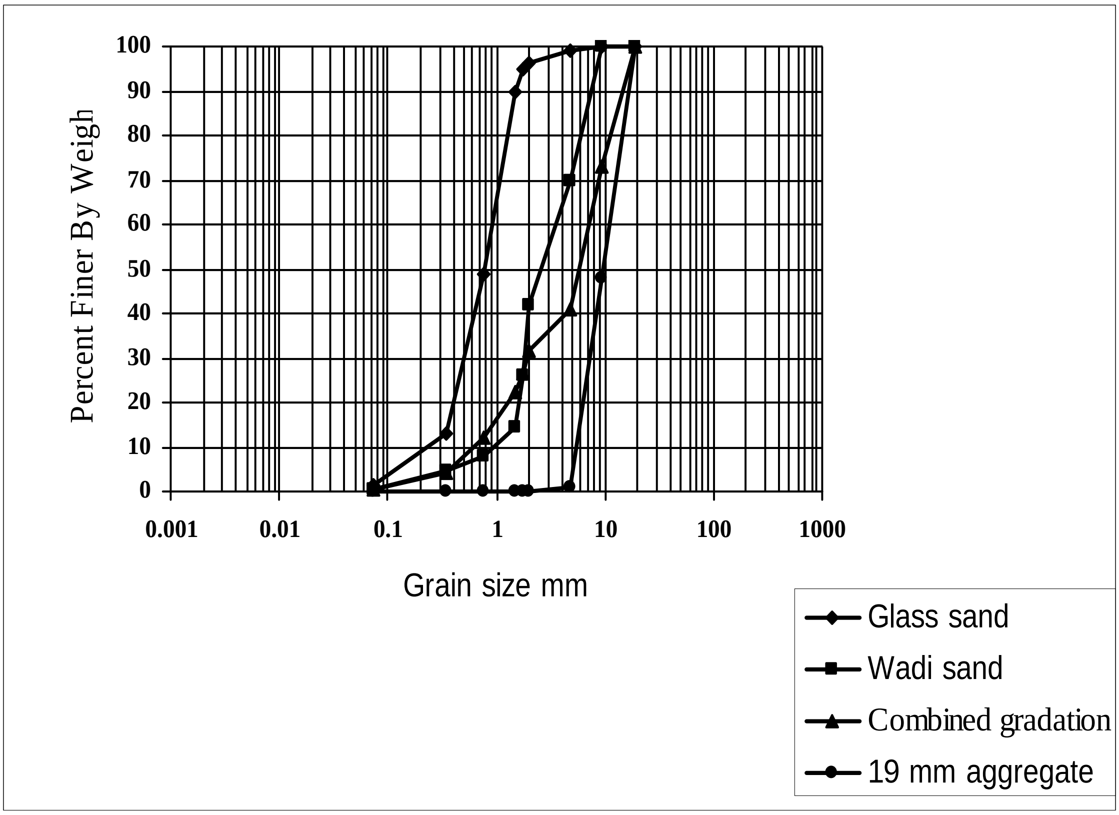

Figure (45) Grain size distribution of course aggregate, wadi and glass sand

The compressive strength at 7, 28 and 56 days and density of the (T0) trial mix samples are included in Table (47). The compressive strength results for the (T0) samples have shown a normal strength buildup with time at the proposed cement content and water /cement ratio.

Table (47) Compressive strength results of the reference mix T0

Age Days	7 days compressive strength kg/cm^2			28 days compressive strength kg/cm^2			56 days compressive strength kg/cm^2		
Sample number	1	2	3	1	2	3	1	2	3
Strength	279	231	252	354	359	346	361	380	391
Average Strength	254			352			377		
Average density kg/m^3	2385			2445			2358		

The compressive strength results of the T0 mix has shown the normal behavior of normal concrete mixes in which OPC is used.

3.7.1 Results of concrete trial mixes with S1 ash

The total water increases with the increase of ash content to get the same workability of the reference mix as shown in Figure (46).

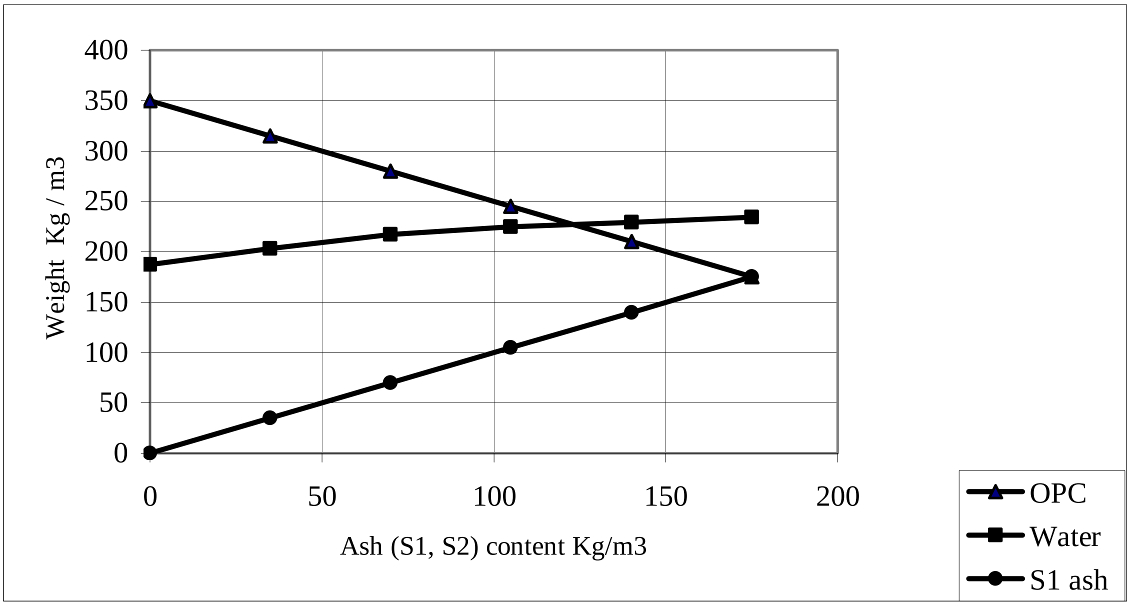

Figure (46) Increase of mixing water with increasing ash content

The increased demand of water while increasing the ash content is related to the increasing CaO content which is usually characterized by high porosity and hence a high absorption capacity. The S1 concrete mixes are completely different from class F ash which is distinguished by very low CaO content and a high pozzolanic content (SiO_2+ Al_2O_3+ Fe_2O_3).

The compressive strength results for S1 ash concrete mixes T1-10, T1-20, T1-30, T1-40, and T1-50 are shown in Table (48). The results have indicated that the compressive strength of the tested samples decrease at 7, 28 and 56 days as a result of increasing the ash content. Since the physical properties of ash depend on its chemical constituents, it is not possible to use some types of ash as a substitute of OPC as S2 which contains a high percent of lime. Such a material will adversely affect the produced concrete quality. This is clear with the comparison of the compressive strength values of the reference mix samples that contains OPC only as it is indicated in Figure (47). This reveals that the OPC paste has higher strength characteristics when compared with the self cementaceous ash.

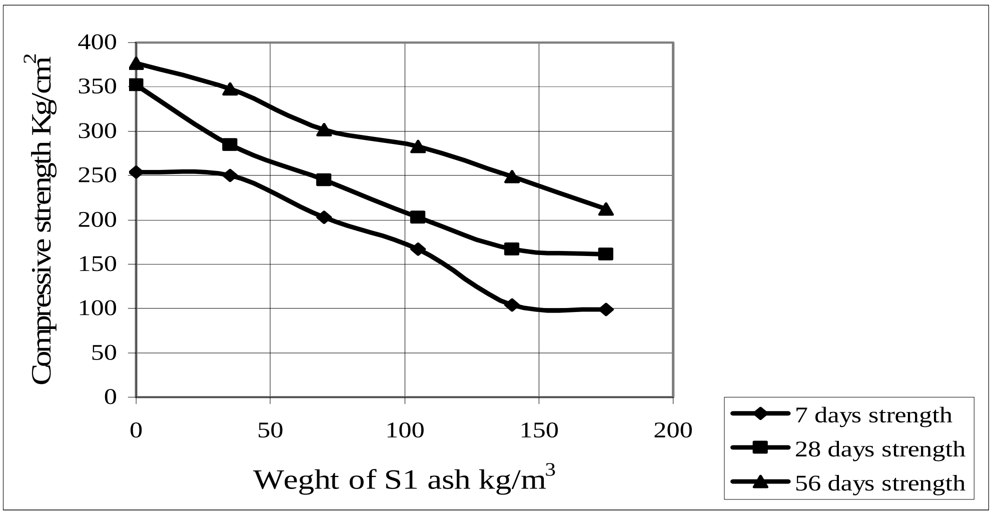

Figure (47) Compressive strength of S1 ash-concrete mixes

Table (48) Summary results of T1 with various S1 ash-concrete trial mixes

Mix No	Ash Weight kg/m^3	Cement Weight kg/m^3	Water liters kg/m^3	7 days strength kg/cm^2			Average	28 days strength kg/cm^2			Average	56 days strength kg/cm^2			Average
				Cube 1	Cube 2	Cube 3		Cube 1	Cube 2	Cube 3		Cube 1	Cube 2	Cube 3	
T0	0	350	187	279	231	252	254	359	346	350	352	361	380	391	377
T1-10	35	315	203	218	288	245	250	286	279	290	285	358	342	346	348
T1-20	70	280	217	200	195	214	203	250	248	237	245	298	302	316	302
T1-30	105	245	225	163	173	165	167	206	198	205	203	288	288	270	283
T1-40	140	210	229	102	108	103	104	163	179	158	167	241	257	250	249
T1-50	175	175	234	96	101	101	99	169	164	151	161	184	227	227	212

The compressive strength results for S2 ash concrete mixes T2-10, T2-20 and T2-30 are shown in Table (49). The results have indicated that the compressive strength of the tested samples decrease at 7, 28 and 56 days as a result of increased the ash content. The reduction of the compressive strength of concrete is higher when the S2 ash is used in the same percent of S1 in the trial mixes. This is related to the higher alkalinity of the S2 ash which drives the deleterious cement-alkali reaction that reduces the concrete strength. Higher water demand of the S2 concrete mixture to get the same workability of the S1 mixes of the same ash content was recorded. The higher water demand is related to the high S2 concrete temperature as a result of the exothermic behavior of the S2. As a result, the water/cement ratio is higher in the S2 mixes and hence the compressive strength decreases.

The compressive strength at 7, 28 and 56 days of the S2 ash-concrete samples has decreased as a result of increasing the ash content as shown in Figure (48)

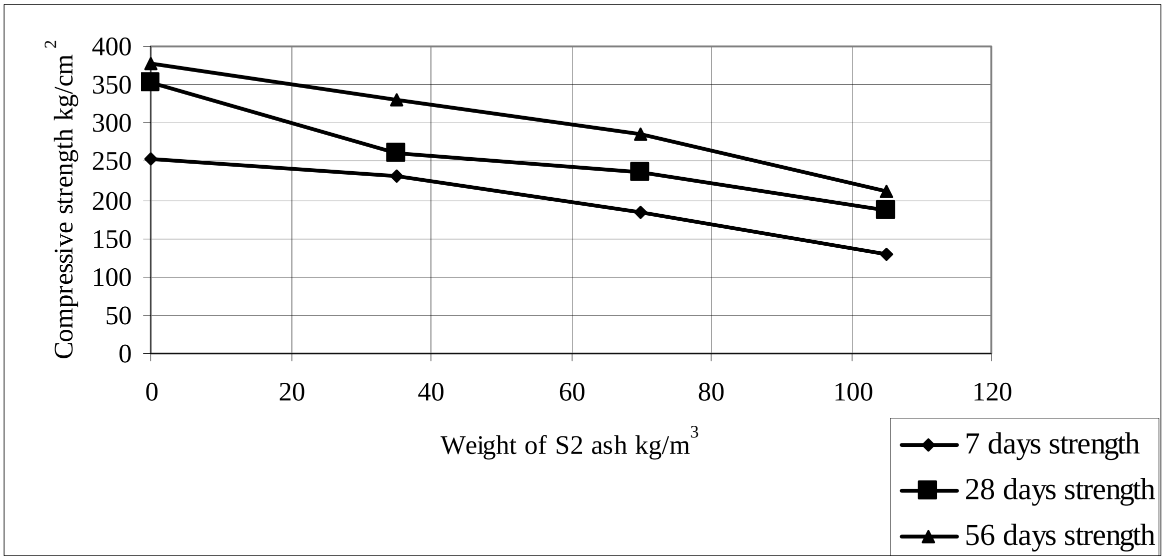

Figure (48) Compressive strength of S2 concrete mixes

Table (49) Compressive strength results of T2 trials with various S2 ash content.

	Ash Weight	Cement Weight	Water liters	7 days strength kg/cm^2			Average	28 days strength kg/cm^2			Average	56 days strength kg/cm^2			Average
	kg/m^3	kg/m^3	kg/m^3												
Mix No				Cube 1	Cube 2	Cube 3		Cube 1	Cube 2	Cube 3		Cube 1	Cube 2	Cube 3	
T0	0	350	200	279	231	252	254	359	346	350	352	361	380	391	377
T2-10	35	315	210	238	227	225	230	253	266	260	260	345	3 20	3 30	330
T2-20	70	280	222	177	190	188	185	245	232	228	235	277	278	300	285
T2-30	105	245	235	127	124	139	130	174	198	186	186	196	225	212	211

The compressive strength results decrease with the increase of the S1 or the S2 ash content. This is related to three main factors:

1. The very high strength of OPC paste when compared with the compressive strength of the used S1 or S2 ash paste strength as a binding material.
2. The progressive increase of the mixing water while increasing the ash content will increase the mix water cement ratio.
3. Extra added water to the mixes with increasing the S2 ash content to obtain the same slump and same workability as in S2 mixes is due to the exothermic behavior of the S2 ash and the high concrete temperature which will cause rapid setting time and less workability. The mentioned factors have led to lower compressive strength of the T2 mixes with reference to the T1 mixes at the same ash content of S1 or S2 and age as shown in Figure (49). The evaporation factor should be taken into consideration in S2 mixes. The water/cement ratio (w/c) is considered as a primary factor in strength development of the OPC concrete mixes or similar cementaceous materials.

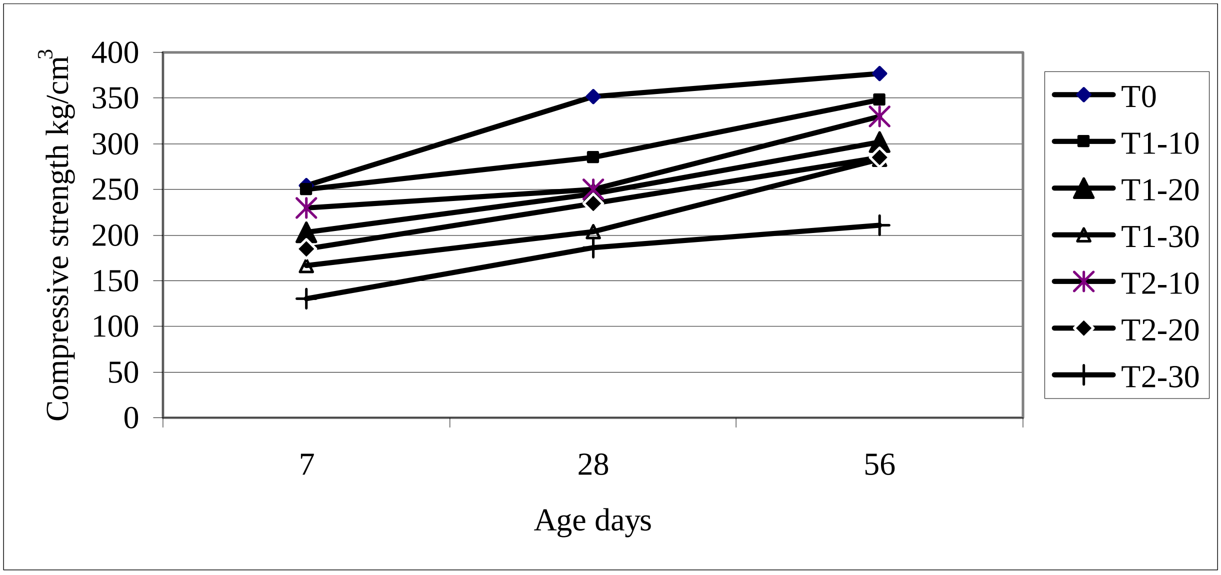

Figure (49) Compressive strength variation of T1, T2 concrete mixes

3.7.2 Results concerning ash-aggregate mixtures

Compressive strength results are determined for ash aggregate mix (AG) at 7 and 28 days. The results are summarized in Table (50). In this trial, ash is used to completely substitutes the OPC in the production of normal concrete which can be utilized in some construction aspects as blinding concrete, under floor tiles, side walks and other construction aspects in which a compressive strength of 70 kg/cm^2 is considered satisfactory.

Table (50) Compressive strength results for S2 ash aggregate mix

Sample No	7days strength kg/cm^2	28 days strength kg/cm^2	Density kg/m^3
AG1	21.5	73.1	2.31
AG2	24.2	71.7	2.29
AG3	22.5	69.4	2.33

3.8 Self cementing mortars

Ash, tuff, polystyrene, glass sand, OPC, Swieleh sand(Kurnub Sandstone) are used in various mortar mixes as ash only, ash and polystyrene, ash and tuff, ash and (glass sand and OPC), ash and Swieleh sand. The physical properties for the used raw materials are given in Table (51).

Table (51) Physical properties of tuff, glass sand, and Sweileh sand

Material	Specific gravity	Absorption %	Passing #200 sieve %
Tuff	2.6	4.0	50
Glass sand	2.56	1.8	4
Sweileh sand	2.54	1.8	5.5
Polyester	0.002	0.0	0.0

3.8.1 Ash-glass sand mortars

The compressive strength results of ash glass sand mortars have shown that the compressive strength at 7 days is low when compared with the 28 days results as shown in Table (52). This is related to the longer curing time which gave the opportunity to the ash alkalis to react with the silica to form extra cementaceous matrix.

Table (52) S1, S2, S3 ash-glass sand mortar results

Mix designation	Glass sand (gr)	Ash (gr)	Water (gr)	Density gr/cm^3	7 days strength kg/cm^2	28 days strength kg/cm^2	56 days strength kg/cm^2
S1-1	1375	500	540	I.87	11.4	42.8	44.4
S1-1	1375	500	540	1.88	10.3	39.7	45.7
S1-1	1375	500	540	1.86	11.9	40.9	47.5
S2-1	1375	500	560	1.87	20.4	55.2	68.3
S2-1	1375	500	560	1.88	18.6	53.9	64.4
S2-1	1375	500	560	1.86	19.2	56.8	65.9
S3-1	1375	500	420	1.76	7.3	18.6	19.2
S3-1	1375	500	420	1. 75	6.5	17.8	18
S3-1	1375	500	420	1. 72	6.7	19.1	20.4

The strength increment from 28 days to 56 days is considered minor, this is due to the continuous decreasing in the free CaO content in the mix and hence the parallel decrease in the pH of the mixture which is considered an important factor that controls the rate and intensity of the ash-alkalis-silica reaction.

The compressive strength results of the S3 ash mixtures at a constant glass sand and S2 ash content (S1+S2 = 500 g) are shown in Table (53).

Table (53) Compressive strength results of S3-S2 - glass sand mixtures

Mix designation	S3 Weight (gr)	S2 weight (gr)	Glass sand weight (gr)	Density (gr/cm^3)	7 days Comp. strength kg/cm^2	28 days comp. Strength kg/cm^2	56 days comp. Strength kg/cm^2
S3-A	250	250	1375	1.92	7.6	35.4	42.2
S3-B	334	166	1375	1.87	6.2	31.1	38.1
S3-C	1375	-	500	1.73	21.5	76.2	92.2

The compressive strength has increased by increasing the S2 content in the standard mixes (500gr ash: 1375, glass sand).

SEM images of the S3-C mixture are shown in Figure (50). The fibrous texture of the binding matrix is clear. Ettringite and other CSH and CASH minerals are expected to be responsible for the binding strength. The XRD results of the S3-C given in (Appendix XVI) have confirmed the SEM results

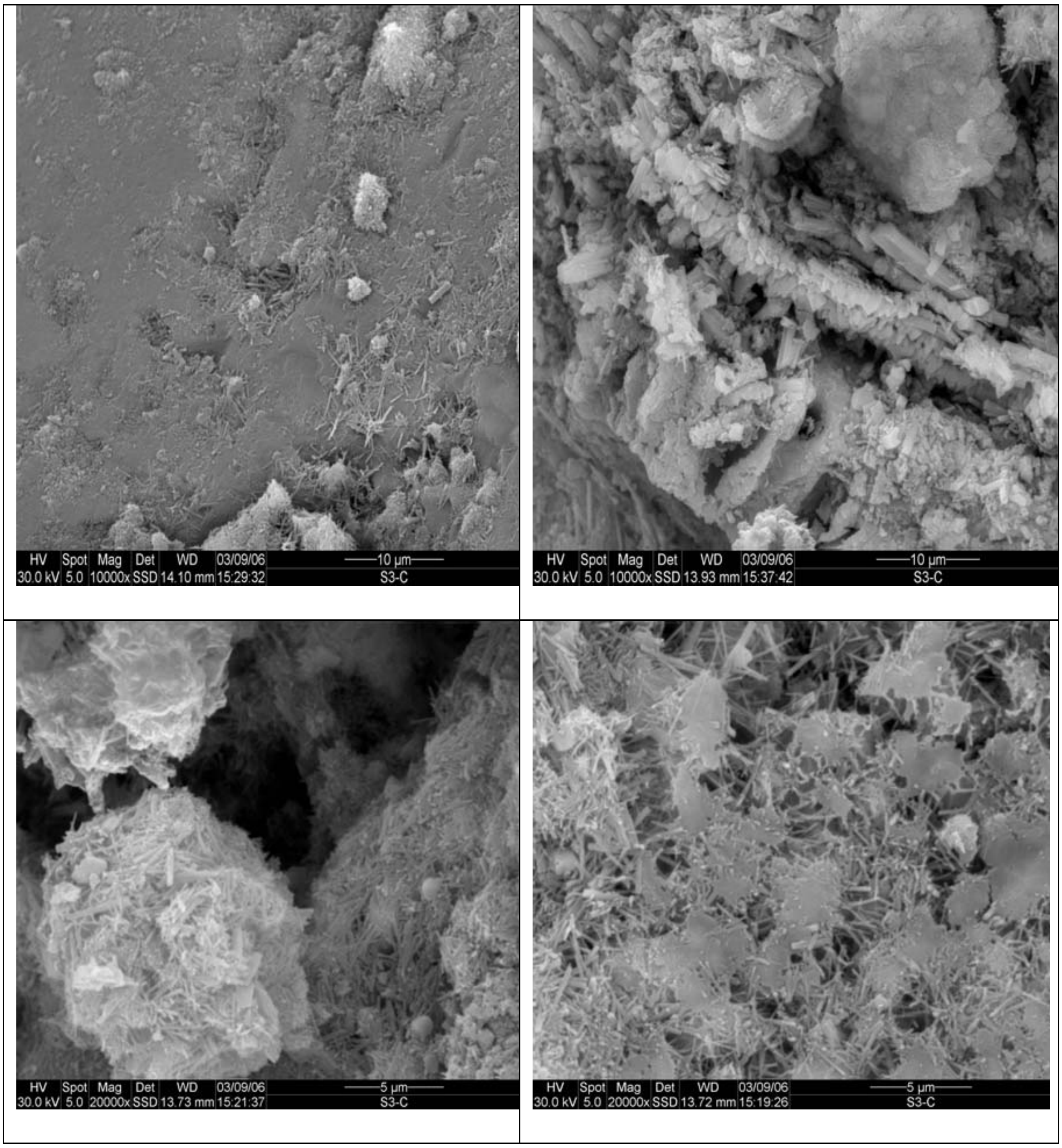

Figure (50) Scanning electron photomicrographs of glass sand-S3 ash showing fibrous growth of ettringite, CASH and CHS phases

3.8.2 Ash-tuff mortars

The chemical composition of the tuff sample shows that the pozzolanic part (SiO_2 +Al_2O_3+Fe_2O_3) is 77% by weight of the tuff sample is indicated in Table (54).

Table (54) Chemical composition of tuff *

Oxide	Weight %
SiO_2	50.6
Al_2O_3	15.2
Fe_2O_3	11.2
CaO	9.0
MgO	5.8
TiO_2	2.5
K_2O	1.2
P_2O_5	4.6

* ICP technique

The compressive strength results for mortars made of S1 and S2 ash mixed with different ratios of ground tuff are summarized in Table (55).

Table (55) Compressive strength of S1, and S2 ash-Tuff mortars

Mix designation	Tuff sand (gr)	Ash type	Ash Wt(gr)	Density gr/cm^3	7 days strength kg/cm^2	28 days strength kg/cm^2	56 days strength kg/cm^2
S1T-200	500	S1	1000	1.59	17.8	56.8	60.4
S1T-200	500	S1	1000	1.70	13.6	54.1	57.7
S1T-200	500	S1	1000	1.75	16.9	55.9	59.4
S2-T-200	500	S2	1000	1.85	19	57	63
S2-T-200	500	S2	1000	1.86	20.5	58	62.5
S2-T-200	500	S2	1000	1.85	18	59.3	61
S2-T-100	500	S2	500	1.87	21.2	60.5	69.4
S2-T-100	500	S2	500	1.86	22	59.2	72.1
S2-T-100	500	S2	500	1.85	24.1	63	71

The compressive strength results have shown an ascending trend for both S1 ash-tuff and S1 ash-glass sand mortars, but the ash tuff mortar reveals higher strength results as indicated in Figure (51).

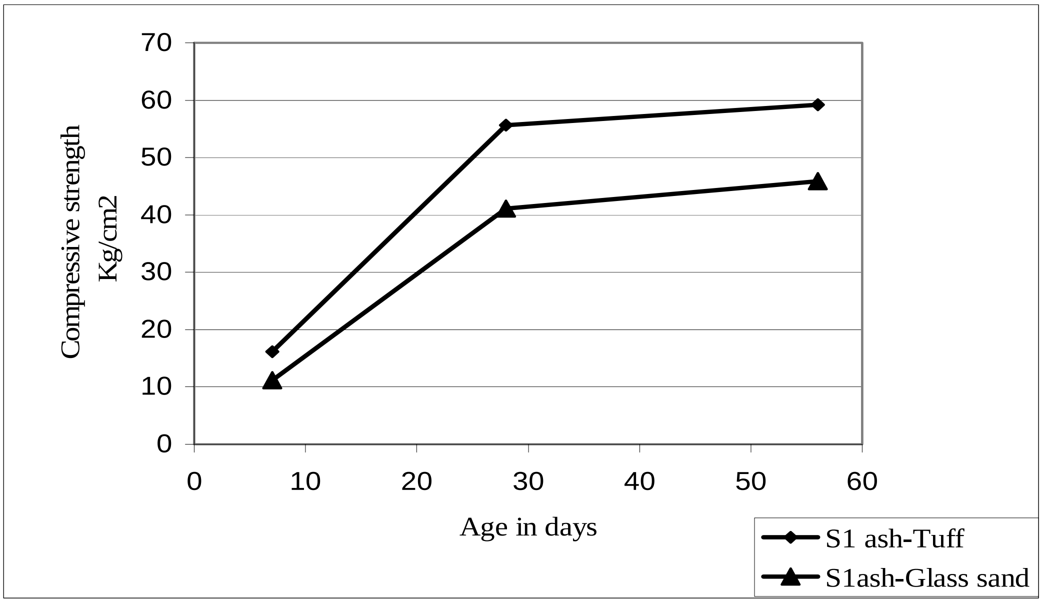

Figure (51) Compressive strength for tuff and glass sand mortars.

This is an expected result due to relatively high concentrations of the pozzolanic part (Al_2O_3, Fe_2O_3 and SiO_2) in the tuff sample which reacts with the alkaline part of the used ash. Such a reaction produces a self cementing material that gains strength gradually with increasing the curing time.

3.8.3 Ash-polystyrene mortar

The results of this trial are good and practical. The samples have revealed a very low density mortar if compared with the other mortars. The mix density and compressive strength results are shown in Table (56).

Table (56) Compressive strength results of ash-polystyrene mortar

Mix designation	Ash2 (gr)	Polystyrene cm^3	Water (gr)	Density gr/cm^3	7 days strength kg/cm^2	28 days strength kg/cm^2
AP	600	1000	600	0.8	13	22.4
AP	600	1000	600	0.81	12.5	23.7
AP	600	1000	600	0.82	12	21.9

The tested samples have shown a plastic behavior under compression. The compressive strength results are considered satisfactory to be utilized in various construction aspects in which low strength with ultra light density is required.

3.8.4 Mortars with S1, S2, S3 with OPC and glass sand

The compressive strength of standard hydraulic mortars (500gr cement+ 1750gr of standard glass sand) showed an increasing compressive strength with the addition of 18% S1 or S2 ash to the standard mortar mix. The addition of extra amounts will be considered as additional fines that decrease the original content of OPC in the original strong paste in the mix. The compressive strength results are shown in Table (57).

Table (57) Compressive strength results of S1, S2 and S3 ash, glass sand and OPC mortars.

Mix designation	Glass sand (gr)	S1 (gr)	S2 (gr)	S3 (gr)	OPC (gr)	Water (gr	Density gr/cm^3	7 days strength kg/cm^2	28 days strength kg/cm^2
M0	1375	00	-	-	500	242.5	2.06	40.2	75
MS1-3	1375	15	-	-	500	242	2.16	88.3	71
MS1-6	1375	30	-	-	500	243	2.18	74.2	85
MS1-9	1375	45	-	-	500	243	2.17	80	222
MS1-12	1375	60	-	-	500	250	2.21	95	215
MS1-15	1375	75	-	-	500	250	2.22	112.2	212
MS1-18	1375	90	-	-	500	290	2.16	113.1	210
MS1-27	1375	135	-	-	500	310	2.16	113.1	210
MS2-15	1375	-	75	-	500	330	2.21	98	218
MS2-18	1375	-	90	-	500	300	2.22	114	220
MS2-27	1375	-	135	-	500	320	2.16	113	218
MS3-9	1375	-		75	500	350	2.21	53.9	200
MS3-15	1375	-		90	500	300	2.22	84.28	200
MS3-27	1375	-		135	500	315	2.06	65.80	185

Increasing the ash content above 18% would lead to decreasing the compressive strength. This means that the addition of low ratios of ash up to 18% by weight of OPC content will act as cementaceous filler in the mortar mix that may result in a gradual increase in the compressive strength.

3.9 Uncombusted ground bituminous limestone mortar

The tested samples of the ground uncombusted bituminous limestone have shown no stability and high susceptibility to disintegration if subjected to wet conditions after short and long dry curing periods. The high organic matter content and the absence of lime and decomposed clay minerals have prevented the formation of bonding phases and cohesion. This reveals that the combustion processes at various temperatures has played an important role in utilizing the produced ash as a self cementaceous material.

4. DISCUSSION

The bituminous limestone ash is similar to a great extent to the Portland cement. The ash and Portland cement are essentially composed of lime (CaO).

Silica (SiO_2) and alumina (Al_2O_3) are present at higher concentrations in the Portland cement and react with CaO at about 1425 ^{0}C to form alite. Heat treatment of the bituminous rocks and Portland cement raw material involves dehydration, thermal decomposition of clay minerals (300- 650 ^{0}C), decomposition of calcite (› 800 ^{0}C), the formation of belite (C_2S), tricalcium aluminates (C_3A), and tetracalcium alumina ferrite (C_4AF). The liquid phase and sintering at about 1425 ^{0}C form alite (C_3S) which is responsible for the strength of concrete.

El-Lajjun bituminous limestone ash has revealed a self cementaceous behavior for the various prepared samples. Three samples for the objectives of this work, S1, S2 and S3 have been combusted at 1200, 950 and 525 ^{0}C respectively. The development of the self cementaceous properties and strength are controlled by the temperature of combustion of the bituminous limestone, the lime content and the curing period. The results are obtained through the standard mixtures of each ash sample. The mixtures are prepared and cured under the same standard procedures and conditions. Each ash type has its own chemical composition as a result of variation of the temperature of combustion. All of the ash samples S1, S2 and S3 are essentially composed of CaO. The variable CaO values are the result of LOI variation. The three types of ash samples S1, S2 and S3 have a relatively similar chemical composition and are analogous to the raw materials used to produce the OPC.

The alkali content which is presented by CaO, the pozzolanic content which is presented by (SiO_2 +Al_2O_3+ Fe_2O_3), and the variable content of SO_3 are found in both the ash samples and OPC raw material. The strength buildup in all the ash samples (S1, S2 and S3) is related to the setting reactions of lime (CaO) with the pozzolanic constituents to produce calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH). High pH solution is highly reactive with amorphous Al-Si rich phases at normal room temperature. Sulfate minerals as ettringite are expected to form because of the availability of SO_3 in the El-Lajjun bituminous limestone and its combusted products (S1, S2 and S3 ash samples).

The hydration products of the S1, S2 and S3 as identified by the XRD technique are portlandite, ettringite, calcium silicate hydrate and calcium aluminum hydrate. The reactions are not spontaneous and are time dependent. Curing period of 28 days and more has influenced the compressive strength results. High compressive strength values are obtained with intact samples indicating no disintegration features under fully saturated conditions. All hydrated samples have shown a similar behavior to the hydrated OPC products but with lower compressive strength.

The compressive strength of S1 mortar is 11.7, 41.1 and 52 kg/cm^2 at 7, 28 and 56 days respectively. For S2 mortar, the compressive strength is 20.4, 55.2 and 68.3 kg/cm^2 at 7, 28 and 56 days respectively. The compressive strength of S3 mortar is the lowest 7.3, 18.6, and 19.1 kg/cm^2 at 7, 28 and 56 days respectively. The CaO content plays an important role in the alkali–pozzolanic reaction. This conclusion is approved by preparing the S3-A mixture where half of the S3 content by weight (500 gr) is replaced by the S2 ash (250 gr S3: 250 gr S2). The compressive strength has been increased from 18.6 to 35.4 kg/cm^2 and from 19.1 to 42.2 kg/cm^2 at 56 days.

The compressive strength of the non standard mixture S3-C has reached unexpected high value results at 7, 28 and 56 days when it is compared with S1 and S2 ash-glass sand mixtures at the same curing conditions and age. The high value results are related to the increase of S3 ash content and the decrease of glass sand content (1375gr ash: 500gr glass sand). The high strength is possibly related to the formation of high ettringite content when it is compared to S1 and S2 ash samples. However, the S1 and S2 samples gave lower compressive strength values as a result of the increase ash content at the expense of the glass sand in the S1 and S2 mixtures. This conclusion is due to the deficiency in the silica content and hence the less CSH and CAS binding phases.

To stabilize the brown soil, S1, S2 and S3 ash-brown soil mixtures are prepared with equivalent weights (ash/soil=1:1, and 1:2). The mixtures are compacted to maximum dry density and optimum moisture content. The strength gain in the S1 and S2 soil mixtures has increased with the increase of silica content in the stabilized soil samples. The unconfined compressive strength of brown soil (sandy silty clay) has been raised from 6 kg/cm^2 to 50, 42 and 32 kg/cm^2 at 28 days respectively. Strength gain of S3-brown soil mixtures has revealed lower strength parameters than S1 and S2-brown soil mixtures using the same soil at the same curing time. This is related to the hydration behavior of the S3 ash, lower CSH and CAS content in the mixtures, and the appearance of ettringite as a bonding material in the S3 mixtures.

The unconfined compressive strength of the ash-clayey marl mixtures are prepared in the same way as ash- brown clay mixtures using the same ash content and curing period of 28 days. The compressive strength has shown 25, 21 and 31 kg/cm^2 for S1, S2 and S3-clayey marl mixtures respectively. The unconfined compressive strength of the S3-brown soil and S3- clayey marly mixtures are very close (31 kg/cm^2). Equal strength regardless of the soil type has been obtained when the S3 type is used, and hence the chemical composition of the S3 ash is the controlling factor that is responsible for the strength buildup in the different ash-soil mixtures.

The CBR values for mixtures of the stabilized brown soil and clayey marl-with S1, S2 and S3 ash have shown that the CBR has increased from 4% for the unstabilized parent soil samples to more than 120%. Complex reactions are expected in the various S1, S2, and S3 mixtures. Lime (CaO), plays an important role in the strength buildup. The increase of CSH and CAS phases in S1 and S2 ash samples and it's related mixtures is responsible for the increase of compressive strength. Strength gain of S3 mixtures has revealed lower strength parameters than S1 and S2 mixtures using the same soil and curing time. This is related to the hydration behavior of the S3 ash (absence of lime and the presence of amorphous Al-silicate phases), lower CSH and CAH content in the mixtures, and the appearance of more ettringite as a binding material in the S3 mixtures. The scanning electron microscope (SEM) testing has been conducted on S3-brown soil and clayey marl. In the treated soils, an abundance of long, platy and fibrous crystals are found in the S3-ash-treated soil mixtures. The morphology of these crystals is typical of ettringite, CAS and CSH.

Subgrade or embankments of brown soil or marly clay as problematic soils can be stabilized efficiently by using any of S1, S2 or S3 ash. The material type has its effect on the strength buildup when S1 and S2 ash types are used. The increase of CBR values is not dependant on the soil type (soil or clayey marl) when S3 type ash is used.

A non combusted background bituminous limestone mortar is used to compare its compressive strength with the strength parameters of S1, S2 and S3 mortars. The background sample has indicated a low strength as it is submerged in water for 28 days as a result of the absence of binding phases. This is due to the absence of lime and amorphous Al-silicates. This result confirms the role of CaO content in the strength buildup.

The reactions that take place during the OPC production are similar to those that have produced S1, S2 and S3 samples at 1200, 950 and 525 ^{0}C. Cement manufacturing involves grinding, blending, pre-calcining of the raw materials, clinker burning and cement grinding. Limestone ($CaCO_3$) and other materials containing calcium, silicon, aluminum and iron are crushed and milled into a raw meal. This raw meal is blended and is then heated in the pre-heating system to initiate the dissociation of carbonate to calcium oxide and carbon dioxide. The meal then proceeds to the kiln for heating and reaction between calcium oxide and other elements to form calcium silicates and calcium aluminates at a temperature up to 1450 ^{0}C. The reaction products leave the kiln as a clinker. The clinker leaves the kiln here to be cooled, mixed with gypsum, ground into a fine powder (cement). The setting of cement involves a number of stages at different rates. A complex series of reactions take place as the cement reacts with water. Setting of C_2S involves slow hydration reactions and the formation of Portlandite $Ca(OH_2)$ and calcium silicate hydrate. These products are expected to be the essential components of S1, S2 hydrated products. Scanning electron micrographs and X-ray diffraction results of S1 and S2 –brown soil and clayey marl mixtures have indicated the growth of fibrous crystals which play an important role in contributing to the strength of the self cementitous material (ash). The fibrous crystals do continue to hydrate late in the setting period. Tri-calcium aluminate in the S1 and S2 ash samples react very rapidly with water, giving crystalline hydrated products. These products are responsible for the initial formation of crystalline network.

In the ash samples S1 and S2, the excess lime is present and the formation of tri-calcium aluminate C3S is favored. In Portland cement, the addition of gypsum could lead to the formation of ettringite $Ca_6\ Al_2O_6(SO_4)_3.31\text{-}32\ H_2O$. As the mixture ages, the fibrous crystalline network continues it’s growth, and the crystals become coarser and form an interlocking network. The critical aspects in setting include the formation of CSH and CASH phases to provide the observed strength of the S1 and S2 mixtures.

Portlandite $Ca\ (OH)_2$ plays an important role in the setting reaction. Portlandite reacts with silicates and aluminum rich phases to form insoluble compounds which contribute to the strength formation (pozzolanic reactions). Excess portlandite reacts with atmospheric CO_2 to precipitate calcium carbonate that helps in strengthening the product after aging.

The S3 ash sample which is prepared at 525 ^{0}C has shown the lowest compressive strength values. This is due to the absence of lime (CaO) and the possible formation of calcium silicate hydrate (CSH) and calcium aluminum silicate hydrate (CASH). Sulfate phases (ettringite, anhydrite and gypsum) are possible binding phases. Ettringite is characterized by a very open sieve like structure. Ettringite is found as a direct product of mixing S3 ash with brown soil, clayey marl and glass sand.

The scanning electron micrographs and X-ray diffraction results of the prepared S1 and S2 ash-brown soil and clayey marl mixtures have proved the presence of fibrous network of calcium aluminum silicate hydrate phases which could contribute in increasing the binding strength of the hydrated mixtures.

The combusted bituminous rocks in central Jordan have indicated the presence of two groups of minerals; high temperature which is equivalent to clinker cement

(Khoury, 1993) and low temperature which is similar to the hydrated cement products (Khoury and Nasser, 1982). The low temperature mineral group has a similar composition to the hydrated cement products and has been precipitated from high alkaline circulating water ($pH > 12.5$). This naturally occurring alkaline water is analogous to the cement percolating water (Khoury, 1993).

The bituminous limestone in central Jordan and as a result of the spontaneous combustion has similar phases as those found in Portland cement, hydrated cement products and S1, S2, S3 ash-brown soil and clayey marl mixtures.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

1. The ash is produced through direct combustion of the parent deposits at 1200^{0}C or 950 ^{0}C in thermal power plants and at 525 ^{0}C in retortion processes.
2. The ash from El-Lajjun area is a solid waste that can be utilized in various engineering applications.
3. The ash is considered a high potential self cementaceous material that can be utilized in various engineering applications.
4. Strength of ash mortars has been developed using normal methods and available natural materials. Cube strength up to 90 kg/cm^2 has been obtained at 56 days.
5. The unconfined compressive strength of S1, S2 and S3 brown clay and clayey marl mixtures has been increased with the increase of the ash content.
6. The unconfined compressive strength has reached 50 kg/cm^2 for brown soil and 35 kg/cm^2 for clayey marl at 28 days.
7. The CBR for both ash-brown soil and clayey marl has reached 100% after 28 days.
8. The El-Lajjun ash has revealed a very high efficiency in the stabilization of problematic soils as brown sandy silty clay and clayey marl in the vicinity and around the capital Amman.
9. The problematic brown soil and clayey marl subgrade under highways and roads can be easily treated and modified to fit the construction purposes.
10. Various types of mortars and construction materials can be produced for appropriate civil works without the use of OPC.

11. The ash is not suitable to be utilized as a partial or complete substitute in normal concrete mixes for structural purposes due to its high alkaline content.
12. Roller compacted concrete (RCC) and cement treated base (CTB) can be produced utilizing ash as a complete substitute of the OPC.
13. Utilization of El-Lajjun ash for various engineering purposes will contribute in minimizing the environmental impact of the expected huge output of solid waste resulting from the retortion of the bituminous rocks.
14. The self cementaceous characteristics of the investigated ash types have shown a variant efficiency in soil stabilization. The chemical composition of the treated soil is thus considered as a major factor in controlling the properties of S1 and S2-soil mixtures.
15. Ettringite is the strength controlling factor in the variation of properties of the S3 ash-soil mixtures.
16. The strength buildup is related to ash content, ash type and curing period.

5.2 Recommendations

1. The high alkalinity character of the bituminous limestone ash could be utilized to neutralize acid waste.
2. The bituminous limestone ash could be used to stabilize the rejected output of quarries waste materials.
3. The high absorbance capacity of the ash could be utilized for the absorption of the leacheate resulting from disposal sites.
4. This study is limited to cohesive subgrade soils; it would be interesting to conduct a similar research on granular subgrade soils.

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Appendices

Appendix I

Calcium carbonate d-values using Co Kά1

Name: Calcium carbonate
Name: Calcite
Formula: $CaCO_3$
Radiation: Co Kά1
Wavelength: 1.78897

d-value	Angle	Relative intensity
3.8520	26.855	29
3.0300	34.340	100
2.8340	36.799	2
2.4950	42.017	7
2.2840	46.112	18
2.0940	50.575	27
1.9261	55.343	4
1.9071	55.942	17
1.8726	57.067	34
1.6259	66.754	2
1.6040	67.788	15
1.5821	68.857	2
1.5247	71.841	3
1.5061	72.869	2
1.4405	76.771	5

Appendix II

Calcium magnesium carbonate d-values using Co Kά1

Name: Calcium magnesium carbonate
Name: Dolomite
Formula: $CaMg(CO_3)_2$
Radiation: Co Kά1
Wavelength: 1.78897

d-value	Angle	Relative intensity
4.0300	25.648	3
3.6900	28.057	5
2.8860	36.111	100
2.6700	39.147	10
2.5400	41.239	8
2.4050	43.669	10
2.1920	48.167	30
2.0660	51.310	5
2.0150	52.707	15
1.8480	57.898	5
1.8040	59.449	20
1.7860	60.110	30
1.7810	60.296	30
1.5670	69.616	8
1.5450	70.754	10
1.4650	75.261	5
1.4450	76.489	4
1.4310	77.375	10
1.4130	78.549	4
1.3890	80.178	15
1.3350	84.138	8
1.2380	92.526	5

Appendix III

Silicon oxide d-values using Co Kά1

Name: Silicon oxide
Name: Quartz, low
Formula: SiO_2
Radiation: Co Kά1
Wavelength: 1.78897

d-value	Angle	Relative intensity
4.2600	24.241	35
3.3430	31.039	100
2.4580	42.681	12
2.2820	46.155	12
2.2370	47.139	6
2.1280	49.712	9
1.9800	53.713	6
1.8170	58.982	17
1.6720	64.685	7
1.6590	65.254	3
1.5410	70.965	15
1.4530	75.993	3
1.3820	80.667	7
1.3750	81.164	11
1.3720	81.378	9
1.2880	87.971	3
1.2560	90.823	4
1.1997	96.419	5
1.1838	98.156	4

Appendix IV

Calcium hydroxide d-values using Co Kά1

Name: Calcium hydroxide
Name: Portlandite, syn.
Formula: $Ca(OH)_2$
Radiation: Co Kά1
Wavelength: 1.78897

d-value	Angle	Relative intensity
4.9000	21.036	74
3.1120	33.408	23
2.6280	39.798	100
2.4470	42.882	3
1.9270	55.315	42
1.7960	59.741	36
1.6870	64.041	21
1.5570	70.128	3
1.4840	74.134	13
1.4490	76.240	13
1.3140	85.802	8
1.1762	99.014	3
1.1432	102.969	11
1.1275	104.969	2
1.0599	115.115	12
1.0366	119.288	5
1.0143	123.738	7
0.9551	138.954	4

Appendix V

Calcium   silicate hydrate d-values using Co Kά1

Name: Calcium   silicate hydrate
Formula: Ca.5 SiO_3. 5x H_2O
Radiation: Co Kά1
Wavelength: 1.78897

d-value	Angle	Relative intensity
3.0400	34.224	100
2.7900	37.399	60
1.8200	58.875	60

Appendix VI

Calcium aluminum silicate d-values using Co Kά1

Name: Calcium aluminum silicate
Formula: Ca-Al-Si-O
Radiation: Co Kά1
Wavelength: 1.78897

d-value	Angle	Relative intensity
9.6900	10.593	100
5.5600	18.516	25
4.9400	20.864	10
4.6800	22.037	8
3.8500	26.869	14
3.5700	29.021	6
3.4400	30.143	14
3.2000	32.464	6
3.0000	34.694	4
2.8800	36.189	10
2.7700	37.679	8
2.7400	38.107	10
2.6600	39.300	6
2.5800	40.571	10
2.5400	41.239	16
2.1570	48.999	4
2.1300	49.662	8
2.0560	51.568	4
2.0340	52.178	4
1.9480	54.668	4
1.9170	55.628	6
1.7860	60.110	2
1.6650	64.990	4
1.6450	65.880	2
1.5590	70.025	2

Appendix VII

Ankarite d-values using Co Kά1

Name: Calcium magnesium iron carbonate
Name: Ankarite
Formula: $Ca(Mg.67FeO.33)(CO_3)_2$
Radiation: Co Kά1
Wavelength: 1.78897

d-value	Angle	Relative intensity
3.7000	27.980	3
2.8990	35.944	100
2.6850	38.919	3
2.5520	41.036	1
2.4110	43.555	3
2.1990	48.004	6
2.0670	51.284	1
2.0200	52.567	3
1.8520	57.761	1
1.8120	59.161	6
1.7920	59.888	6
1.5690	69.514	1

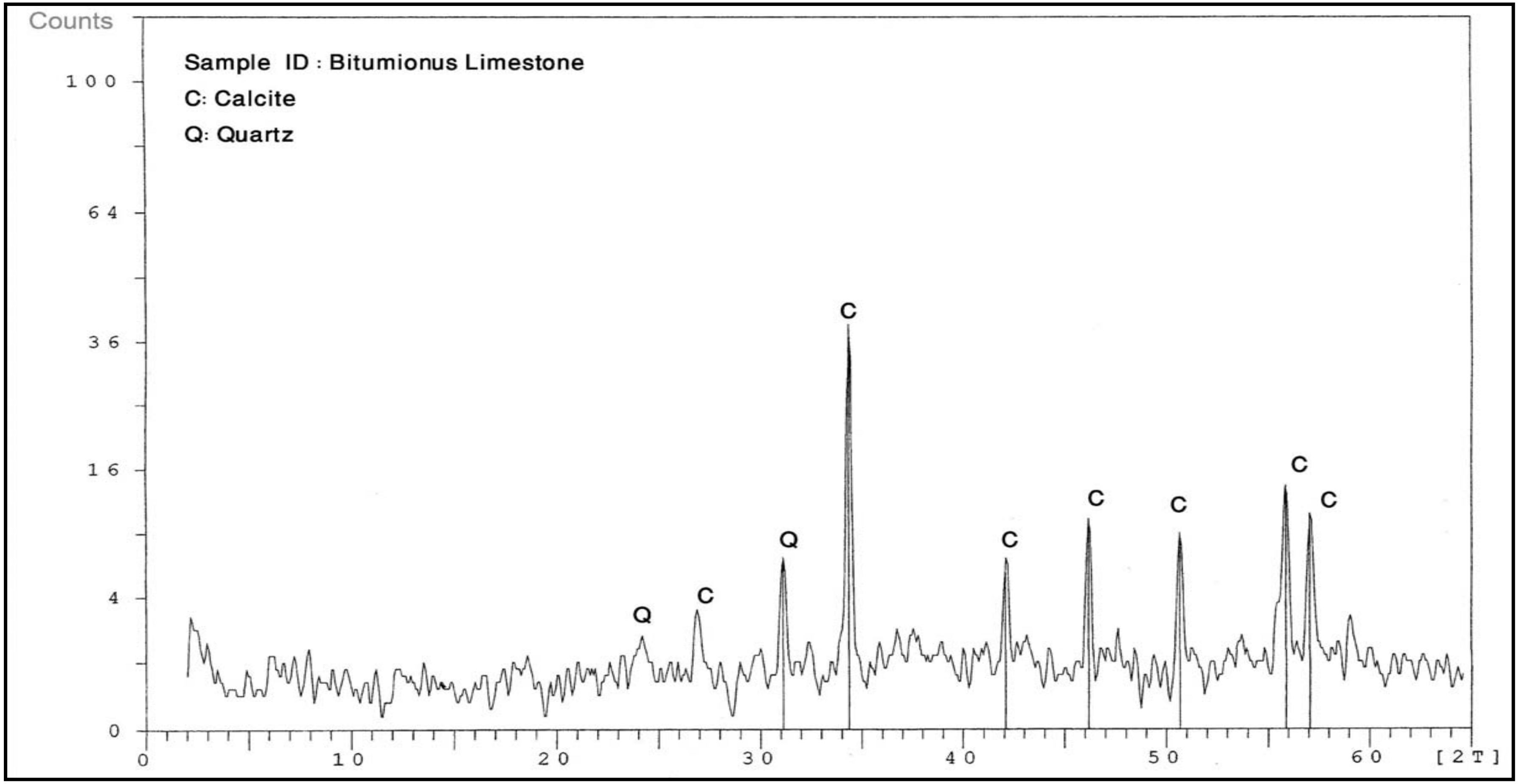
Appendix VIII

Ettringite d-values using Co Kά1

Name: Calcium aluminum carbonate silicate sulfate hydroxide
Name: Ettringite
Formula: $Ca_6Al_2(SO_{4,}\ SiO_4, CO_3)3(OH)_{12.}$ 26 H_2O
Radiation: Co Kά1
Wavelength: 1.78897

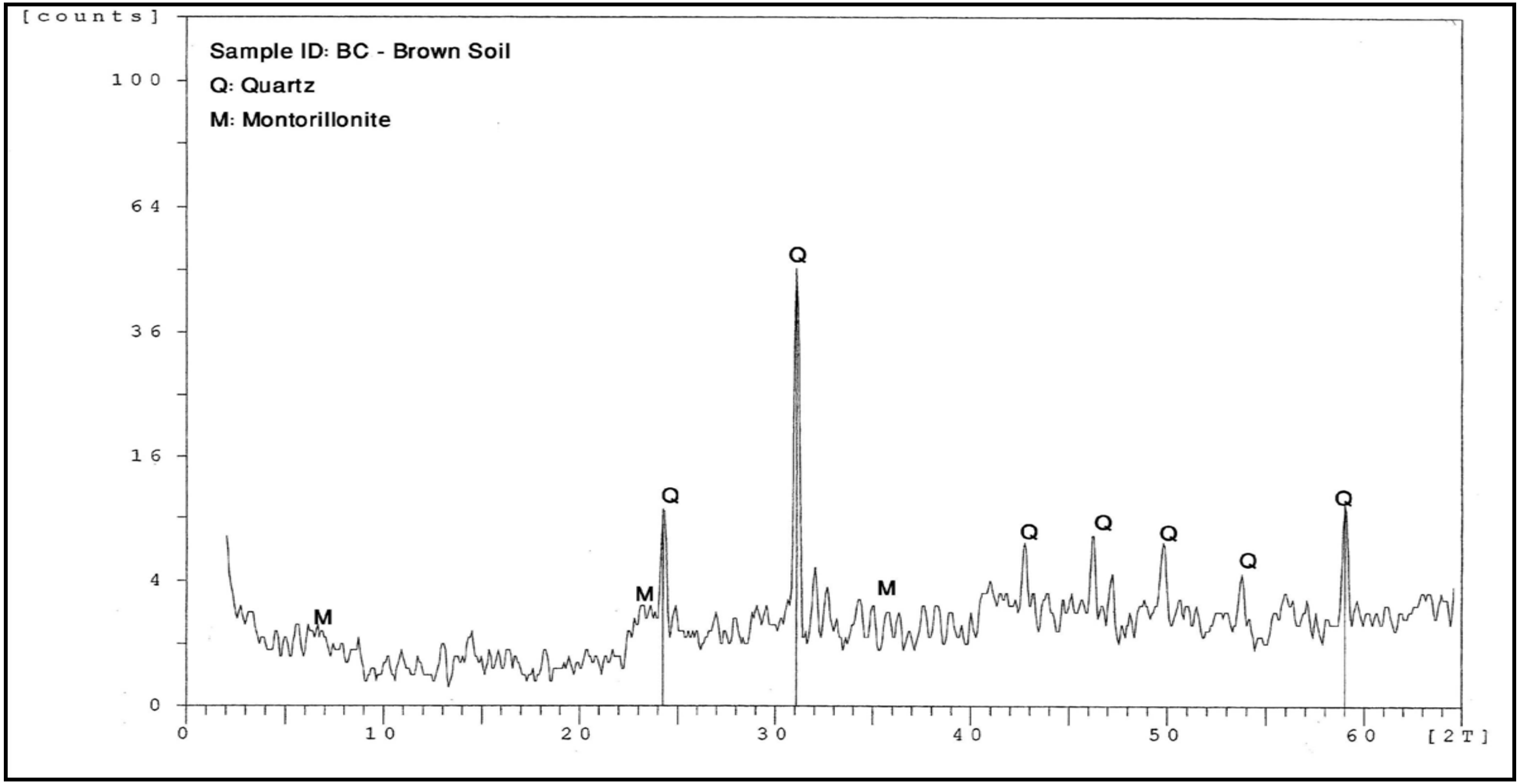
d-value	Angle	Relative intensity
10.500	9.774	5
9.6700	10.615	100
8.8800	11.562	5
6.1200	16.809	5
5.6000	18.382	90
4.9800	20.695	30
4.6800	22.037	40
4.2900	24.069	5
4.0300	25.648	5
3.8800	26.657	60
3.6500	28.371	5
3.5900	28.856	5
3.4800	29.788	40
3.2400	32.052	30
3.0200	34.457	5
2.7600	37.821	50
2.6900	38.844	5
2.6100	40.085	20
2.5600	40.902	60
2.4970	41.982	5
2.4160	43.460	10
2.3500	44.746	5
2.2040	47.888	60
2.1550	49.048	30
2.1280	49.712	5
2.2.0800	50.940	5
1.9460	54.729	20
1.6630	65.078	20

Appendix IX



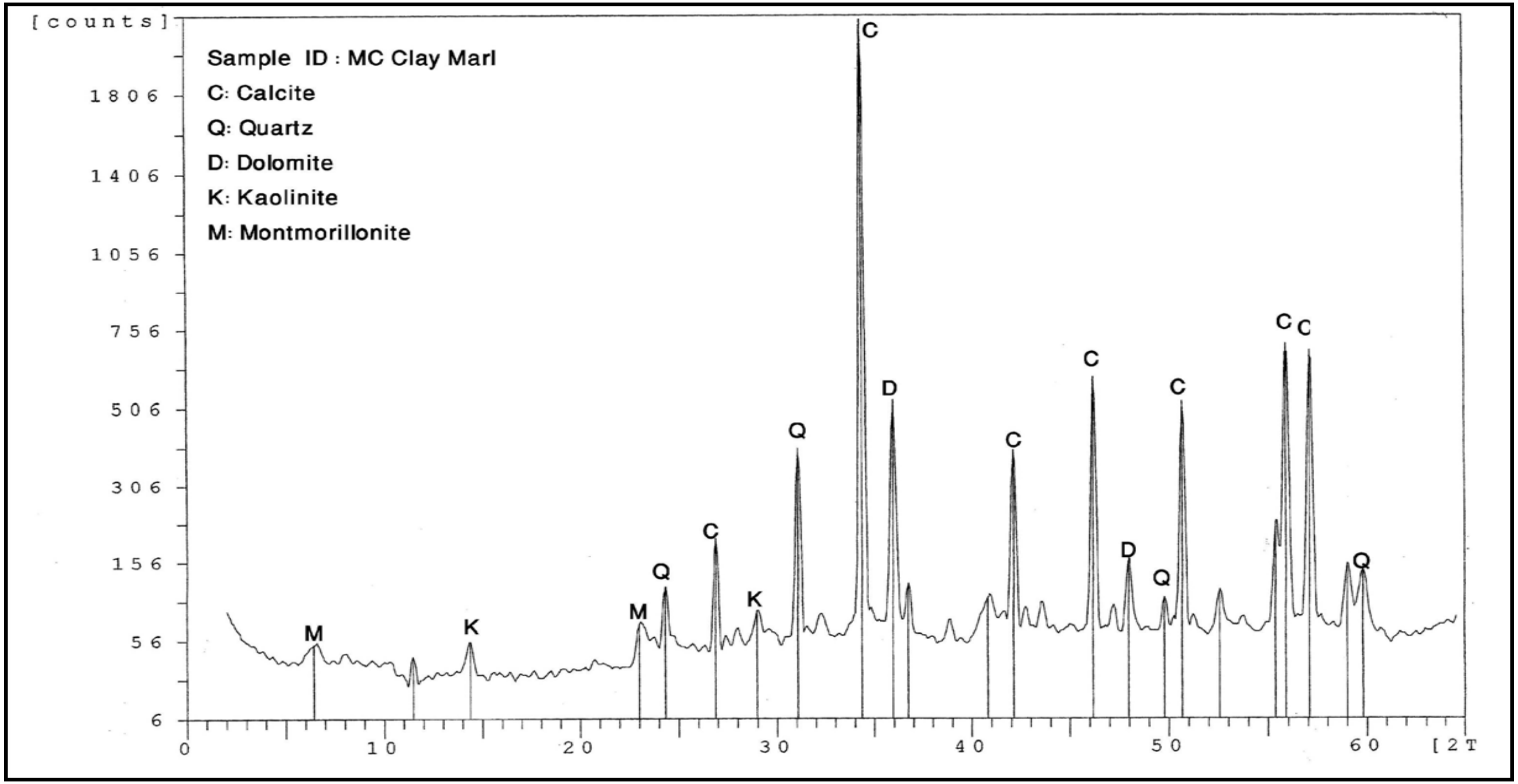
XRD pattern of the bulk bituminous limestone sample

Appendix X



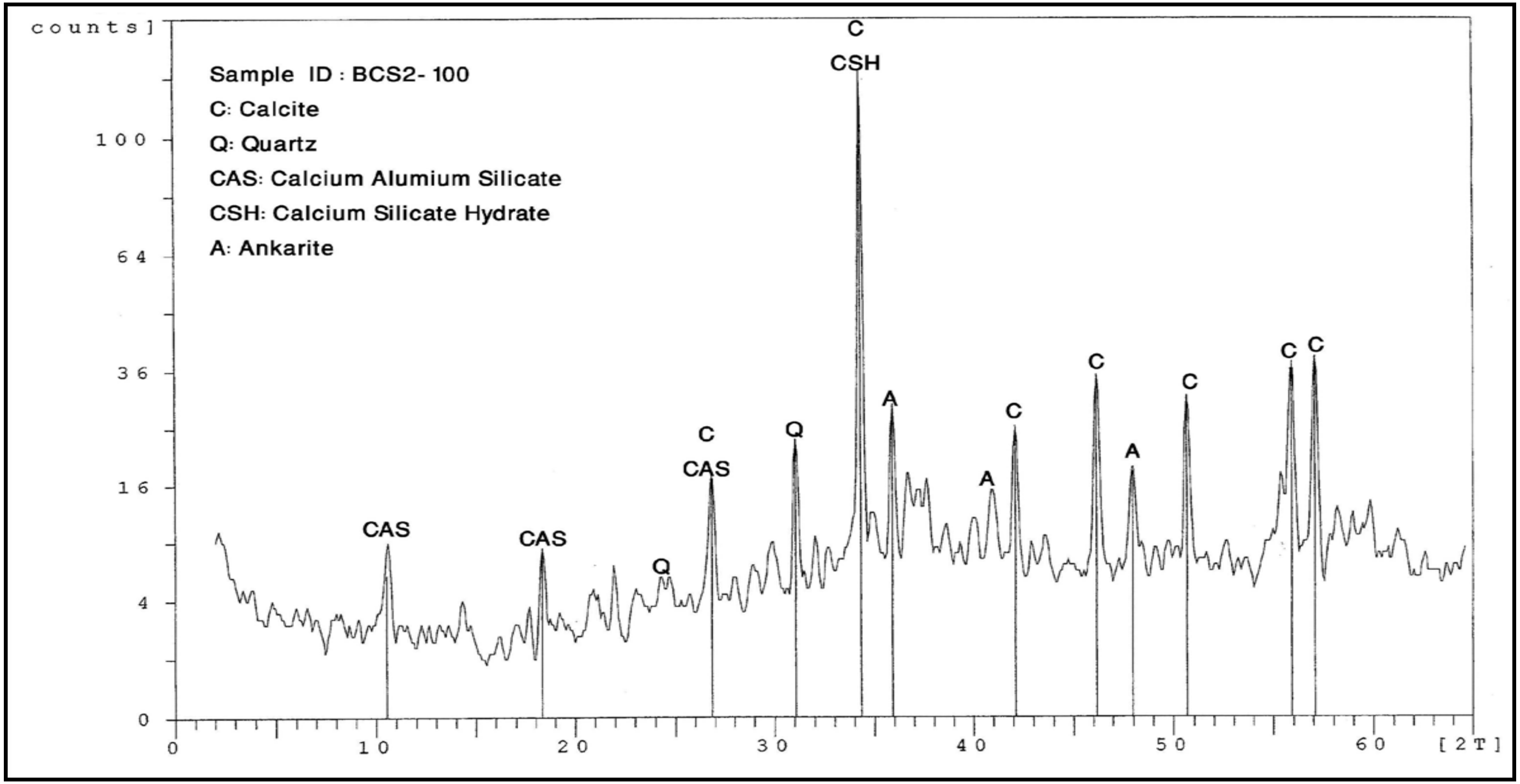
XRD Pattern of the brown soil

Appendix XI



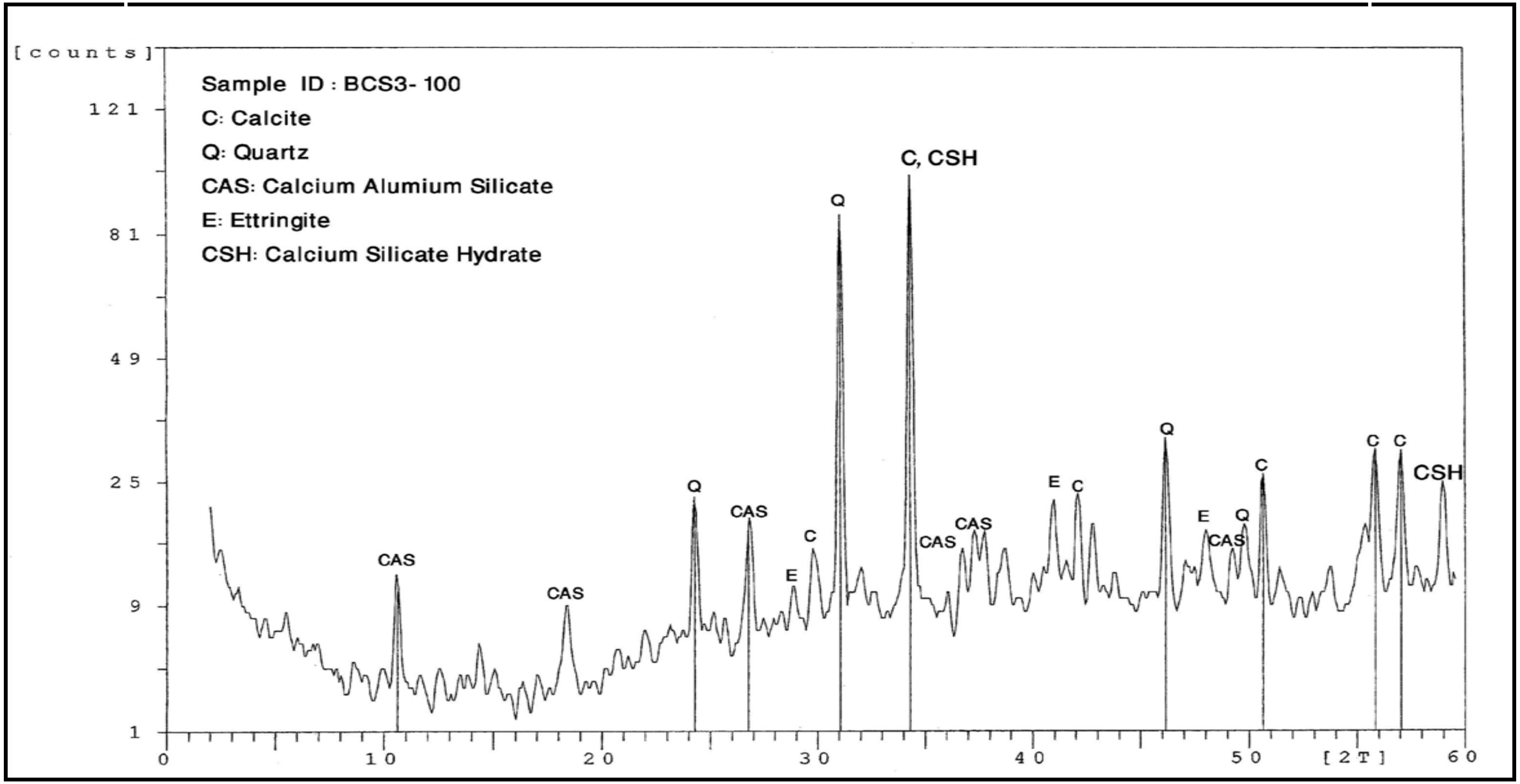
XRD pattern of the clayey marl sample

Appendix XII



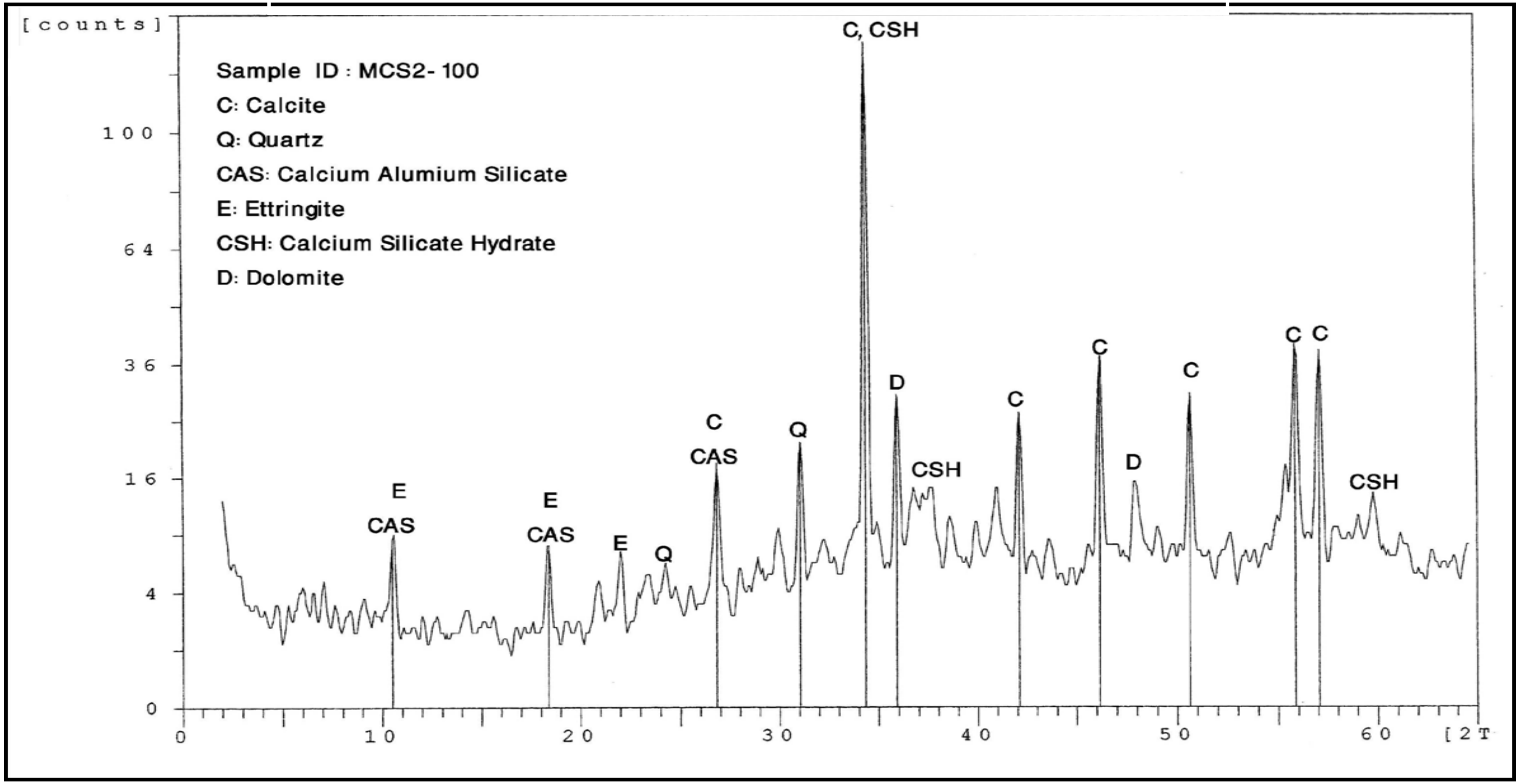
XRD pattern of brown soil-S2 mixture

Appendix XIII



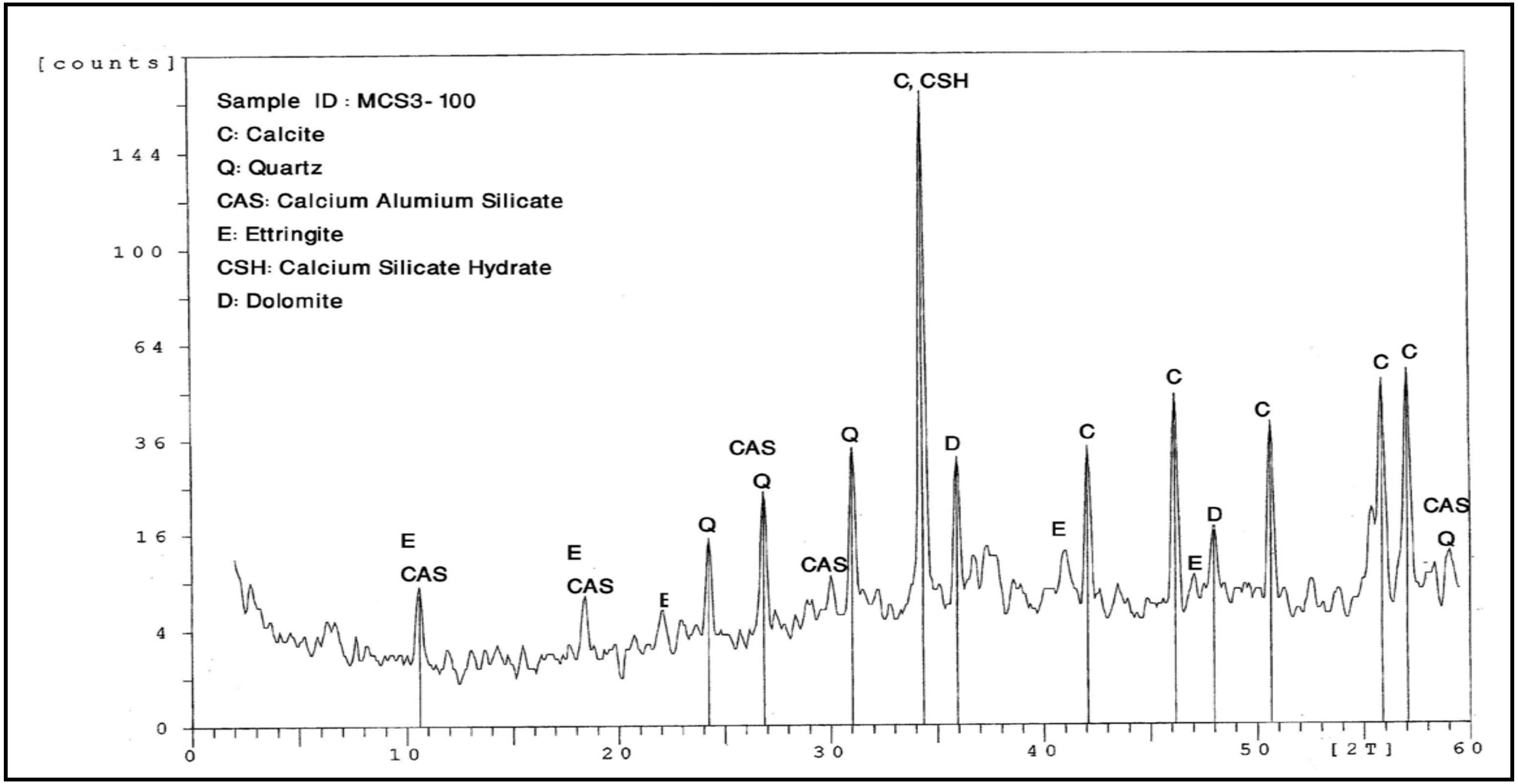
XRD pattern of the brown soil-S3 ash mixture

Appendix XIV



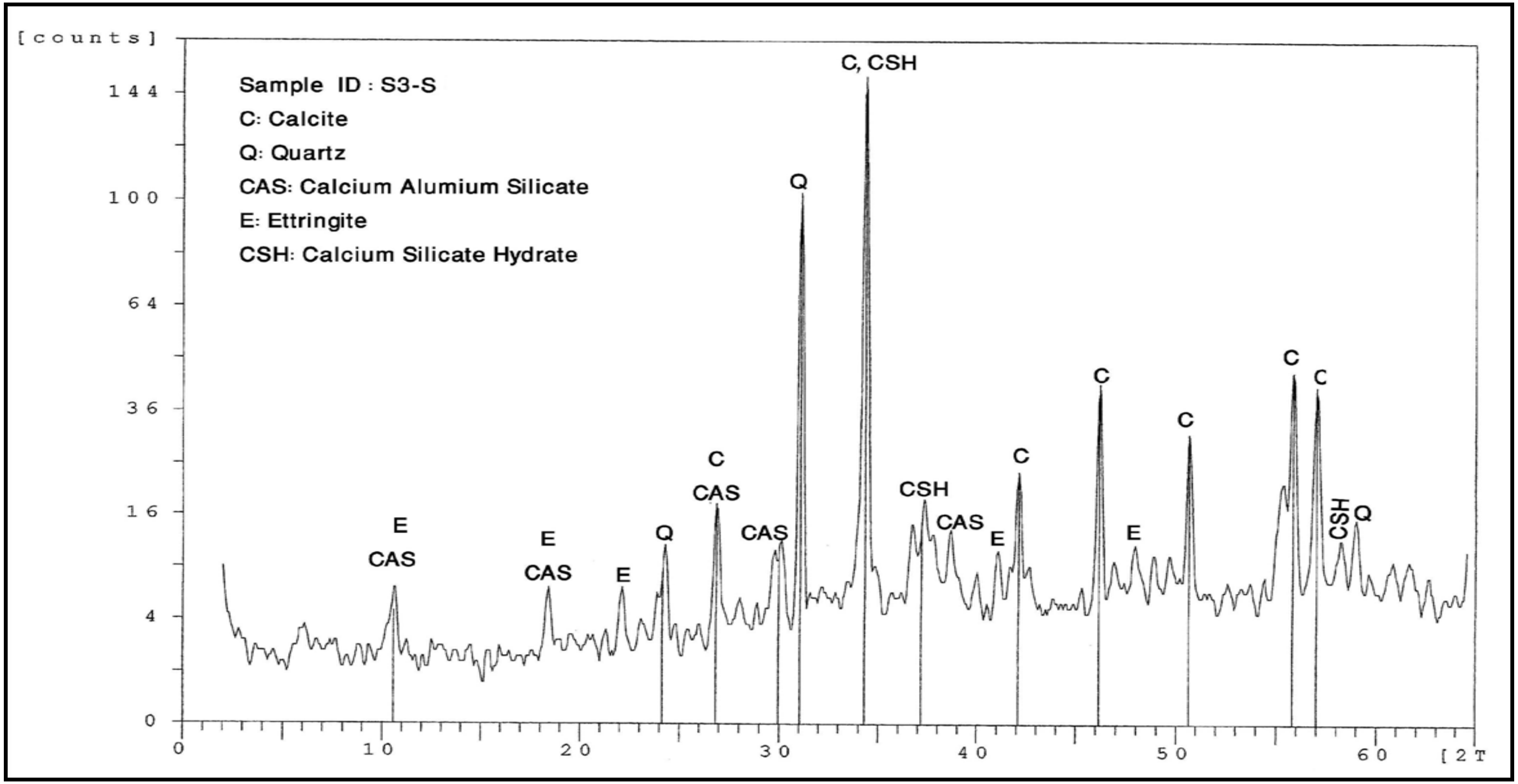
XRD pattern of the clayey marl-S2 ash mixture

Appendix XV



XRD pattern of the clayey marl-S3 mixture

Appendix XVI



XRD pattern of the glass sand-S3 ash mixture

Appendix XVII

Ultra light weight Ash- polystyrene mix

Ash – tuff mix

Light weight ash block

Ash – granite aggregate concrete

Appendix XVIII

 Ash- tuff –sand mixture	 Ash stabilized clayey marl
 Brown soil, S3 ash stabilized sample	 Ash stabilized brown soil

Appendix XIX

OPC free, ash cemented materials

Ash stabilized clayey marl