

Assesment of the geotechnical properties of stabilized sand-marl mixtures for constructional purposes.

Khalid A. Al-Gunaiyan

Civil Engineering

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Abstract

The use of abundant eastern Saudi sands in construction helps control sand movements which are known to result in many hazards, especially in windy seasons. These sands may not be used as a foundation material in their natural condition. However, when improved, they can be utilized in many applications. Therefore, there is a need to conduct an investigation on sands to help delineate their relevant engineering properties and establish a data base for their eventual classification, as part of regional codes. This is the first primary objective of this study.

Desert and beach dune sands samples were collected from ten (10) different locations in the eastern province of Saudi Arabia. These samples were carefully retrieved from areas having abundant sands that can be used in construction. The samples were characterized and their geotechnical properties were determined. Results revealed that most of these sands are classified as poorly graded and all sands show somewhat similar behavior.

Marls are considered the best construction material in eastern Saudi Arabia. However, the extensive use of marl has depleted local supplies. Such a situation has necessitated the search for alternative material sources. Therefore, blending sand with marl in construction will result in an alternative material, that can be treated or stabilized to improve its engineering properties. This will save the good quality aggregate for future construction projects. In addition, there is a potential need in stabilizing such blends using chemicals such as Portland cement or bitumen (emulsified and cutback asphalts) that result in a material with acceptable engineering characteristics and are economically advantageous. This constituted the second primary objective of this study.

Results have revealed that most of eastern Saudi sands are classified as poorly graded and all sands show somewhat similar behavior. It also showed that stabilizing sand-marl mixes using Portland cement has brought up a material with the most acceptable engineering characteristics. The mix meets the strength and durability requirements and is economically advantageous. On the other hand, stabilizing the mixes using bitumen was rejected.

Assessment of the Geotechnical Properties of Stabilized Sand-Marl Mixtures for Constructional Purposes

by

Khalid A. Al-Gunaiyan

A Thesis Presented to the

FACULTY OF THE COLLEGE OF GRADUATE STUDIES

KING FAHD UNIVERSITY OF PETROLEUM & MINERALS

DHAHRAN, SAUDI ARABIA

In Partial Fulfillment of the
Requirements for the Degree of

MASTER OF SCIENCE

In

CIVIL ENGINEERING

December, 1998

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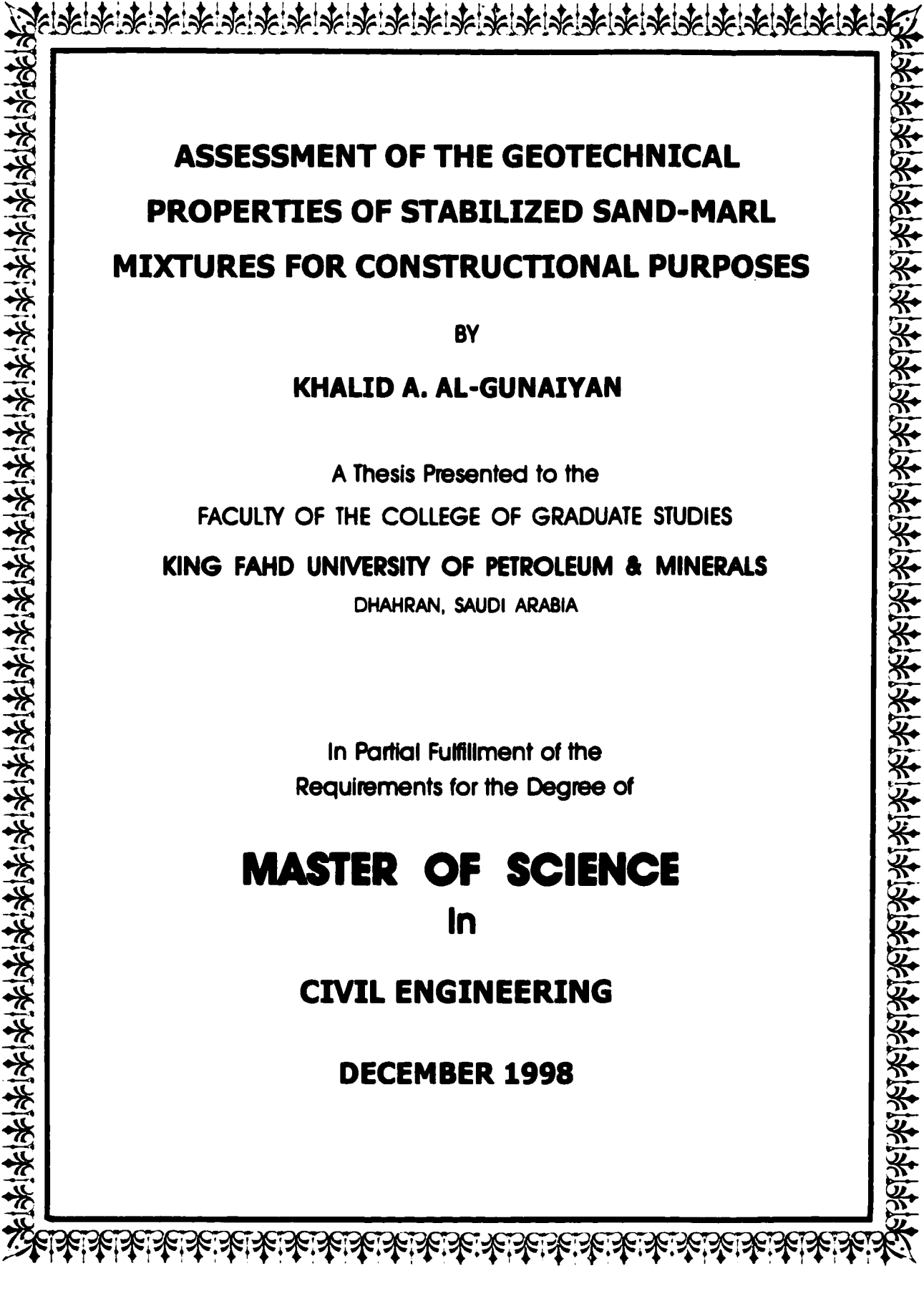
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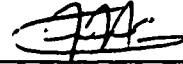
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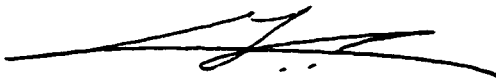
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TABLE OF CONTENTS

	page
Acknowledgments	i
List of Tables	v
List of Figures	vi
Abstract (English)	viii
Abstract (Arabic)	ix
1 Introduction	1
1.1 Background	1
1.2 Problem Statement	3
1.3 Objectives	5
1.4 Thesis Organization	6
2 Literature Review	7
2.1 Sands	7
2.1.1 Aeolian Deposits in the Arabian Peninsula	9
2.1.2 Mineralogical Composition of Sands	12
2.1.3 Gradation of Sands	14
2.1.4 Sand as a Construction Material	15
2.1.5 Behavior of Cemented Sands	15
2.2 Marl Soil	22
2.2.1 Background	22
2.2.2 Mineral Composition of Marl	27

2.2.3	Engineering Behavior of Marl	30
2.2.3.1	Background	30
2.2.3.2	Consistency Limits	31
2.2.3.3	Grain-Size Distribution	33
2.2.3.4	Strength	34
2.2.4	Soil Stabilization	36
2.2.4.1	Cement Stabilization	38
2.2.4.2	Bituminous Stabilization	40
2.2.4.3	Mix Design for Bituminous Stabilization	42
2.2.4.4	Sand and Marl Stabilization	44
2.2.4.5	Additives	46
3	Experimental Program	54
3.1	Collection of Sand Samples	56
3.2	Sand Characterization	57
3.2.1	Specific Gravity Test	57
3.2.2	Grain-Size Distribution	61
3.2.3	Relative-Density Determination	61
3.2.4	Conventional Triaxial Compression Test	62
3.2.5	X-Ray Diffraction (XRD) Analysis	63
3.3	Sand Selection for Detailed Study	64
3.4	Selection of a Marl Sample	64
3.5	Optimization of Sand-Marl Mixtures	65
3.5.1	California Bearing Ratio (CBR) Test.....	67
3.6	Stabilization of Sand-Marl Mixtures	67
3.6.1	Optimization of Sand-Marl Mixtures Stabilization	68
3.6.1.1	Cement Content	69
3.6.1.2	Curing Conditions	70
3.6.2	Unconfined Compressive Strength	71
3.6.3	Modulus of Resilience	73

3.6.4	Durability	75
3.6.5	Emulsified and Cutback Asphalts Mix Design	79
3.6.5.1	Specimen Preparation	81
3.6.5.2	Curing Conditions	82
3.6.5.3	Marshall Stability Test	83
3.6.5.4	Creep Test	83
4	Results and Discussion	85
4.1	Characterization of Collected Samples	85
4.2	Characterization of Sand-Marl Mixes	96
4.2.1	General	96
4.2.2	Modified Proctor Compaction Tests	96
4.2.3	California Bearing Ratio (CBR) Test	98
4.3	Stabilization of Sand-Marl Mixtures	101
4.3.1	Stabilization using Portland cement	106
4.4	Cement Stabilized Sand-Marl Mixtures	117
4.4.1	Unconfined Compressive Strength	117
4.4.2	Modulus of Resilience	122
4.5	Bitumen Stabilized Sand-Marl Mixtures	124
4.5.1	Optimum Mix Design	125
4.5.2	Results and Discussion	127
5	Summary, Conclusions and Recommendations	142
5.1	Summary	142
5.2	Conclusions	143
5.3	Recommendations for Further Study	145
	References	146

LIST OF TABLES

2.1	Summary of Cited Information on Naturally and Artificially Cemented Sands...	18
2.2	Geological Formations of Eastern Saudi Arabia	23
2.3	Lists of some definitions for marl soil used by different authors.....	26
2.4	Chemical Analysis of selected marl soils.....	29
2.5	Variables affecting stability of sand stabilized with cationic emulsion	48
2.6	The influence of processing procedures on the strength of sand-cationic emulsion mixes	49
3.1	Locations of the collected sands	58
3.2	Description of the collected sands.....	59
4.1	Physical properties of the collected sands.....	87
4.2	Approximate Crystalline Minerals of some of collected sands	95
4.3	Classification of the Sand-Marl mixes	104
4.4	Physical properties for Sand-Marl Mixtures	126
4.5	Sand-Marl bitumen mix.....	128
4.6	Results of creep curves for bitumen-sand-marl mixes.....	138

LIST OF FIGURES

2.1	Sand Terrains in the Arabian Peninsula	8
2.2	Geological Formations of Eastern Saudi Arabia	24
3.1	General Optimization Procedure for sand-marl stabilization	55
3.2	Approximate locations of sand samples	60
3.3	Base Course Gradation specified by various organizations	66
3.4	The setup for Slake Durability Testing	78
4.1	Dry and washed grain size distribution for Dammam Sand #1	88
4.2	Dry and washed grain size distribution for Baggah Sand #1	88
4.3	Dry and washed grain size distribution for Abqiaq Sand #1	89
4.4	Dry and washed grain size distribution for Dhahran Sand #1	89
4.5	Dry and washed grain size distribution for Dhahran Sand #2	90
4.6	Dry and washed grain size distribution for Dhahran Sand #3	90
4.7	Dry and washed grain size distribution for Qurayyah Sand #1	91
4.8	Dry and washed grain size distribution for Qurayyah Sand #2	91
4.9	Dry and washed grain size distribution for Jubail Sand #1	92
4.10	Dry and washed grain size distribution for Dahana Sand #1	92
4.11	Dry sieve grain size distribution for Sand	93
4.12	Washed sieve grain size distribution for Sand	94
4.13	Moisture-density relationship for sand/marl mixes at different ratios	97
4.14	CBR-moisture content variations of sand/marl mixes at different ratios	100
4.15	Variation of the maximum CBR values with Baggah Sand content	102
4.16	Variation of the maximum CBR values with Dhahran Sand #2 content	103
4.17	Gradation of the sand/marl mixes at different ratios based on dry sieving	105
4.18	Variation of the dry density with moisture content	108

4.19	Variation of CBR with varied cement contents.....	109
4.20	Variation of UCS with moisture content for different percentages of cement content	110
4.21	Variation of UCS with cement content for sand/marl mixes	111
4.22	Variation of the weight loss with cement contents	113
4.23	Durability samples with hairline cracks for sand/marl mixes	115
4.24	Variation of the weight loss with sand contents for sand/marl mixes	116
4.25	Variation of the UCS with curing temperature for sand/marl mixes	120
4.26	Variation of the UCS with curing period for sand/marl mixes	121
4.27	Variation of M_R with curing temperature for 7 days cured samples.....	123
4.28	Variation of the dry stability with emulsion and added water contents.....	129
4.29	Variation of the dry and wet stability with emulsion content.....	130
4.30	Variation of the dry and soaked stability with cutback content.....	131
4.31	Variation of the Resilient Modulus with testing temperature.....	133
4.32	Density of sand-marl mix with and without additives.....	135
4.33	Variation of the unsoaked CBR with molding moisture content	136
4.34	Variation of the CBR with molding moisture content.....	137
4.35	Creep curves for sand/marl mixes treated with emulsion	139
4.36	Creep curves for sand/marl mixes treated with cutback.....	140

ABSTRACT

Name: Khalid A. Al-Gunaiyan
Title: Assessment of the geotechnical properties of stabilized sand-marl mixtures for constructional purposes
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The use of abundant eastern Saudi sands in construction helps control sand movements which are known to result in many hazards, especially in windy seasons. These sands may not be used as a foundation material in their natural condition. However, when improved, they can be utilized in many applications. Therefore, there is a need to conduct an investigation on sands to help delineate their relevant engineering properties and establish a data base for their eventual classification, as part of regional codes. This is the first primary objective of this study.

Desert and beach dune sands samples were collected from ten (10) different locations in the eastern province of Saudi Arabia. These samples were carefully retrieved from areas having abundant sands that can be used in construction. The samples were characterized and their geotechnical properties were determined. Results revealed that most of these sands are classified as poorly graded and all sands show somewhat similar behavior.

Marls are considered the best construction material in eastern Saudi Arabia. However, the extensive use of marl has depleted local supplies. Such a situation has necessitated the search for alternative material sources. Therefore, blending sand with marl in construction will result in an alternative material, that can be treated or stabilized to improve its engineering properties. This will save the good quality aggregate for future construction projects. In addition, there is a potential need in stabilizing such blends using chemicals such as Portland cement or bitumen (emulsified and cutback asphalts) that result in a material with acceptable engineering characteristics and are economically advantageous. This constituted the second primary objective of this study.

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ملخص الرسالة

الاسم: خالد بن عبد المحسن بن إبراهيم القنيان.
عنوان الرسالة: تقييم الخواص الجيوتقنية لغلطات الرمل و المارل المعسنة في الاستخدامات الإنشائية.
التخصص: هندسة مدنية (جيوتقنية).
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إن استخدام رمال شرق المملكة العربية السعودية في الأعمال الإنشائية يساعد في السيطرة على حركة وانتقال تلك الرمال ومن ثم تقليل أخطارها و خصوصاً في موسم هبوب الرياح. وهذه الرمال قد لا تكون صالحة للاستخدامات الإنشائية في حالتها العادية، ولكن بتحسينها يمكن استخدامها في كثير من الاستعمالات الإنشائية. لذلك فإن هناك حاجة لإجراء بحث مخبري موسع لمعرفة الخصائص الجيوتقنية لرمال الكثبان ، ومن ثم تصنيفها وتحديد المشاكل المتعلقة باستخداماتها في الأغراض الإنشائية و تحديد افضل السبل لتحسينها حتى يتم استخدامها في المشاريع الهندسية المختلفة للحصول على أفضل النتائج.

وقد تم جمع عينات رمال الكثبان بعناية من عشرة مواقع مختلفة في المنطقة الشرقية . يتوفر الرمل بها بكميات تجارية و يمكن استخدامها كمصدر للأغراض الإنشائية. و بعد تحديد الخواص الجيوتقنية المميزة لكل منها، اتضح أن أغلب هذه الرمال متشابهة الى حد كبير في خواصها الهندسية و لا يمكن استخدامها بوضعها العادي كمواد أساس.

و تعتبر التربة الجيرية المعروفة ب(المارل) من أفضل أنواع التربة المستخدمة في الأعمال الإنشائية في المنطقة، ولذلك استخدمت بشكل كبير و مكثف في المشاريع الهندسية المختلفة، مما أدى الى نفاذ كميات كبيرة منها. و بناءً على ذلك تمت دراسة الطرق الميكانيكية و الكيميائية لخليط الرمل و المارل مع إضافة بعض المعونات كالأسمنت و الإسفلت كبديل مناسب و غير مكلف للاستخدامات الإنشائية في المستقبل . وقد دلت النتائج على أن رمال المنطقة الشرقية متشابهة الخواص و أنها ذات تدرج حبيبي سيء . كما دلت أيضاً على كفاءة خليط الرمل و المارل المعسن بالإسمنت وأنه يفي بمتطلبات القوة و الديمومة (المائة) مع كونه قليل التكاليف . وعلى العكس من ذلك فإن خصائص خليط الرمل و المارل المعسن بالإسفلت رديئة مما أدى إلى رفضه .

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CHAPTER 1

INTRODUCTION

1.1 Background

Sands are found in many places of the world where arid or semi-arid conditions prevail; a typical example is the Arabian Peninsula. Approximately, one third of the Arabian Peninsula is covered by sand in one form or another. A major part of the Arabian Peninsula is covered by windblown fine silty sand dunes with varying thickness. The primary source of most of the sand is the large granite batholiths underlying the Arabian Shield (Al-Sayari and Zoti, 1978). Natural sands may exist in uncemented or cemented conditions, with various degrees of cementation. Therefore, the sand, although cohesionless, may have slight interparticle cementations in some areas.

The use of abundant sands in construction helps control sand movements which are known to result in many hazards, especially during windy seasons (Watson, 1985; Stipho, 1992). Typical problems associated with the abundance of sands in arid and semi-arid regions can be summarized as continuous sand movement, ground surface erosion and dust clouds (Stipho, 1992). Sand particles movement can also reduce visibility, cause abrasion of natural and man-made structures and may bring about the wholesale movement of dunes (Watson, 1985). These sands can not be used as a foundation material in their natural condition. However, when improved, they can be utilized in many applications (Al-Sanad et al., 1993).

Marl is available in the Arabian Gulf region including the Eastern Province of Saudi Arabia. The formation of marl soils is believed to be a result of physical and chemical weathering of the parent carbonate rocks (e.g. limestone, dolomites, carbonate sandstone, etc.) (Akili, 1980). Marl, as designated in Saudi Arabia, is essentially soft limestone contaminated with varying amounts of clay. In eastern Saudi, marls have been extensively used as backfill materials behind retaining walls and in base-course construction of roads and highways. Past experience revealed that marl exhibits wide variations in terms of origin, color, mineral composition, plasticity and other engineering properties (Aiban et al., 1999; Qahwash, 1989) and thus their behavior. Previous treatments of marl (Abduljawad and Al-Abdul Wahhab, 1989; Al-Abdul Wahhab and Abduljawad, 1989) did not contribute significantly to a better understanding of their characteristics, particularly the expansive nature of some cohesive marl types, the water sensitivity of marls, and the variation in the characteristics of these soils and their associated problems.

In the Eastern Province of Saudi Arabia, difficulties in obtaining good quality soils for different projects necessitate the search for alternatives whereby some of the locally abundant soils can be treated or stabilized to improve their engineering properties. Sand and marl are the most predominant types of soil in the region. Stabilization of sand and marl, for example, improves their load-bearing capacity and provides good quality soil that can be used for highways, runways, embankments and shallow foundations. Marls are notably used in construction due to their relatively good performance, particularly when used above the ground water table. Roads, where marl has been used

as a base course, performed well in many areas when water was prevented from reaching the soil. However, quick deterioration of paved roads has frequently been observed when the water reaches the base. The time required for the deterioration is highly dependent on the marl type, environmental conditions and the magnitude and frequency of loading, in addition to the design and construction quality (Aiban et al., 1999).

1.2 Problem Statement

During the last two decades, Saudi Arabia as well as the other Arabian Gulf States have witnessed a period of extensive construction programs at a remarkable rate. The lack of good quality aggregate has resulted in the use of marl soils as a backfill material for retaining walls, foundations and in base-course construction of all roads, highways, parking lots and airports throughout the region; in spite of the published information, which indicates that marls exhibit a great deal of variation in their origin, texture, color, mineral composition, plasticity and engineering properties (Aiban et al., 1999; Qahwash, 1989 and Akili, 1980). Extensive alligator cracks are frequently observed even in lightly-trafficked and newly-constructed roads, especially when marls are used as a base material in areas where the water table is shallow. The occurrence of such a problem indicates a substantial reduction in the bearing capacity of the supporting layers (Aiban, 1995).

In the Eastern Province of Saudi Arabia, sand is the most abundant soil type where the area is confined by deserts from three directions. There are, principally, two types of sands: Dune sand and beach sand; both are somewhat eolian in nature. Despite the efforts that have been made to understand their origin, movement and environmental

impact; there is, however, scarcity in the literature concerning their identification, quantification and the geotechnical characteristics (Al-Sanad, 1993). Consequently, there is a need to conduct an investigation on sands that will provide laboratory test results leading to the development of regional codes that will help delineate their relevant engineering properties and establish a data base for their eventual classification. Such information is also considered beneficial for soil stabilization and ground improvement.

Due to the difficulties in obtaining good quality soils for different projects (Aiban, 1994b), and since the extensive use of marl has depleted the local resources (Qahwash, 1989), there is an urgent need to stabilize and utilize the locally available marginal soils. Moreover, there are speculation and wide discrepancies in the behavior of marl soils in their untreated conditions (Aiban, 1995). Therefore, it seems appropriate to consider the potential of large-scale use of dune sands blended with marls in construction to reduce cost and environmental hazard and to save the good quality aggregate for future construction projects.

Because of the abundance of sand and marl and their associated environmental impacts, there is a potential need to stabilize these soils by mechanical, physical and chemical means to improve their load-carrying capacity and provide good quality soil that can be used for constructional purposes. Although there are previous works performed on the stabilization of sand and marl soils individually, there was no attempt to stabilize premixed sand-marl mixtures which may be considered a suitable and better alternative construction material. Stabilizing such mixes with traditional or novel treatment techniques using cement and/or bitumen will result in a material with

acceptable engineering characteristics and are economically advantageous. This trend conforms with the Saudi Arabia Sixth Development Plan (1995-2000) objectives where both cost and quality have become a major concern.

1.3 Objectives

The following are the primary objectives of this research:

- i) To characterize sands samples obtained from various locations in the Eastern Province of Saudi Arabia. The main task is to sufficiently define their properties.
- ii) To evaluate the geotechnical properties of blends of sand and marl, where one typical dune and one typical beach sands are mixed with one marl (of medium to poor quality) at different ratios. The aim is to investigate the effect of sand content on the bearing characteristics of the mixes using California bearing ratio (CBR) as the cardinal strength parameter. Such blends may provide a suitable soil source for future projects.
- iii) To stabilize the optimum sand-marl mix obtained from part (ii) using portland cement, emulsified asphalt and cutbacks. The performance of the stabilized mix is to be evaluated taking into account the effect of different parameters on the strength and durability; such as curing regime, curing temperature, curing time and wetting and drying cycles. In the case of cement treated-mixes, the effects of these parameters on the strength and durability are to be investigated using California bearing ratio (CBR), unconfined compressive strength (UCS), resilient

modulus (M_R), and durability (ASTM D559 and Modified Slake Durability). However, in bitumen-treated mixes, the strength, stiffness and durability are to be investigated using marshal stability, creep, resilient modulus (M_R), and California bearing ratio (CBR).

1.4 Thesis Organization

In order to accomplish the above-stated objectives, a thorough survey of the published literature has been conducted to provide a theoretical basis for this research work. Literature review is presented in Chapter two. It mainly focuses on sand and marl soils and the previous experimental results related to their stabilization with different additives. Chapter three, devoted to the experimental program, presents the laboratory testing program and discusses the procedures and methodologies of the different testing techniques carried out during the experimental part. Chapter four, entitled results and discussion, outlines the results obtained from the laboratory testing program and discusses and explains these results thoroughly. Lastly, Chapter five summarizes the main conclusions that can be derived from the findings of this research program. It also gives some recommendations for future work in this field.

CHAPTER 2

LITERATURE REVIEW

2.1 Sands

Approximately one third of the Arabian Peninsula is covered by sand. Being confined by deserts from three directions, all eastern Saudi soils are intercalated one way or another by sand, particularly at the surficial layers. This sand, although cohesionless, may have slight interparticle cementations in some areas. There are, principally, two types of sandy soils: Dune sand and beach sand; both of which are somewhat aeolian in nature.

Geographically, the sand dunes in the Arabian Peninsula are broadly divided into three major zones. The great Nefud in the north (57,000 km²) links to Ar-Rub Al-Khali (the Empty Quarter) in the south (600,000 km²) through the arched Ad-Dahna that runs in a north-direction extending nearly 1300 km. Figure 2.1 shows the major sand terrains within the Arabian Peninsula. The sands of Ar-Rub Al-Khali, the great Nefud and Ad-Dahana are medium to fine and bright red-orange in color due to a coating of iron oxide on the quartz grains (Al-Sayari and Zotl, 1978). On the other hand, Al-Jafurah sands are buff to tan in color (faint yellow) due to the presence of carbonates. The primary source of most of the sands is the large granite batholiths underlying the Arabian

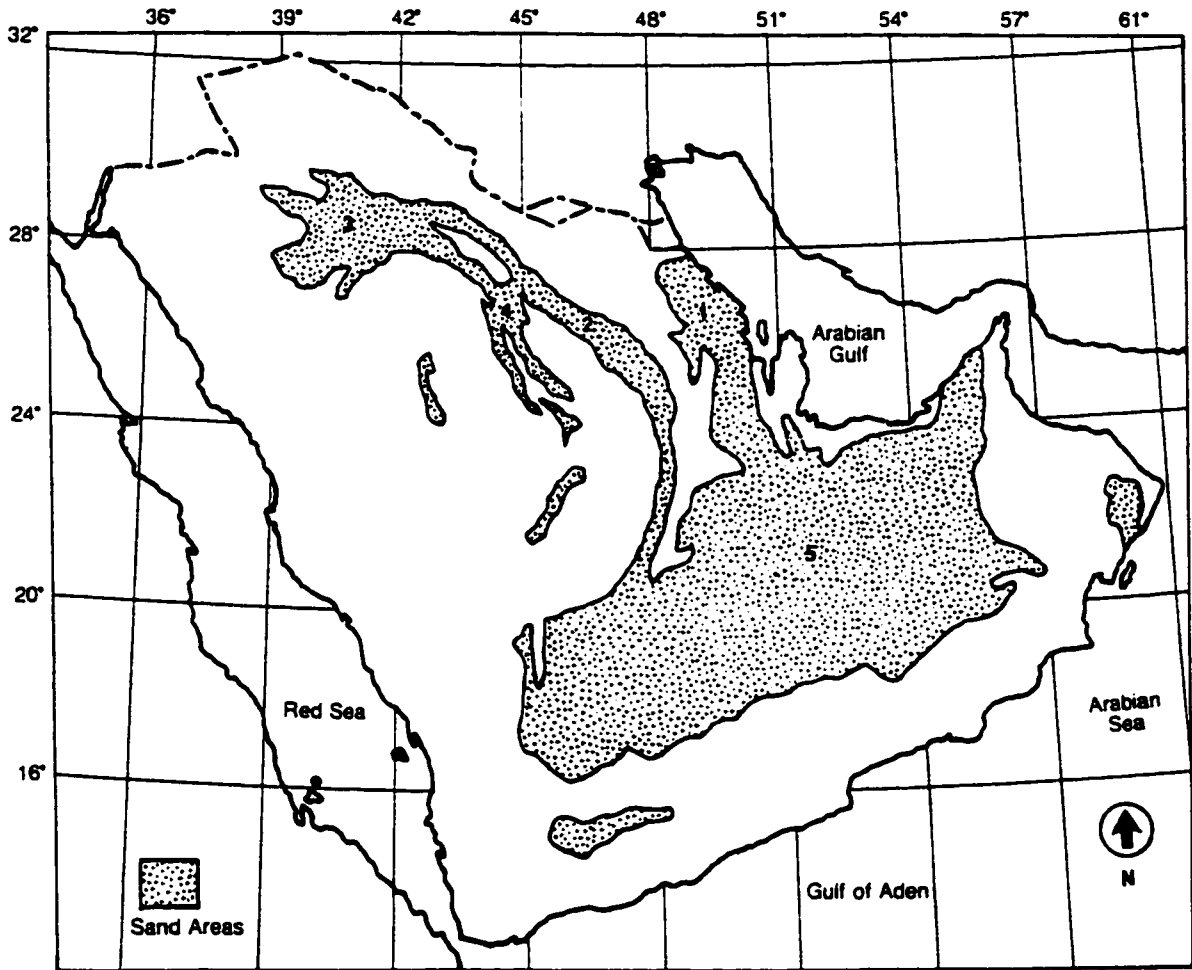


Figure 2.1: Sand Terrains in the Arabian Peninsula: 1. Al-Jafurah, 2. Ad-Dahna, 3. The Great Nefud, 4. Nefud Ath-Thuwayrate and As-Sirr, and 5. Ar-Rub'al-Khali (Ministry of Communication, 1978)

Shield (Al-Sayari and Zotl, 1978). In these regions where unconsolidated surface sediments are mobilized by wind, problems associated with drifting sand and moving dunes are severe. In eastern Saudi Arabia, annual drift rates could reach 30 m³/m width and dunes have a yearly average rate of movement of 15 m (Watson, 1985). The majority of desert soils are granular, and their engineering behavior is directly related to their gradation (Fookes, 1978).

2.1.1 Aeolian Deposits in the Arabian Peninsula

Aeolian deposits cover more than 35% of the Arabian Peninsula surface. These deposits can further be subdivided into the following four main types, depending on their mode of occurrence and genesis (Al-Sanad et al., 1993; Chapman, 1978):

a) Sand Sheets

Sand sheet can be defined as a thin accumulation of coarse sand or fine gravel consisting of grains that are too large to be transported by saltation. Particles are characterized by an extremely flat or plain-like surface broken only by small sand ripples (Bates and Jackson, 1980). These sheets or layers were formed either by the coalescence of small sand dunes and drifts, or by free deposition of sand by wind in a broad depression. Their development is mainly attributed to the distribution of a protective layer of grit on the sand sheet surfaces. Generally, sand sheets are smooth, rugged, vegetated, and very active depending upon the prevailing wind direction, vegetation and type of sand.

b) Sand dunes

Sand dune can be defined as an accumulation of loose sand, heaped up by the wind and commonly found along low-lying seashores above high-tide level as well as in various desert regions, where there is abundant dry-surface sand during some part of the year (Bate and Jackson, 1980). Various types of sand dunes occur as isolated, sporadically-distributed dunes or they may be clustered in the form of dune belt with their axes oriented in a certain direction. They are composed of yellowish to reddish medium-grained sand.

c) Sand drifts

Sand drift is a general term for surface movement of wind-blown sand, occurring in deserts or along the shore (Bates and Jackson, 1980). They are either fixed or mobile drifts. Fixed sand drifts are dome-shaped sand mounds commonly fixed by vegetation. They are composed of fine to silty sands and are mostly gypsiferous. On the other hand, mobile sand drifts are longitudinal in shape and normally composed of coarse sand particles. Sand drifts are usually smaller than sand dunes.

d) Aeolian wadi fill sand deposits

They occur as sand sheets, mostly developed by the coalescing anchored vegetated sand drifts. Most of the wadi fill deposits are of aeolian origin. The wadis act as sand traps because of their orientation which is generally transversed to the prevailing wind direction.

Geologists recognize the following four classes of eolian sand terrains in the Arabian Peninsula (Hötzl et al., 1978):

- (1) **Transverse:** Predominantly simple and compound barchan dunes in the areas of more mobile sand and/or simple rounded ridges, oriented transverse to the prevailing wind direction.
- (2) **Longitudinal:** Primarily dikakah (bush- or grass-covered sand) and various types of undulating sand sheets, which are generally characterized by elongation of the individual forms parallel to the prevailing wind direction, and often partly stabilized by sparse vegetation.
- (3) **'Uruq:** Various forms of long, nearly parallel, sharp-crested narrow sand ridges and dune chains separated by broad sand valleys usually including elements of transverse sand terrain. These forms are the resultant of a system of two dominant wind directions and are identical to the "sayf" dunes of North Africa.
- (4) **Sand mountains:** Dominated by large and massif mountains of sand, commonly cresting 50 to 300 meters above the substratum, often with superimposed dune patterns consisting of various types of complex barchans. Common forms are giant barchans spanning several kilometers from horn to horn, giant sigmoidal and pyramidal sand peaks as well as other less common peak forms, and giant oval to elongated sand mounds.

2.1.2 Mineralogical Composition of Sands

Aeolian sands are made up, mainly, of quartz that is composed of silica tetrahedral grouped in such a way as to form spiral, with all the tetrahedral oxygen's bonded to silicon. Quartz is an oxide, and thus has no weakly bonded ions in its structure, and has high hardness. These factors make it the most stable, the most common among rock-forming minerals and thus becomes abundant (Mitchell, 1993), because of its frequent occurrence in crystalline rocks as primary crystals already close to sand size (Al-Sayari and Zotl, 1978). Al-Salloum (1973) mentioned that Bagnold (1971) relates the predominance of quartz in dune sand to its ability to survive under the different erosive agents.

Al-Salloum (1973) identified the minerals forming a typical Yuma dune sand of Arizona, USA, using X-ray diffraction and found that 74% of the mineral composing the Yuma dune sand is crystalline silica (quartz), feldspar was also found and minerals such as mica, chlorite and clay were traced. Furthermore, he mentioned that Bagnold (1933) indicated that the sands of the Libyan dunes are almost 100% quartz. Mattox (1954) and Cressey (1928) found that quartz constitutes approximately 99% for the dune sands of northwestern Florida and more than 90% quartz for the dune sands of southeastern shore of Michigan Lake, respectively. Dunn and Salem (1971) carried out a study using Leighton Buzzard sand, and they found that quartz constitutes about 99% of the sand.

Al-Tayyib et al. (1985) have studied various types of sands from 58 locations within Saudi Arabia. Chemical as well as mineralogical analyses were performed for all samples and indicated that the silica contents in the Eastern Province sands ranged from 61.40% to 98.19%. In addition to quartz, calcite with minor components of feldspar, dolomite, chlorite and hornblende were present. Similar findings have also been reported for the Central and Western Provinces.

Al-Sanad (1987) has carried out a comprehensive evaluation of the subgrade soils in Kuwait for pavement design and evaluation. Mineralogical analysis using X-ray diffraction showed that quartz is the main mineral in dune sands (67-89%). Trace of feldspar was found, calcite and clay mineral were rarely found. Similar results were also reported on a study performed by Al-Sanad et al. (1993) to determine the dune sand properties in Kuwait.

In general, the minerals occurring in the fine fraction of the sands are frequently containing sodium chloride, gypsum, carbonates and iron oxides, normally coating every grain of sand irrespective of their size and shape. The coating, by different oxides, leads to various colors of dunes ranging from almost white over yellow to a pale red (Al-Sayari and Zoti, 1978).

2.1.3 Gradation of Sands

While sand may be synonymous with the topological environment of Saudi Arabia, the bulk of it falls short of good gradation and quality, making it unattractive for commercial exploitation as well as for use in routine concrete construction (Maslehuddin et al., 1991). Most, if not all, dune and beach sands are classified as poorly graded sands (SP) according to the Unified Soil Classification System and mostly (A-3) according to the AASHTO Soil Classification System.

Mattox (1962) stated that all previous researches regarding the size characteristics of dune sands have revealed that these deposits usually consist of grains smaller than 2 mm and larger than 1/32 mm, with the vast bulk of the material limited to the size range of 1/2 mm to 1/8 mm. He stated that his results did not differ from the previous analyses in any significant respect.

Dune sands commonly have specific gravity ranging between 2.6 and 2.8 and median grain diameters between 0.2 and 0.4 mm and particles size ranging between 0.1 and 0.7 mm (Fookes, 1978). Sands in the Eastern Province have fineness modulus median of 1.8 and ranging between 0.9 to 3.0 (Al-Tayyib et al., 1985). The coefficient of uniformity C_u ranges between 2.0 and 4.0 (Akili, 1983; Al-Abdul Wahhab et al., 1988; Al-Sanad et al., 1993; Hötzl et al., 1978). Bader et al. (1994) reported that sand of the barchan dunes (cressentic and whale-back) is mainly fine-grained, while that of dome and parabolic dunes is fine- to medium-grained.

2.1.4 Sand as a Construction Material

Dune sands are often difficult to compact as a fill material, even after careful laboratory tests, due to the poor gradation and the lack of fines. However, they are generally not collapsible. On the other hand, wind-blown sand sheet (aeolian) deposits normally exist in a rather loose state and often mixed with soluble salts that typically act as cementing agents (Rahim, 1989a). These conditions have rendered these soils collapsible characteristics upon inundation; as evidenced from some field and laboratory data presented by Rahim (1989a), Touma et al. (1989), and Swan (1989). When dry, however, the aeolian deposits are relatively incompressible with an appreciable shear strength (Fooks, 1978; Rahim, 1989b). These sands can not be used as a construction material in their natural condition, however, when improved they can be utilized in many applications (Al-Sanad et al., 1993; Aiban, 1994b).

2.1.5 Behavior of Cemented Sands

Cemented sands are found in many places of the world and the degree of cementation varies from strong to very weak. The presence of cementation in these sands manifests itself in their ability to stand in very steep slopes and their high resistance during pile driving. Cemented sands may be divided into two categories namely: naturally-cemented and artificially-cemented.

Interest has grown in exploring the properties and behavior of cemented sands which possess distinctive geotechnical properties that are quite different from other soils. A good understanding of the behavior of these soils is necessary for foundation design, pile driving slope stability analysis, earthworks, and highway construction. Naturally-cementing agents include principally: carbonates, hydrous silicates, iron oxide, and gypsum at a rate depending on the local conditions and the geologic history of the deposits.

On the other hand, the artificially-cemented sands are produced in the laboratory or in the field. The literature reveals that artificially-cemented sands were used in the laboratory in order to study the behavior of naturally-cemented sands. The testing of artificially-cemented samples is much easier than that of block sampling operations associated with naturally-cemented sands. In addition, the laboratory samples allow greater control over the important variables that are normally difficult to assess in naturally-cemented sands because of their heterogeneity, variation in cement content, and the difficulty normally encountered in sampling naturally-cemented sands (Clough et al., 1981).

Among the agents that have been used for artificial cementation are portland cement, asphalt-cutback, emulsified asphalt, lime, sulfur and acids at different percentages. The soil-cement mixture is designed for maximum strength and durability,

and it is tailored to the particular construction use and soil type. Table 2.1 summaries some of the cited information on the geotechnical properties of uncemented, naturally-cemented and artificially-cemented sands.

Results from Table 2.1 reveal that the work done by Mitchell (1976) on sands treated with portland cement showed that the unconfined compressive strength of sand-cement mixture was high. In addition, the friction angle was slightly high. The behavior of an artificially-cemented sand was studied by Sitar (1979) in which 2% and 4% portland cement were used. Results show that the cemented sands are brittle at low confining pressures and become increasingly more ductile with increasing the confining pressure. Similar results were revealed from the work of Clough et al. (1981). They noted that the stiffness and strength increase with the increase in confining pressure. Furthermore, the friction angle and cohesion intercept are increasing with the increase in relative density; which reflected that density, packing, or both, have a significant influence on the strength of cemented sand, in addition to the nature of the cementing material.

Aiban (1985 and 1994b) found that the addition of a small amount of portland cement (2%) to sand results in cementation bond that resulted in a cohesion intercept which has a noticeable effect on the stress-strain and strength. His results have also shown that the sample's strength increases with increasing the cure period, confining pressure, cement content, and density.

Table 2.1 : Summary of Cited Information on Naturally and Artificially Cemented Sands

REFERENCE	Sitar (1979)	Clough et al. (1981)	Aiban (1985)	Aiban (1985)	Molenaar and Venmans (1993)
SOIL TESTED	Artificially Cemented Sand	Artificially Cemented Sand	Artificially Cemented Sand	Artificially Cemented Sand	Artificially Cemented Sand
CEMENTING AGENT	Portland Cement	Portland Cement	Portland Cement	Portland Cement	Calcium Carbonate
SAMPLE TYPE	Compacted in molds	Compacted in molds	Compacted in molds	Compacted in molds	Compacted in molds
TYPE OF TESTS	Static Triaxial; Unconfined Comp.; Brazilian Tension; Simple Shear	Unconfined Comp.; Brazilian Tension; Drained Triaxial	Unsaturated Undrained Drained Static Triaxial	Unsaturated Undrained Drained Static Triaxial	Unconfined Comp.; Triaxial compression; Oedometer
STRESS-STRAIN CURVE PRESENTED	Yes	Yes	Yes	Yes	Yes
INT. FRICTION ANGLE (degrees)	34 - 35	29 - 41	41	Not Available	Not Available
COHESION (kPa)	46 - 143	46 - 152	60.34	Not Available	Not Available
DRY DENSITY (kN/m ³)	15.5 - 15.8	14.6 - 16.9	17.2 - 17.5	17.25 - 17.67	17.0 - 19.4
WATER CONTENT(%)	8	8	7.2	0.28 - 0.44	0
UNCONFINED COMPRESSION (kPa)	146 - 474	180 - 670	Not Available	Not Available	252 - 6294
PEAK STRENGTH (kPa)	474 - 1609	1200 - 1600	534 - 964	592 - 678	Not Available
CONFINING PRES. (kPa)	0 - 414	414	69	69	Not Available
APPROX. STRAIN AT FAILURE(%)	0.52 - 2.85	0.37 - 5.07	0.57 - 0.96	0.61 - 0.74	0.03 - 1.89
COMMENTS	2% & 4% Portland Cement were used Dr = (74%) Ave Tensile Strength = (20 - 69 kPa)	2% & 4% Portland Cement were used Dr = (60 - 90 %)	1-4%Portland Cement Were used; Samples cured in wax for 14 days	2% & 4% calcium Carbonate were used; Samples cured in wax for 7 days and out of wax for 14 days	Two artificially Cemented sands with 9.7 - 38.9% Calcium Carbonate were used

Table-2.1 : Continued (a)

REFERENCE	Alfi (1978) *	Bachus (1978) *	Salamone et al. (1978)	Saxena and Astrico (1978)	Korbin and Brekke (1975)	Mitchell (1976)
SOIL TESTED	Naturally Cemented Sand	Naturally Cemented Sand	Naturally Cemented Sand	Naturally Cemented Sand	Artificially Cemented Sand	Artificially Cemented Sand
CEMENTING AGENT	Carbonate & clay	Carbonate & clay	Carbonate	Carbonate	Shaping Wax	Portland Cement
SAMPLE TYPE	Hand trimmed	Hand trimmed	76 mm Sampler	76 mm Sampler	Compacted in molds	Compacted in molds
TYPE OF TESTS	Drained static Triaxial	Drained Static Triaxial; Indirect Tension	Isotropically Consolidated cyclic Triaxial	Isotropically Consolidated Undrained Static Triaxial	Static Triaxial; Indirect Tension	Unconfined compress.; Static triaxial; Indirect Tension flexure
STRESS-STRAIN CURVE PRESENTED	Yes	Yes	No	Yes	Yes	Partial
INT. FRICTION ANGLE (degrees)	48	39 - 42	37 - 39	37 - 39	11.5 - 35	30 - 45
DRY DENSITY (kN/M3)	17.8	16.0 - 17.1	11.8 - 15.7	11.8 - 15.7	16.7	Not Available
WATER CONTENT(%)	10.6	3.8 - 18.5	20 - 40	20 - 40	Not Available	Not Available
UNCONFINED COMPRESSION (kPa)	2700	60	Not Available	Not Available	337	1000 - 15000
APPROX. STRAIN AT FAILURE(%)	1.5	0.6	Not Available	2.0 - 23.5	0.6	0.35 - 3.0
COMMENTS	Dynamic test not done; high static strength	Dynamic test not done;	Stress-Strain Not presented; Unconfined Comp. Strength unknown; Effect of sample Disturbance unknown	Stress-Strain unlabaled; Unconfined Comp. Strength unknown; Effect of sample Disturbance unknown	Dynamic test not done; Soil has time dependent Response	Data is mostly in generalized form; No Dynamic Data; Post Failure in stress-strain data not Available

Table-2.1: Continued (b)

REFERENCE	Ismael et al. (1986a)	Ismael et al. (1986b)	Khan et al. (1988)	Al-Sanad et al. (1993)	Sitar (1979)	Clough et al. (1981)
SOIL TESTED	Naturally Cemented Sand	Naturally Cemented Sand	Naturally Cemented Sand	Uncemented Dune Sand	Uncemented Sand	Naturally Cemented Sand
CEMENTING AGENT	Carbonates & Sulphate	Carbonates & Gypsum	Clay & Silt	None	None	Silicates & Iron oxides with minor Carbonate
SAMPLE TYPE	Hand Trimmed	Hand Trimmed	Statically Compacted	Bulk Sample	Not Available	Hand Trimmed
TYPE OF TESTS	Direct Shear Unconfined Compress.	Direct Shear	Unconsolidated Undrained Triaxial (UU)	Direct Shear; Consolidation	Static Triaxial	Unconfined Comp.; Brazilian Tension Drained Triaxial
STRESS-STRAIN CURVE PRESENTED	Yes	Yes	Yes	Yes	Yes	Yes
INT. FRICTION ANGLE (degrees)	40 - 54	40.4 - 41.7	14 - 43.5	42	34	37 - 49
COHESION (kPa)	10.0 - 50.0	0 - 24	0 - 100	0	5.5	12 - 365
DRY DENSITY (kN/m ³)	17.1 - 17.5	16.53 - 17.07	19.12 - 21.23	16.18	15.5	16.5 - 17.6
WATER CONTENT(%)	6.8 - 10	0.9 - 1.8	4 - 13.5	0.11 - 0.45	Not Available	5.0 - 15.0
UNCONFINED COMPRESSION (kPa)	83 - 2900	Not Available	Not Available	---	---	50 - 1930
PEAK STRENGTH (kPa)	85 - 200	40 - 60	480 - 1050	30 - 110	103 - 1068	1400 - 4800
CONFINING PRES. (kPa)	100	50	100 - 140	27 - 111	35 - 414	414
APPROX. STRAIN AT FAILURE(%)	0.08 - 0.23	0.1	2.0 - 15	Horz Dis (1.8 - 3.3 mm) Vert. Disp (0.20 mm)	5.0 - 8.50	1.7 - 8
COMMENTS	Three Different Naturally Cemented Soils were Tested	Four Different Naturally Cemented Soils were Tested	Control Strain Type; Sr = (50 - 100 %) DC = (90 - 100 %)	Dune Sand were taken from four Sites; Dr = (62 %)	Residual Int. Friction Angle = 30.5 Degrees	Four Different Naturally cemented Soils were Tested

Table-2.1 : Continued (c)

REFERENCE	Ismael and Al-Sanad (1993)	Acar et al. (1986)	Wayne et al. (1981)	Surendra et al. (1977)
SOIL TESTED	Naturally Cemented Sand	Artificially Cemented Sand	Artificially Cemented Sand	Naturally Cemented Sand
CEMENTING AGENT	Carbonates, Gypsum and Iron Oxide	Portland Cement 1, 2 and 4%	Silicate, Iron Oxide and Portland Cement 2 and 4%	Calcite
SAMPLE TYPE	Hand Trimmed	Lab Prepared	Hand Trimmed and Lab Prepared	76 mm Sampler
TYPE OF TESTS	Simple Shear SPT and Plate Bearing Test	Resonant Column Equipment Triaxial Mode and Unconfined Compression	Unconfined Drained Triaxial Test and Tensile Strength	Consolidated Undrained Triaxial compression Test
STRESS-STRAIN CURVE PRESENTED	Yes	Yes	Yes	Yes
INT. FRICTION ANGLE (degrees)	35	33 @ 0% Cement 39 @ 2% Cement	29 - 49	38 - 40
DRY DENSITY (kN/m ³)	1.81 - 1.99	Relative 31 - 90%	Relative 60 - 90%	13.34 - 17.23
WATER CONTENT(%)	1.4 - 4.4	Not Available	Not Available	2.7
UNCONFINED COMPRESSION (kPa)	Not Available	0 - 77	Not Available	0 - 2000
APPROX. STRAIN AT FAILURE(%)	Not Available	Not Available	Not Available	2 - 23.5
COMMENTS	Bearing Capacity Nat. Ground = 1300kPa Soak. Ground = 600kPa Comp Ground = 420kPa SPT Range = 15 - 35		Tensile Strength vary between 0 and 70 kPa	Deviator Stress at Failure = 124 - 159 kPa Confining Pressure 62 - 83 kPa

2.2 Marl Soil

2.2.1 Background

The Arabian Peninsula is partly covered by carbonate rocks and soils. It is lying between Africa and Western Asia, and can be divided into two structural provinces, namely the Arabian Shield which is located on the west and the Arabian Shelf towards the east (Al-Sayari and Zotl, 1978). Generally, the Arabian Shield is part of the Precambrian crustal plate, locally covered by Tertiary volcanic rocks at some places. In the Arabian Shelf, the plate is covered by thick sedimentary sequences. The Eastern Province of Saudi Arabia is a part of the Arabian Shelf. The carbonate rocks and soils in the area is a result of sedimentation processes which prevailed in the area over millions of years.

The eastern edge of the Arabian Peninsula, from the southern end of the Arabian Gulf northwestward through Iraq and into Syria, lies in a rectangular depression formerly known as Mesopotamia (Al-Sayari and Zotl, 1978). The southern part of the depression includes the western half of the Arabian Gulf and a narrow strip of the Arabian Peninsula, known as the Arabian Gulf coastal region. The surface rocks of this part of Saudi Arabia include consolidated sedimentary formations ranging from Paleocene to Middle Eocene age, and Miocene to Pliocene age. Unconsolidated materials of Tertiary age and sediments of Quaternary age are also present. The surface formations of the region are shown in Table 2.2 and Figure 2.2.

Table 2.2 : Geological Formations of Eastern Saudi Arabia (after Roger, 1985)

Formation	Lithology
Hofuf	CONGLOMERATE: Red and white conglomerate; boulders and pebbles of limestone in quartz matrix. Basalt unit greenish-gray to red, in part sandy marl.
Dam	MARL, CLAY, and LIMESTONE: Upper part red to green partly sandy and silty clay with minor marl and sandstone beds and buff to white limestone with yellow, fossiliferous limestone and sand; lower part white very calcareous fossiliferous marl; minor sand and clay.
Hadruk	SHALE, SANDSTONE, and MARL: Green and grayish-green generally finely sandy clay and green to gray calcareous sandstone; commonly weathers to fine concretionary pellets. Minor amounts of cream to gray marl and gypsum are also present. Chert occurs at a number of levels. Basal 5-10 m commonly cream sandy limestone or marl.
Dammam	<p><u>Alat</u>: LIMESTONE and MARL: Upper part light-colored chalky porous commonly dolomitic limestone; abundant molds of mollusks and other organic remains. Lower part light-colored dolomitic marl.</p> <p><u>Khobar</u>: LIMESTONE and MARL: Light- to dark-brown, in part dolomitic limestone becoming off-white soft marly limestone and marl in lower part.</p> <p><u>Alveolina Limestone</u>: LIMESTONE: Tan limestone.</p> <p><u>Shaila Shale</u>: SHALE: Poorly exposed yellow-brown shale.</p>
Rus	<p>LIMESTONE: White soft chalky porous limestone; calcarenite beds at top.</p> <p>MARL and LIMESTONE: Light-colored marl; local irregular masses of crystalline gypsum, occasional thin limestone beds and geodal quartz at several levels. In other areas, unit is highly variable, including as common equivalents: (a) white compact finely crystalline anhydrite with interbedded green shale and minor amounts of dolomitic limestone or (b) gray marl with coarsely crystalline calcite and interbedded shale and limestone.</p> <p>LIMESTONE: Gray to buff compact commonly partially dolomitized limestone; minor amounts of soft limestone made porous by leaching of small organic remains. Quartz geodes occur rarely in the lower part, and are typical of upper part.</p>
Umm er Radhuma	DOLOMITE: contains <i>Lockhartia tipperi</i> Davies.

LEGEND

- | | | | |
|---|---|---|--------------------|
|  | Eolian Sand (including Sabkha deposits) |  | Dam Fm. |
|  | Dammam Fm. |  | Hadrukh Fm. |
|  | Neogene und. |  | Umm Er Radhuma Fm. |
|  | Hafuf Fm. |  | Rus Fm. |

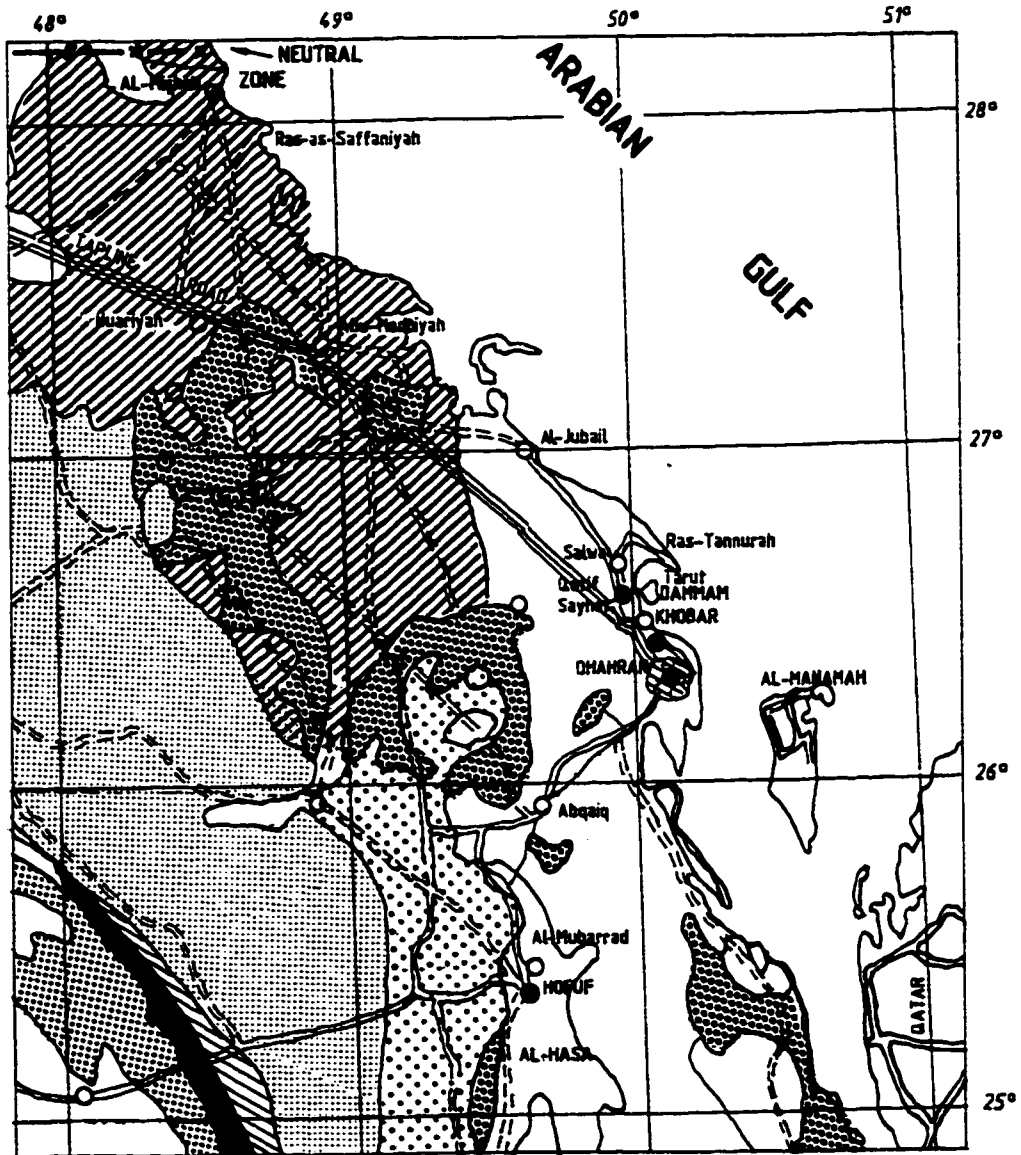


Figure 2.2: Geological Formations of Eastern Saudi Arabia (After Ministry of Agriculture and Water, 1969).

The main sedimentary rock resulting from the calcium carbonate sedimentation is limestone. The limestone is formed due to the precipitation, deposition and consolidation of calcium carbonate that can be subsequently dolomitized and therefore converted to dolomite. Limestone and dolomite represent about 20% of all sedimentary rocks and occur in all continents, in strata of every age (Encyclopaedia, 1980). Calcareous materials deposit in high energy areas while carbonate muds deposit in quiet lagoons (Al-Sayari and Zotl, 1978). The high energy area is characterized by high turbulence of sea water, keeping the fine clay particles to remain in suspension. While in the low energy areas, the clay and other sediments settle simultaneously to provide the dual character to the rock. After deposition, limestone undergoes mineralogical changes to form dolomite, a process known as dolomitization. The formation of marl soils is believed to be a result of physical and chemical weathering of parent carbonate rocks such as limestone, dolomites and carbonate sandstone (Akili, 1980).

The rock formed due to simultaneous deposition of calcareous material and clay is termed marl. There exists large differences in marl types and characteristics. Therefore, and due to the large differences in marl types, composition and characteristics, there is no standard definition for marl. Some of the definitions used by different authors are given in Table 2.3. The outcome of the different definitions used is that the marl is a mixture of calcium carbonate and clay and, therefore, the reuse of the word marl for such deposits is questionable in the absence of clay. It would be appropriate to call them 'calcitic soils' as proposed by Ahmed (1995) or would be even better to call them 'calcareous soils' to include dolomitic soils (Aiban et al., 1999).

Marls are widely distributed in many places in the Arabian Gulf region. In eastern Saudi Arabia, marls exist in Abqaiq, Dhahran, Dammam, Abu-Ali, Hofuf, Berri,

Table 2.3: Lists of some definitions for marl soil used by different authors.

Author(s)	Year	Definitions
Terzaghi and peck	1967	Stiff to very stiff marine Calcareous clay of greenish color.
Pettijohn	1975	A rock of 35% to 65% carbonate and complementary content of clay.
Saudi-Aramco	1978	Soft limestone contaminated with varying amount of clay.
Sowers and Sowers	1979	A water deposited sand with silt or clay containing calcite and sometimes colloidal organic matter.
Akili	1980	A binary mixture of calcium carbonate and clay.
Bates and Jackson	1980	An old term of loosely applied to a variety of materials, most of which consist of an intimate mixture of clay and calcium carbonate.
McCarthy	1982	A soft limestone.
Mitchell	1985	Soft calcareous clay-rich material, often barely consolidated, with or without distinct fragment of shells.
Al-Tayyib et al.	1985	Local carbonate soils, the formation of which is attributed to physical and chemical weathering of parent carbonate rocks.
Qahwash	1989	Calcareous sediments. Also mentioned that the Europeans frequently define marl as argillaceous.
Mitchell	1993	A range from relatively pure calcium carbonate to a mixture of calcium carbonate with mud and organic matter formed by biochemical process.
Aiban	1994	Fine-grained calcareous sediments
Ahmed	1995	Calcitic soils

Fadhli, Jubail, Abu-Hadriyah and Safaniyah areas (Aiban, 1995). These soils are extensively used as backfill for retaining walls, as foundations, and in base-course construction of roads and highways in the region. Marl, as designated in Saudi Arabia, is essentially soft limestone contaminated with varying amounts of clay (Saudi Aramco, 1978). Marls exhibit wide variations in terms of origin, color, mineral composition, plasticity and other engineering properties (Qahwash, 1989). Almost all types of marl are lacking hardness and resistance to disintegration in water (Bayomy, 1988). The use of marls in construction is limited due to their poor performance under harsh environmental and loading conditions. According to Fookes and Higginbottom (1975), the main variables which influence the engineering properties of calcareous soils are:

- mineral composition,
- type of carbonate minerals present, whether calcite (CaCO_3), dolomite $\text{CaMg}(\text{CO}_3)_2$, or siderite (FeCO_3),
- origin and forming process, whether formed as a result of physical or chemical weathering of parent carbonate rock,
- grain size, and
- degree of cementation.

2.2.2 Mineral Composition of Marl

There are huge variations in mineralogical composition of marl. Qahwash (1989) has carried out chemical analysis of samples obtained from three different sites in the Dammam-Dhahran region. The test results revealed that the main components are

calcite, dolomite and clay minerals. Quartz, halite and gypsum were also found. The combined amount of carbonates (calcite and dolomite) ranges from 55% to 80%.

The analysis performed by Al-Tayyib et al. (1985) indicated that marls from Abqaiq and Abu-Hadriyah contain about 93% dolomite and 5-7% quartz and feldspar, whereas, marl from Dammam-Dhahran area is composed of about 70% dolomite, 15% quartz and feldspar, and 5-10% gypsum. The analysis conducted by Aiban (1994a) indicated that Abqaiq marl has about 80% dolomite and 13% calcite.

Chemical and mineralogical analysis of two selected eastern Saudi marls namely Abu-Hadriyah and Shedgum marls were performed by Ahmed (1995). Results obtained from the chemical tests, presented in Table 2.4, showed the following observation:

- 1) Abu Hadriyah marl has both calcite and dolomite, with calcite as the primary mineral and dolomite as a trace mineral.
- 2) Shedgum marl contains high percentage of calcite, while dolomite is not present.

Whereas, the mineralogical tests results indicated that Abu Hadriyah marl has a very high percentage of calcite, dolomite is present in traces. Shedgum marl has a high percentage of calcite and quartz, while dolomite is absent.

Table 2.4: Chemical Analysis of selected marl soils (Ahmed, 1995)

Marl	Fines	Ca (%)	Mg (%)	CaCO ₃	CaMg(CO ₃) ₂
Abu-Hadriyah	-* #40	42.8	3.8	39.0	3.8
(M-AB9)	- #100	56.3	0.0	56.3	0.0
Shedgum	- #40	37.5	0.0	37.5	0.0
(M-SHD1)	- #100	62.6	0.0	62.6	0.0

*- implies passing sieve #

2.2.3 Engineering Behavior of Marl

2.2.3.1 Background

The widespread occurrence of offshore soils consisting, mainly, of calcium carbonate has led to numerous investigations regarding the behavior of carbonate sediments. The lack of knowledge about the performance of such soil has prompted research oriented toward understanding the behavior of specific sediments from areas in which offshore oil and gas explorations have led to heavily loaded foundations (Hull et al., 1988). Early recognition of the problems associated with the installation of foundations in carbonate soils came from the fact that piles placed in carbonate soils offered lower capacities compared to those in non-carbonate soils.

In addition, the evaluation of performance of these soils when used in roads have been recently performed by Aiban et al. (1999). Field trials of marl stabilization were conducted using 4% cement on a few of the main roads in Dammam, eastern Saudi Arabia. The stabilization program was carried out on severely damaged paved roads where marl was used as a base course. Although construction was properly carried out, the untreated sections deteriorated three months after construction. Comparison of the performance of the stabilized sections with that of untreated ones, where both are exposed to the same traffic volume for three years, indicates that none of the stabilized sections have shown any signs of cracking or rutting. On the other hand, the untreated ones are severely damaged and the asphalt layers are completely removed from the surfaces.

2.2.3.2 Consistency Limits

Carbonate soils are formed by the weathering of rocks containing calcium carbonate as their main constituent. The source of calcium carbonate constituting these rocks is mostly organic. Organic carbonates are skeletal remains of the microscopic plants and animals settled at the sea floor; such as pteropods, foraminifera (forams), and coccolithophorids (nannofossils or nannos because of their small size) (Demars and Chaney, 1982). These organisms live in the surface waters of the sea; and, on demise, they settle to the sea floor.

The shells or skeletons of these organisms are porous in nature and contain intraparticle water. Demars and Chaney (1982) estimated that the ratio of the volume of intraparticle water to the volume of particle solids was 5.0 for forams and 1.05 for nannos. This intraparticle water is responsible for the erroneous values of the liquid and plastic limits (LL and PL) of the carbonate soils. The water contributing towards the plasticity behavior of the soil particles is only the surface or adsorbed water portion of the interparticle water. The moisture content for a soil, by drying the sample, is the sum of the interparticle and intraparticle water. The apparent liquid and plastic limits determined for calcareous soils are always higher than their actual values. Since the difference of the two values eliminates, to some extent, the above-mentioned effect, the plasticity index (PI) is therefore not greatly affected by the presence of intraparticle water (Demars and Chaney, 1982). In addition, the shrinkage limit (SL) for calcretes is higher than that for normal soils ($SL > 20\%$) and is found to be very close to or even greater than the plastic limit (Horta, 1980). Calcium carbonate equivalent (carbonate content of the material passing the ASTM No. 40 sieve) of calcareous soils can be used

in place of PI for the construction material (Horta, 1988). It was found that PI decreases with an increase in calcium carbonate content.

The specified maximum accepted value of PI for the soil to be used for the construction of base course is 6%. However, for calcretes a value of PI higher than 6% can safely be used (Horta, 1988). The use of the above specified limit can result in discarding of good construction calcareous soils. The upper limit of PI is therefore relaxed and values up to 10 to 15% can be accepted for the use in base course construction.

Akili (1980) found that the behavior of marl, obtained from the south of Dammam city, eastern Saudi Arabia, was affected by the presence of plastic material. The plasticity in the marl was due to the presence of clay. Similarly, Qahwash (1989) found that the liquid limits and PI's of the calcareous sediments in the Dammam-Dhahran area, were attributed to the existence of some montmorillonite and/or soluble salts as displayed by the X-Ray diffraction (XRD) results.

Ahmed (1995) performed a detailed characterization on 24 marl samples collected from different sites in eastern Saudi Arabia. The mineralogical composition and the behavior of these soil samples showed the absence of clay minerals in most of these samples, while only trace of less than three percent (<3%) of clay minerals are present in few others. Some of these contain calcium carbonate as the main constituent, while dolomite is present in others as the main mineral. The plasticity was used to classify the soil. He found large variation in the Atterberg limits of these soils. These marls exhibit LL values in the range of 17 to 123 and PI of 2 to 44. Plastic soils are considered unsuitable in pavement structures, and there is always a maximum plasticity index (PI)

that should not be exceeded. He observed that the marl soils of eastern Saudi Arabia, which are plastic in nature can be classified as 'satisfactory marls'; while non-plastic marls show a complete loss of strength on the wet side of optimum or upon inundation when compacted on the dry side of optimum; and can be termed 'marginal marls'.

2.2.3.3 Grain-Size Distribution

The sedimentary carbonate rocks are formed by the deposition and the consequent cementation and induration of the calcium carbonate particles in the marine environment. The cementation is mostly due the calcite precipitation. These calcium carbonate particles are mostly of biological origin as shown in Table 2.4. The grain-size distribution of the calcareous soils depends on the degree of cementation of the soil grains (Datta et al., 1982). The cemented particles can lead to misleading grain-size analysis because the cementation is lost upon remolding and, to some extent, by the dissolution in water. This dissolution is pronounced if the water is acidic ($\text{pH} < 7$).

The grain-size distribution of carbonate soils is not representative of their engineering behavior if cementation and/or induration is not considered in the analysis (Datta et al., 1982). In the case of cemented calcareous soils, the specific gravity of the fines is higher than the coarser particles formed by the cementation of the finer particles whereby the calcium carbonate is the main cementing agent. The grain size distribution, based on weight, of calcareous soils, such as calcrete, overestimates the volume occupied by the fine particles. In addition, the washed and dry sieve analyses show large differences in the percentage of fines, the unwashed sieving being unreliable for calcareous soils. Moreover, when the fines (passing ASTM No. 200 sieve) are subjected

to the hydrometer analysis for estimating their grain-size distribution, the hollow particles float on the surface of the water, making the test unreliable for these types of soils (Datta et al.1982).

Ahmed (1995) reported that a large difference exists in the case of dry and wet sieving of the eastern Saudi marls. In the presence of some moisture, the fines form aggregations, which are responsible for the apparent coarse gradation during dry sieving. These aggregations break and transform to individual particles during wet sieving. Besides being in the form of lumps, the fines tend to coat the larger particles. On the other hand, the wet sieving provides a clear picture of all particle sizes as well as their percentages. Moreover, the fines obtained in the wet sieving are used for hydrometer analysis, as these are truly representative of the particle sizes less than 75 μm . In the case of soils having large percentage of clayey particles, wet sieving gives accurate results. However, carbonate particles are susceptible to dissolution in water and therefore, may affect the wet sieving results.

2.2.3.4 Strength

The strength of a soil is function of its structure and composition, in addition to other factors. The structure in turn depends on the particle orientation and their interaction with each other. The strength of carbonate soils is controlled by some additional factors such as grain crushing, and dissolution in pore water. The grain crushing results in a reduction of the angle of internal friction at relatively low stress levels (Ahmed, 1995). The strength of carbonate soils is also controlled by the carbonate cementation at the grain contacts. Carbonate-cemented soils show high strengths, but

dissolution of this cementation in pore water results in a complete loss of cohesion (Ahmed, 1995).

Akili (1980) performed repeated loading tests on limestone and argillaceous limestone (marl) obtained from the south of Dammam city, eastern Saudi Arabia. The results show that for stress levels above a certain critical value, plastic strains increased with repeated stress application until failure of the sample occurred. Below the critical stress, plastic deformation continued to increase only with increased number of stress cycles even at very low stress levels. For both soils, the critical stress was approximately 40% of the undrained shear strength under static loading at the same static confining pressure. Increasing the number of stress cycles led to an increase in the resilient modulus (impulse deviator stress divided by resilient or recoverable strain) of soil derived from limestone, and a decrease in the modulus of resilient of clayey soil. This indicates that the former is a strain hardening while the later is a strain softening material.

Aiban (1994a) studied the strength of a marl soil obtained from Abqaiq, eastern Saudi Arabia. The results showed that the strength of samples compacted at optimum moisture content gave higher strength compared to the samples compacted on the dry as well as on the wet side of the optimum. Aiban (1995) explained the above behavior in relation to the fabric of the soil. On the dry side of optimum, the presence of small macropeds serves as a bridge or a connector between large macropeds. These macropeds are extremely sensitive to the moisture content and become weaker upon contact with water. The availability of water during compaction is responsible for the lower strength on the wet side of optimum since the macropeds are too weak to resist the stresses imposed on them. At the optimum, the macropeds are closely packed and have regular connectors.

The engineering properties of compacted soils depend on the compaction energy and method, and the molding moisture content. Compacting a soil to the same dry density but at different molding moisture contents produces entirely different fabric and, consequently, the properties become different. The strength based on the CBR tests for 24 marl soils varies widely over a narrow range of moisture content. At or near the optimum moisture content on the dry side, the CBR values are well above 100% for all the marl soils tested, indicating very high strength. In most cases, increasing the moisture content above optimum by about 1% causes complete loss of the soil bearing strength. On the wet side of optimum, the soil forms mud-like lumps and the CBR value drops to almost zero. This behavior was observed with most of the marl samples. However, some marls showed strength and stability even on the wet side of optimum. Based on these findings, the marl samples showing poor behavior on the wet side can be classified as “Marginal marls”, while those showing strength and stability on the wet side can be termed “Satisfactory marls”. In the characterization stage, the variation of CBR with the molding moisture content was taken as an indication of the strength behavior for the collected marl samples. Marl soils, classified as A-1-a (AASHTO) or GW (USCS), show an abrupt decrease in the strength (CBR) on the wet side of optimum moisture content (OMC). On the other hand, some marls classified as A-2 showed a much better strength even on the wet side of OMC and up to 4% above the OMC (Ahmed 1995).

2.2.4 Soil Stabilization

The concept of soil stabilization has been around for several thousand years (Terrel et., 1984). In the last few decades, the art of soil stabilization became the subject

of intensive research programs and investigations. The aim of such programs is mainly to improve the engineering properties of soils, to evaluate and improve the stabilization techniques, and to quantify their advantages and disadvantages based on the observed performance. Laboratory and field testing and monitoring are employed for the evaluation. The success and economy of different alternatives are dependent on many variables, and what might be said for soils in an area is not necessarily true for other areas. The stabilization mechanism takes many forms and covers wide range of applications for different soils.

The literature presented in the last few decades reveals that most of the work performed on the stabilization of shallow grounds consists mainly of admixtures. Many additives and inclusions have been used for different applications and the most widely used ones are: (1) cement, (2) lime, (3) bitumen, (4) geosynthetics, and (5) other chemicals and inclusions. Combination of two or more of these additives or mixtures with other products such as waste materials, or soils have also been used.

Chemical stabilization consists of mixing the soil with one or a combination of chemical admixtures for the general objectives of improving or controlling its volume stability, strength and stress-strain behavior, permeability, and/or durability (Winterkorn and Pamukcu, 1991). Admixtures can be in the form of powder, slurry, or liquid. The fundamental processes that take place in a chemically-stabilized soil system are cementation and ion-exchange reactions, alteration of soil surface properties, plugging of

voids, and coating the soil particles thereby binding them together (Winterkorn and Pamukcu, 1991). The most commonly used chemical admixtures are portland cement, lime, fly ash, and bitumen. The selection of an admixture depends on the physical and chemical properties of the natural soil, workability of the mixture, economic and safety constraints, the function of the treated soil, and the conditions of construction. The water affinity and water-retention capacity, clay content, grain-size distribution, and the porosity are the soil parameters that affect the interaction of the admixture with the soil particles. Cement and bitumen (emulsion/cutback) have been used successfully for the stabilization of different soils with varying degrees of improvement. Those additives will be discussed in the following paragraphs.

2.2.4.1 Cement Stabilization

The most commonly used additive for soil stabilization is portland cement. It has been used in roads in the United States since the mid 1910's. Since then, large usage of cement-stabilized soils for airfield and highway construction have been reported. Cement stabilization of soils involves mixing of pulverized soil, cement and water. The mixture is then compacted to a high density which renders the material resistant to various loading and environmental conditions. The reaction of cement and water forms cementitious calcium silicate and alumina hydrates which bind soil particles together. The hydration releases slaked lime Ca(OH)_2 , which in turn may react with soil particles. The hydration is independent of the soil type and therefore cement can be used for a wide

range of soils. However, problems may be encountered with soils which contain excessive amount of organic matter, sulfates or salts (Aiban et al ., 1999).

Cement stabilization of soils has many advantages including the increase in strength and stiffness, and the improvement of volume stability and durability. Depending on the type of soil and the amount of cement, there are various types of cement-stabilized systems. The two major types are:

- (1) soil cement which contains sufficient cement to produce a hard and durable construction material. The resulting material has a well-defined resistance to weathering and stresses. ACI 116 R defines soil cement as a mixture of soil and measured amounts of portland cement and water compacted to a high density. Cement content is normally expressed as a percentage of the soil dry weight or volume. A typical amount of cement required for sands is between 5 and 11% by weight, for a soaked compressive strength (28 days) of 2.76 to 6.90 MPa.
- (2) cement-modified soil or simply cement-treated soil which contains smaller quantities of cement compared to the soil-cement. The construction (mixing and compaction) of cement stabilized material has been discussed in many Portland Cement Association (PCA) publications such as Soil Cement Construction Handbook (PCA, 1979). The strength of cement-stabilized soils is affected by the cement content and type, degree of compaction, moisture content and means of mixing the soils with cement, as well as the type and duration of curing. The laboratory and field testing has been discussed in many standards and publications.

2.2.4.2 Bituminous Stabilization

Bituminous stabilization refers to the methods in which bituminous materials are added to a soil or a soil-aggregate mixture. Bituminous substances consist almost entirely of carbon and hydrogen with very little oxygen, nitrogen and sulfur and have highly variable consistency. Bitumen is nonaqueous, elastic or solid hydrocarbon either naturally occurring (asphalt) or obtained from destructive distillation (pyrogenic) of organic substances such as tars and pitches. Bituminous stabilization have been used in the United States since 1870. Projects utilizing soil and sand-asphalt stabilization were constructed in the United States in 1930 (Terrel et al., 1984). Since then, many low traffic roads have utilized mixed-in-place asphalt stabilization techniques. In addition, hot, central plant asphalt stabilization have been used. The main objectives of stabilizing soils with bituminous material are:

- 1- waterproofing fine-grained soils;
- 2- construction expediency;
- 3- upgrading of marginal materials;
- 4- reducing dust; and
- 5- providing cohesion to granular materials.

Stabilization of soil with bitumen, for various engineering works, is becoming more popular because of its binding and waterproofing properties. It is used for the stabilization of soil for road construction, particularly in water-logged or sandy area, and also for the preparation of improved mud mortar and sundries bricks (Uppal, 1967).

Asphalt stabilization will result in an increase in the cohesion of the stabilized material and occasionally will reduce the angle of internal friction. Generally, one stabilizer is required to improve the quality of sands. However, the use of combined stabilizers, is common. Lime or cement is added to the emulsified asphalt mixes to accelerate water loss and enhance the early strength gain, while lime is added to asphalt cement sand mixes or cutback asphalt sand mixes to improve the mix resistance to water and thus will reduce stripping.

Sandy gravels, sands, clayey and silty sands, and fine-crushed rock are the most suitable materials for bituminous stabilization. Highly plastic clays can be treated successfully but may require high quantities of bitumen. The performance and properties of the bituminous-stabilized silt-clay soils are affected by: (1) clay type, (2) type of exchangeable cations present in clay, (3) soil organic matter, and (4) bitumen type and composition. Bituminous uses, applicability, testing procedures, construction, and characteristics of the mixture have been discussed in many standards and publications such as ASTM and the Asphalt Institute.

In addition to the above-mentioned stabilizing agents, numerous chemicals have been used for soil stabilization in many engineering projects. Chemicals such as calcium chloride, sodium chloride, magnesium chloride, sodium hydroxide, gypsum, sodium silicate, iron oxides, phosphoric acid, aniline-furfural, lignosulfonate derivatives and many others have been tried in soil stabilization for different purposes. Waste and by-

product industrial materials have also been used as full or partial replacement of soils and various degrees of success have been reported (Aiban et al., 1999).

2.2.4.3 Mix Design for Bituminous Stabilization

The mix design methods were established to determine the optimum asphalt content that would perform satisfactory, particularly with respect to stability. Stability is defined as the resistance to deformation with an implied emphasis towards resistance to flow or rutting. While durability is defined as the resistance to the effects of weather and its combination with other forces, and enhanced with high asphalt content. As a consequence, the amount of asphalt to be used in a mixture must be in a balance to optimize durability, but yet maintain adequate stability (Jimenez, 1986).

a) Marshall Stability Test

This test is used in laboratory mix design of bituminous mixtures where specimens are prepared and tested for a maximum load and flow. The Marsall test was conceived by Bruce Marshall in the late 1950's. In order to determine laboratory method of compaction for specimens and to establish criteria on certain mixture properties as evaluated by Marshal testing device, the U.S Army Corp of Engineers (USACE) constructed a full scale test track that incorporated variables such as asphalt content and gradation of aggregates. Loaded trailers were pulled over the track for number of times, so that the effect of compaction due to traffic loads can be evaluated. Later,

USACE conducted extensive research and correlation studies that improved and added more features to the Marshall test procedure. This procedure has been standardized by the American Society for Testing and Materials (ASTM) and given in detail under ASTM designation D-1559. Since its development in the 1940's, the Marshall method has increasingly been accepted by highway agencies throughout the world to design and control bituminous paving mixtures. This wide acceptance is due to its simplicity and good portability for field control of paving mixtures. Benson (1952) found that the methods' success in preventing shear deformation (rutting) failure comes from the control of aggregate texture and gradation, asphalt content, and compaction.

b) Creep Test

The creep test is a measure of pavement rutting potential. It has been designed to measure compressive stiffness or compliance properties of the mixture, and establish plastic flow potential of the mixture under stress states in terms of viscoplastic strains. Van der Poel (1954) indicated that static and dynamic test measurements have provided similar stiffness trends; hence, the static creep was viewed to be an adequate test for establishing such trends. Thus the creep deformation of a cylindrical specimen under a uniaxial, static compressive load is measured as function of time and the data obtained can be used to predict the permanent deformation under different traffic loading and temperatures. In this test, a constant stress (σ_0) is applied to the specimen and the resulting time dependent strain (ϵ_t) is measured. The relevant quantity is the stiffness

modulus of the mixture, defined as:

$$S_{\text{mix}} = \sigma_o / \varepsilon_t \quad (2.1)$$

Shell researchers (Hills et al., 1974) have devoted considerable efforts to develop laboratory creep test methods, that allow the characterization of the mechanical properties of an asphalt mix. These methods enable the prediction of the depth of rut that may occur when this mix is used in pavements subjected to certain loading and climatic conditions.

Limiting values of creep test moduli, to be used in conjunction with mix design to insure its suitability, were developed. The minimum creep modulus of 80 Mpa (120,000 psi) at 40 °C and stress of 200 kPa (30 psi) for conditions of heavy, slow moving traffic (Kamyar, 1990) was adopted for hot mixed asphalt concrete.

2.2.4.4 Sand and Marl Stabilization

Mineralogically, sands are mostly quartz, which is considered the most resistant mineral to chemical and mechanical degradation. Thus, the water sensitivity of the sand is low. In Saudi Arabia, difficulties in obtaining good quality soils for different projects, in addition to the low bearing capacity of sand in its natural and compacted conditions necessitate stabilization of sands to improve their strength and quality for future projects. The engineering properties of sands can be improved economically by the use of admixtures. Some of the widely used admixtures include portland cement, fly ash, and

bitumen. Moreover, sand can be blended with other available marginal soils such as marls to produce a better quality soil.

There have been several attempts to investigate the effect of sand content on the properties of marl. Qahwash (1989) blended marl with three types of sand (fine, medium and coarse) at different percentages, to study the effect of sand content on the uniformity coefficient, specific gravity and plasticity index. He observed that these three parameters decrease as the sand content increases. On the other hand, the maximum dry density increases with the increase in the sand content. All mixes gave lower optimum moisture content as more sand was introduced into the mix. Such a decrease may be attributed to the total decrease in fines as more sand was added to the mix, and to the lower water affinity of the sand. The range of moisture content from the dry side to the wet side of the optimum moisture content was also reduced with the increase in sand content. It was noticed that as more marl is present in the mix, a greater amount of water is absorbed.

In addition, the effect of sand content on the bearing characteristics of various samples is investigated using the CBR test. The results indicate that the sand content considerably affects the load bearing capacity of the mixes. Mixes with less than 70% sand content showed lower CBR values than mixes with more than 70% sand content. Furthermore, there is a significant difference between soaked and unsoaked CBR values. The influence of soaking was characterized by a reduction in the bearing capacity that could amount to one-third the unsoaked CBR value.

2.2.4.5 Additives

Previous studies have shown that the incorporation of additives in small amounts to the sand and marl soils results in a high strength gain of the mix. The bases for any investigation that could result in adoption of such additive are the extent of strength gain as related to: (1) the type of soil, (2) the influence of the additives on the cohesion and angle of internal friction of the soil, and (3) its effectiveness in terms of the durability of the stabilized soil.

Paquette and McGee (1961) have used the unconfined compression strength test (UCS) to evaluate the stability of five stabilized soils of widely varying physical properties, where three of them were sandy soils. Stabilizing agents were portland cement, lime and fly ash mixture, phosphoric acid, and cutback asphalt (RC-3). The results revealed that portland cement was the most effective stabilizer in giving high UCS (2691-5244 kPa). In addition, the strength was improved by lime-fly ash admixture at a ratio of 1:2. The results also indicated that RC-3 at a dosage of 2.5% gave maximum UCS of 345 kPa; whereas phosphoric acid at a dosage of 0.50% gave maximum UCS of 138 kPa. They have noticed that the angle of internal friction increased sharply with small amounts of portland cement, but finally reached a constant value; while the cohesion increased as the cement content increased. The rate of cohesion increase decreases at higher cement contents.

Laguros and Davidson (1963) performed a study where the effects of portland cement, lime, sodium hydroxide and magnesium sulfate were quantitatively evaluated by

the unconfined compression and triaxial compression tests. Eight soils were used; two of which were sands. Triaxial compression tests were performed on clean sand, and sand treated with 12% portland cement. Cement-sand-chemical mixtures indicated that the sandy soil-cement-chemical containing magnesium or sodium ions increased both the cohesion and angle of internal friction. The results revealed that sand-cement-lime mixes have the highest UCS (8970 kPa) and an angle of internal friction of 65° but the cohesion was less than the other mixes (242 kPa).

Dunn and Salem (1971) studied the effect of processing procedures on the strength of sand stabilized with emulsified asphalt. They mentioned that the strength exhibited by the stabilized sand is influenced by numerous variables; both natural and man-made, as shown in Table 2.5. Unconfined compression and triaxial compression tests were used and the results are summarized and presented in Table 2.6. The unconfined tests were used to investigate the influence of the viscosity of the base bitumen and the initial moisture content of the sand. Triaxial tests were used to determine the effects of emulsion content on the cohesion and angle of internal friction of the stabilized sand. The results have indicated that an emulsion content in the range of 7 to 10% gave the highest cohesion (58.7 kPa), whereas, the angle of internal friction was reduced to about $26-34^\circ$. The results have also shown that the unconfined compressive strength of sand-emulsion mixtures was the highest (269.1 kPa) when the emulsion was made using a low penetration grade bitumen base.

Table 2.5: Variables affecting stability of sand stabilized with cationic emulsion
(After Dunn & Salem,1971)

Primary Variables	Natural Variables	Controlled Variables
SAND	Mineralogy, Gradation, Grain size strength, Shape and Surface texture	Moisture content prior to mixing.
BITUMEN EMULSION		Bitumen content, viscosity of base bitumen, emulsifier type & content and amount to be mixed with sand.
FILLER		Chemical properties, gradation and amount to be added.
MIXING	Ambient temperature	Type of mixer, mixing energy per unit weight of mix, order of addition of additive to sand, mixing time, and perhaps mixing temperature.
COMPACTION	Ambient temperature	Mode of compaction, delay between mixing and compaction and dry density achieved.
CURING	Ambient temperature and Humidity	Method of curing and curing time.
ENVIROMENTAL CONDITIONS	Rate of loading, Ambient temp., Rainfall and duration, Evaporation rate and duration, and Drainage condition.	Drainage condition.

Table 2.6: The influence of processing procedures on the strength of sand-cationic emulsion mixes ; Based on the work of Dunn and Salem,1971

Variable	Effect on the strength of sand-cationic emulsion mixes	Remark
Emulsion Content	The addition of emulsion increases the cohesion but reduces the friction angle. The strength depends on: 1) type of test used, 2) rate of loading, 3) efficiency of mixing, 4) density, 5) moisture content, 6) confining stresses, 7) temperature at which sample is mixed, cured, and tested, and 8) age of the specimen	Two types emulsions. Triaxial tests (4"x2"). Conventional undraind. Confin. Stress (0, 60 psi). Temp. Cond. (18 °C).
Initial Moisture Content	It facilitates mixing and distribution of binder; it provides sufficient liquid in the mix to bring it closer to optimum compaction ; and it hydrates any chemically active filler that may be present. 3% MC gave the highest unconfined compressive strength.	Emulsion content (3, 5%). Unconfined compr. test. Stand. vibr. compaction. 7 days air-dry condition.
Emulsion Type and Viscosity	The highest viscosity gave the highest strength. Differences in emulsion properties yielded different in the optimum emulsion content that gave the highest strength.	3 Emulsions were used. Unconfined compr. test. 7 days sealed condition.
Addition of filler	The addition of filler increases the dry density of the mix. Improving the mechanical stability of sand-cement filler has greatly increased the compressive strength of sand-cement-emulsion mixes. Whereas, hydrated lime, crushed limestone dust and sand fines were increasing strength respectively, to a lesser degree.	Unconfined compr. test. Water content (9%). Emulsion cont. (9, 12%). Lime content (5, 10%). Dust content (7%, 12.5%).
Cement & Emulsion On Compaction	The addition of cement only to the sand had increased the dry density. Whereas adding emulsion only reduces the dry density. When adding both cement and emulsion, the dry density increases.	Kango Hammer (4" Dia). 2 Emulsions were used. Emulsion cont. (3,6,9 %). Cement content (10 %).
Mixing Procedures	It was apparent that mixing times in excess of optimum period resulted in slight stripping , but loss of strength was not great. Mixing order of 8 different combination of sand, emulsion, water, and filler revealed that the highest strength were obtained by adding the filler last. While the lowest strengths were obtained by adding the cement filler first due to emulsion adherence to the surfaces of the sand before the cement is added. Whereas, when the emulsion is added to a sand-cement mix, the greater affinity of emulsion for finer particles resulted in poor coated sand.	Unconfined compr. test. 7 days sealed cured cond. 1 Emulsion was used. Emulsion content (6, 9%). Mix time 7.5, 240 second.
Curing	The unconfined compressive strength of emulsion-stabilized sand significantly increased with age. Tests on specimens cured for 12 weeks in air-dry conditions showed that the more emulsion used (within the limit of economic proportions of emulsion), the higher was the strength.	1 emulsion was used. 7 days sealed cured cond. 1,2,4,6,8,12 weeks air-dry Emulsion cont. (6,9,13%). Water cont. (4, 3.5,3%).

Akili and Monismith (1978) conducted an experimental program to define the influence of stress state and curing period on the stiffness and permanent deformation characteristics of a cement-emulsion stabilized sand under repeated triaxial compression loads. Asphalt emulsion together with 1.5 % cement were used. The results showed that the permanent deformation characteristics of the cement-emulsion stabilized sand are functions of stress applications. This was represented by a hyperbolic equation to relate the permanent strain to the applied stress at specific number of stress applications.

Fatani and Sultan (1982) studied the effect of temperature on the compressive strength of dune sand mixed with asphalt and sulfur with percentages of 80, 5 and 15%, respectively. The UCS of samples cured overnight at room temperature was found to be 6440 kPa, 2703 kPa, and 1610 kPa when tested at 40°F (4.4°C), 77°F (25°C) and 140°F (60°C), respectively. The results indicated that when no sulfur was added to the mixture, the unconfined compressive strength was reduced to 4071 kPa, 1104 kPa, and zero for temperature of 40°F, 77°F and 140°F, respectively. They stated that such mixes are weak, unstable, easily deformed under light loads, and non-durable. They recommended the addition of 15% sulfur to the sand asphalt mix in order to improve its quality.

Another work, performed by Al-Barazi (1984) on sulfur-asphalt sand mixes, revealed that the mix gave its highest strength when 18% binder (liquid sulfur into hot asphalt at controlled temperature) and a sulfur: asphalt ratio of 83.33: 16.67 at all confining pressures. Natt and Joshi (1984) used 20% cementitious (cement-lime-flyash)

material and 80% sand as stabilization mixture. Their results indicated that the UCS after 28 days of curing was high (6600-13,250 kPa) at cement: lime: flyash ratio of (1.11: 1.11: 17.28%) and minimum at (0: 2.22: 17.28%).

Al-Halhouli (1986) has conducted a study to evaluate the feasibility of using locally available dune sand blended with various percentages of crusher fines and cement and treated with cationic slow-setting emulsified asphalt (CSS-1h) to construct low-volume roads in Saudi Arabia. His results showed that dune sand treated with emulsified asphalt alone was weak and unstable, and did not resist rutting under traffic loads, especially in hot climates. The addition of crusher fines and portland cement improved the mix properties significantly.

Arora and Arabiat (1986) have blended marl with 30% dune sand and locally produced Emulsified Asphalt (CSS-1h). Portland cement was added to improve the durability and strength characteristics of the emulsified mixture. Dynamic testes were used to predict the in-service performance (stress-strain behavior). The results revealed that there is potential for the utilization of locally available marl and dune sand when stabilized with locally produced Emulsified Asphalt (CSS-1h) and cement for low volume roads in Saudi Arabia.

Al-Abdul Wahhab and Arora (1988) conducted a study to evaluate the use of emulsion, in stabilizing sand and marl, in the Kingdom. Portland cement was added to enhance early curing. They found that increasing Portland cement content (from 2% to

5%) increases the stability from 2900 to 3800 lb, for dune sand emulsion mix and from 3800 to 4600 lb for marl emulsion mix. They stated that dune sand mixes tend to be soft, with highly temperature dependent properties. However, this behavior can be reduced by the addition of low percentages of Portland cement or by cross blending with another material such as marl or crusher fines. Marl emulsion mixes tend to give higher stability than dune sand emulsion mixes. Marl emulsion mixes have the capability to withstand higher in-service temperatures, above 55°C (122°F), with reasonably high modulus values. Marl and sand emulsion can be used for road construction, where both of them have produced satisfactory and economical mixes. However, sand emulsion mix must be avoided in road surface, to minimize temperature effects.

A study was performed by Al-Abdul Wahhab and Hicks (1988) to evaluate the behavior of sand and marl treated with emulsified asphalt. Dynamic tests were used to predict the fatigue and rutting performance of the mixtures under field-simulated conditions. Models were developed and used in a mechanistic design approach to arrive at an optimum road cross section for local conditions. The results of the structural analysis are presented in the form of design charts. Finally, they concluded that dune sand and marl soils, when stabilized with emulsified asphalt, represent an attractive construction material for local roads.

Another study of soil stabilization was carried out by Al-Abdul Wahhab and Al-Abduljawwad (1989) using marl, sabkha and dune-sand obtained from the eastern province of Saudi Arabia. Two types of liquid asphalt were used, cationic slow setting

emulsified asphalt (CSS-1h) and medium curing cutback asphalt (MC-70), due to their availability and ability to treat fine sands. The results revealed that marl and dune-sand liquid asphalt mixes with small percentage of portland cement can be used successfully to build road bases, subbases and foundations for most structures.

Although it can be clearly seen from the reviewed literatures that there are previous works performed on the stabilization of sand and marl soils individually, there was no attempt to stabilize premixed sand-marl mixtures which may be considered a suitable and better alternative material for construction uses. Therefore, there is a potential need for further studies in stabilizing such mixes by mechanical, physical and chemical means using cement and/or bitumen. Results from such a study may bring up a material with acceptable engineering characteristics and economical advantages.

CHAPTER 3

EXPERIMENTAL PROGRAM

Although there are previous works performed on the stabilization of eastern Saudi sand and marl soils individually, there was, however, no attempt to stabilize premixed sand-marl mixtures using different additives. The intention of this chapter is to describe the experimental procedures used to study the characteristics of sands and sand-marl mixes. Moreover, such mixes will be stabilized with traditional or novel treatment techniques using cement and/or bitumen. In this experimental program, the work was carried out in three phases, as shown in Figure 3.1. The first phase is comprised of collection of sand samples from different sites. In the second phase, these samples were characterized using relevant ASTM and AASHTO standards. Based on the results obtained in the second phase, some of these sand samples were selected along with one marl soil for further “detailed” characterization and stabilization which constituted the third phase. All phases of the experimental program were executed in harmony to fulfill the objectives cited in section 1.3 and discussed thoroughly in the following paragraphs.

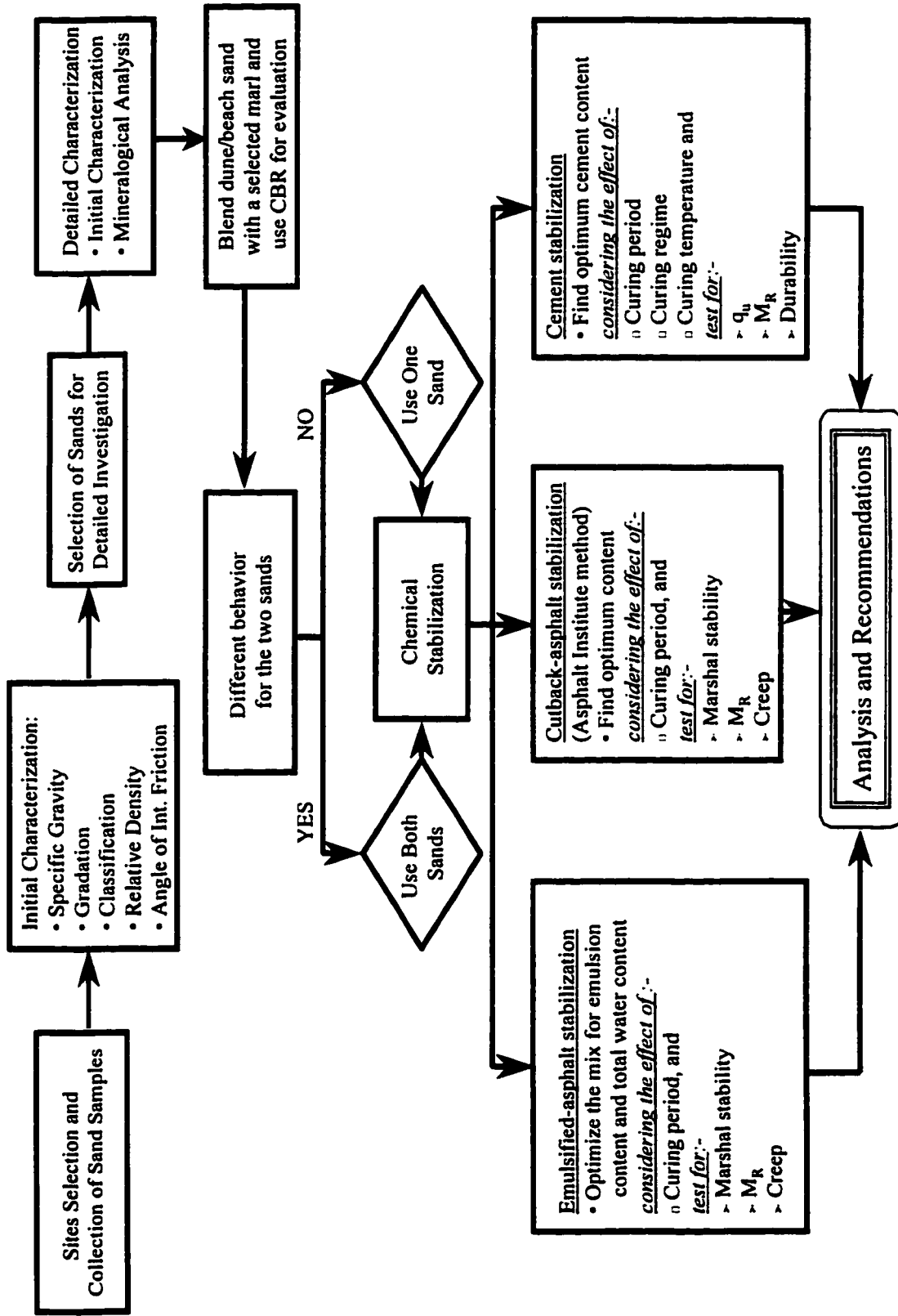


Figure 3.1: General Optimization Procedure for sand-marl stabilization

3.1 Collection of Sand Samples

Desert and beach dune sands were collected from several locations in the Eastern Province. Beach sands were collected from the Dhahran and Qurayyah dunes while the desert sands were collected from Dammam, Abqaiq, Baggah, Jubail and Dhahran dunes. The samples were carefully retrieved from areas having abundant sands that can be used in construction. A total of four beach dune sands and six desert dune sands were collected and transferred to the Geotechnical Laboratory at the Department of Civil Engineering, KFUPM, for preliminary analysis. It must be mentioned that currently there is no active wadi's in the Eastern Province and uncontaminated wadi sands do not exist and, therefore, no wadi sand was collected. In low-lying areas, which might be considered as wadi or wadi-like areas, sand normally exists and it is either contaminated with vegetation or it may evolve into an evaporating pan and will become sabkha in future. In both cases, sand will be contaminated with salt or vegetation in the long run.

Samples were labeled according to their locations. The sands were labeled with 6 or 7 entries. The first and common to all is S for sand followed by three letters for location. The fifth is a serial number for that particular location. The last one or two entries indicate the type of sand, D is for desert dune and B for beach dune. Some are shown with D/B indicating that the sand is somewhat in between. The collected sand

sample's name, place, symbol, location and general description are given in Tables 3.1 and 3.2. The locations of the collected samples are also shown in Figure 3.2.

3.2 Sand Characterization

The collected sand samples were subjected to a preliminary testing program to explore their general properties including classification, specific gravity, grain-size distribution, maximum and minimum density values, peak angle of friction and mineralogical composition (X-ray diffraction). These tests were conducted according to the following standards:

- Classification (USCS, AASHTO M145),
- Specific gravity (ASTM D 854),
- Grain-size distribution (dry and washed) (ASTM D 421 and ASTM D 422),
- Maximum and minimum dry density (ASTM D 4253 and ASTM D 4254),
- Peak friction angle at 75% relative density and a confining-pressure of 69 kPa (ASTM D 2850).

3.2.1 Specific Gravity Test

Although the specific gravity is not a direct measure of the strength and other engineering properties of the soils, it is used as a parameter in the determination of some

Table 3.1: Locations of the collected sands

Sand Name & Place	Abbreviation Symbol	Location
Dammam Sand-1	S-DAMI-D	Abu Hadriyah Hwy-85 North, at Station 77, closer to the north boundary of King Fahd Airport, 3-4 km west side of the Hwy. Sample was taken from the lower third of the dune.
Baggah Sand-1	S-BAG1-D	Dammam-Riyadh Hwy-40 west, at Station 1300, 20 m east side of Hwy. Sample was taken from the middle of the dune to avoid intrusion of salt from groundwater table.
Abqaiq Sand-1	S-ABQ1-D	Dhahran-Abqaiq old Hwy-10 west, at Station 1503, north side of the road, 35 km from Dhahran. Sample was taken from the middle of the dune.
Dhahran Sand-1	S-DHA1-D/B	Al-Aziziyah-Half Moon Bay road, 500 m north-east of the road opposing the Municipality water tower. Sample was taken from the middle of the dune.
Dhahran Sand-2	S-DHA2-D/B	Al-Aziziyah-Half Moon Bay road, 500 m north-east of the road parallel to exit no. 4. Sample was taken from the top of the dune.
Dhahran Sand-3	S-DHA3-D/B	Al-Aziziyah-Half Moon Bay road between exit 2 and 3, west side of the road. Sample was taken from the flat (upper part) of the large dune near the shoreline.
Al-Qurayyah Sand-1	S-QUR1-D/B	Dhahran-Al-Qurayyah-SCECO road, about 75 km from KFUPM, 200 m north of Abqaiq-Al-Qurayyah road. Sample was taken from the middle of the dune.
Al-Qurayyah Sand-2	S-QUR2-D/B	Dhahran-Al-Qurayyah-SCECO road, about 54 km from KFUPM, 400 m east of the road. Sample was taken from the middle of the dune.
Dahna Sand-1	S-DAHI-D	Dammam-Riyadh Hwy-40 west, at Station 1058, 50 m north side of Hwy. Sample was taken from the top of the dune to assure salt-free sample.
Jubail Sand-1	S-JUB1-D	Jubail-Abu Hadriyah Hwy connection near the hazardous waste landfill.

Table 3.2: Description of the collected sands

Sand Name & Place	Abbreviation Symbol	Soil Description / Remarks
Dammam Sand-1	S-DAM1-D	Fine to medium size, white sand with very light cementation (the sand can stand in vertical cut when dry).
Baggah Sand-1	S-BAG1-D	Fine to medium size sand obtained from light brown large dune. The water table was 6 to 8 m below the Hwy. Al-Ajeinah Co. is utilizing this sand for construction.
Abqaiq Sand-1	S-ABQ1-D	Light brown sand, fine to medium size.
Dhahran Sand-1	S-DHA1-D/B	Layering with grey dust seams was observed. Some minerals, such as gypsum, were present at the bottom. Capillary rise was observed, heavily utilized for construction.
Dhahran Sand-2	S-DHA2-D/B	Whitish dry sand, dune was high in elevation and large in size.
Dhahran Sand-3	S-DHA3-D/B	Generally same characteristics as S-DHA1-D/B and S-DHA2-D/B but relatively coarser.
Al-Qurayyah Sand-1	S-QUR1-D/B	Fine to medium, white sand, obtained from a medium size dune. Soft, small, white shells, fragments were observed.
Al-Qurayyah Sand-2	S-QUR2-D/B	Fine to medium, light brown sand, obtained from a very large dune.
Dahna Sand-1	S-DAH1-D	Fine to medium, orange to red sand.
Jubail Sand-1	S-JUB1-D	Fine to medium, brown sand, obtained from flat dune. All dunes in the area are small to medium in size.

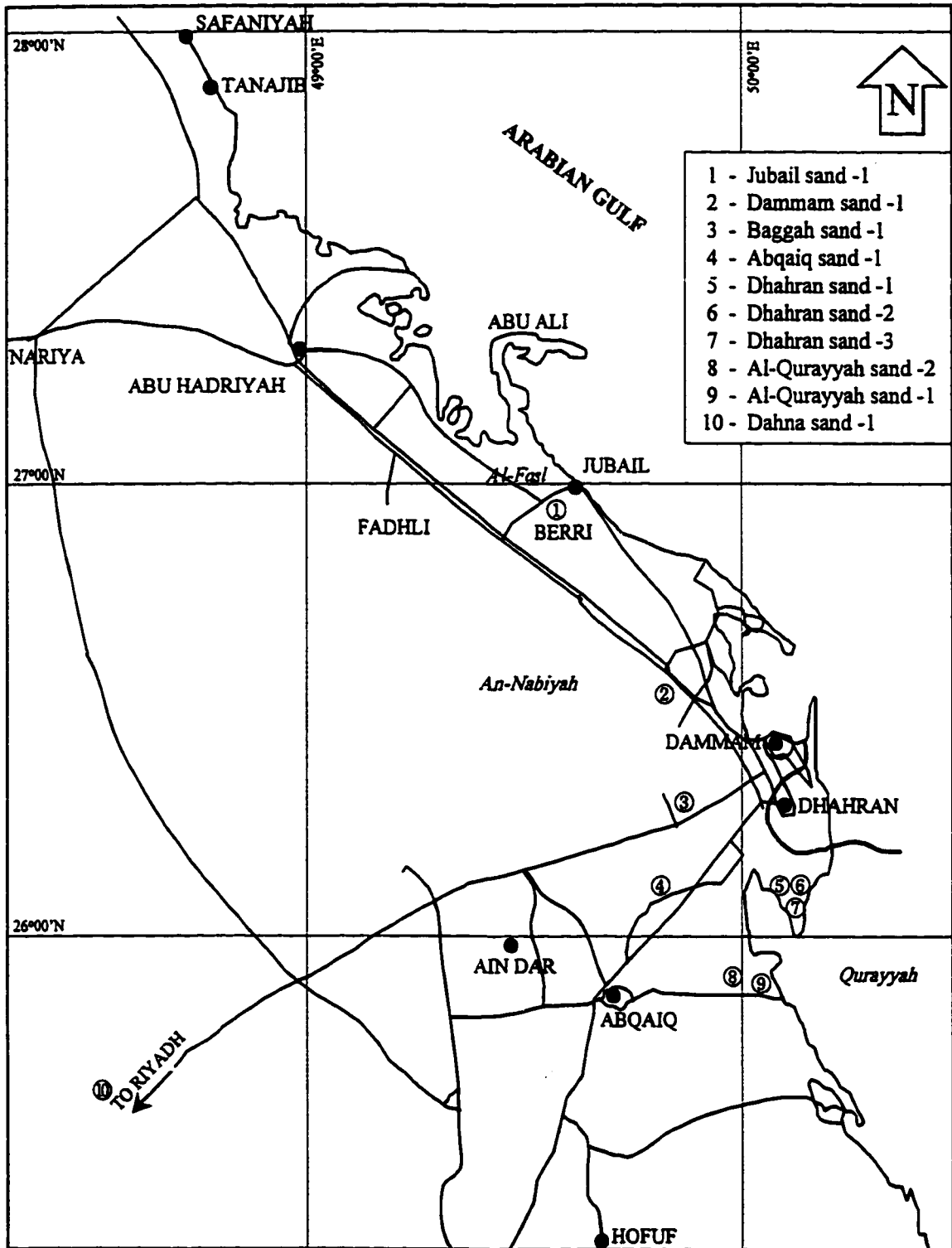


Figure 3.2: Approximate locations of sand samples

important properties of the soil such as void ratio, unit weight and soil particle-size determination.

For each sand type, the soil passing ASTM Sieve No. 4 was used for the determination of specific gravity as per ASTM D 854. The average of triplicate specimens was taken as the specific gravity value of the soil.

3.2.2 Grain-Size Distribution

The soil passing ASTM No. 4 sieve was subjected to both wet and dry sieving (ASTM D 422). These methods are also known as washed and unwashed sievings, respectively. A set of sieves, including ASTM No. 10, 20, 30, 40, 60, 80, 100, 140 and 200, was used for both wet and dry sieving. After dry sieving, the soil retained on each sieve was weighed. These soil portions were collected in a dish and remixed. The same soil was also sieved in the wet process through the same set of sieves. Distilled water was used in the washing process. Soil portions retained on each sieve as well as passing the No. 200 sieve were dried in the oven and then weighed.

3.2.3 Relative-Density Determination

The state of compactness of a natural granular soil is commonly expressed by its relative density, D_r , and defined as:

$$D_r = (\gamma_{d \max} / \gamma_{df}) [(\gamma_{df} - \gamma_{d \min}) / (\gamma_{d \max} - \gamma_{d \min})] \quad (3.1)$$

Where, $\gamma_{d \max}$ = dry unit weight of soil in its densest state,

$\gamma_{d \min}$ = dry unit weight of soil in its loosest state, and

γ_{df} = dry unit weight of soil in the field.

Briefly, as per ASTM D 4253 and ASTM D 4254, $\gamma_{d \min}$ is determined by pouring dry sand, from a fixed height, into a mold in loosest form; whereas, $\gamma_{d \max}$ is determined by vibrating a sample subjected to a surcharge weight. The $\gamma_{d \max}$ is obtained from densifying either a dry or saturated sample (dry or wet method).

3.2.4 Conventional Triaxial Compression Test

Triaxial cylindrical samples (71 mm in diameter and 142 mm in height) were densified to the required relative density (i.e. 75%) by pouring the dry sand into the mold from a fixed height. Axial compression testing was performed for all samples in as-molded condition in an air-dry condition.

The obtained data were used to plot a Mohr's stress circle. The cohesion is zero and the angle of internal friction (ϕ) can be computed from the geometry of a Mohr's circle from the relation:

$$\sin \phi = (\sigma_1 - \sigma_3) / (\sigma_1 + \sigma_3) \quad (3.2)$$

and thus one test was enough for sands since the cohesion intercept was zero.

3.2.5 X-Ray Diffraction (XRD) Analysis

This technique is used to define the mineralogical composition of a material. Knowledge of the mineralogical and chemical composition of a soil is important and can help understanding the behavior of the soil. The method is qualitative as it provides information about the type of existing minerals but it is semi-quantitative as it gives the percentages of crystalline minerals only.

For XRD analysis, the soil sample is ground into fine particles by a ceramic mortar pestle and into powder form by agate mortar pestle. The sample is pressed into a sample holder. This method is called Powder Method of XRD. The crystals are present in a random orientation in the sample. A monochromatic (single-wavelength) beam of X-rays is made to strike the crystals. Since the crystals are composed of parallel planes, represented by Miller indices (jkl), X-rays striking these planes reflect at a certain incident angle θ . A series of reflections occurs depending on the type of minerals present in the soil. This reflection takes place only if the incident beam makes an angle which satisfies Bragg's law. The reflected beam is recorded on a strip chart or computer file according to the intensity of the reflected beam.

Comparison of the intensity and the angle of incident (θ) with those of standard minerals gives the type and percentage of various minerals present in the sample. The comparison of diffraction pattern is made with standard diffraction patterns for different phases established by the Joint Committee of Powder Diffraction Standards (JCPDS).

3.3 Sand Selection for Detailed Study

Due to the similarity in physical properties that exists between the different sands, except for Qurayyah Sand-1 which does not exist in commercial quantities, it was decided to select two sands from two different environments, namely desert and beach, for the detailed stabilization. For this reason, Baggah sand (S-BAG1-D) and Dhahran Sand-2 (S-DHA-2/B) were selected for the detailed study. Baggah sand was selected due to its proximity to construction activity and its abundance. As a matter of fact, several contractors such as Al-Ajeinah, utilizes this sand for construction projects. Dhahran Sand-2, which is sampled from beach dunes close to the shoreline, was selected to study its potentiality as a construction material and to compare it to the desert dune sand.

3.4 Selection of a Marl Sample

Marl is considered a main source for subbase, base course and foundation materials in the Eastern Province of Saudi Arabia. It is also used for liners in waste containment bonds and landfills. There exists in a wide variations in the marl types of eastern Saudi Arabia. It is, therefore, necessary to select a marl that has a potential use and with medium quality. Abu Hadriyah Marl (M-ABH2) from Al-Dossary crusher was selected for the detailed investigation in this work.

Abu Hadriyah Marl (M-ABH2), as reported by Ahmed (1995), is located about 10 km to the west of Dammam-Abu Hadriyah Highway, about 130 km from Dammam. It is the main borrow area for Abdullah A. Al-Dossary Company. The soil from this borrow

area is accepted by the Dammam Municipality for its maintenance and construction projects. The borrow area at the time of sampling is 10 to 12 m below the ground surface (excavation depth). The profile of the borrow area showed a large variability in the material with different layers. Well-defined and distinct layers of sound limestone, pure and relatively loose sand, loose chalky material, soft limestone were clearly visible. The marl of this area was obtained from mixing 2 to 3 m thick layers located at a depth of 10 m below the ground surface. The soil sampled was light yellowish in color. Sandstone and some softer materials were found to be fossiliferous in nature.

Abu Hadriyah Marl (M-ABH9) is being used by the Dammam Municipality for the construction and maintenance projects. The material was taken from the area of M-ABH2, but being blended as a base course material labeled as M-ABH9. The marl was reconstituted to a specific base course gradation in the KFUPM Geotechnical Laboratory, to insure reproducibility of samples. The gradation limits to which the soil was reconstituted is shown in Figure 3.3 as used by the Dammam Municipality. Moreover, other standard limits such as ASTM and MOC gradations are also shown.

3.5 Optimization of Sand-Marl Mixtures

An optimization program of the dune sand by blending with Abu Hadriyah marl was undertaken to determine the geotechnical properties of such mixes, which may be considered as a suitable alternative construction material. Baggah dune sand and Dhahran sand-2 were mixed separately with Abu Hadriyah marl (M-ABH9) at different ratios and optimized based on CBR values.

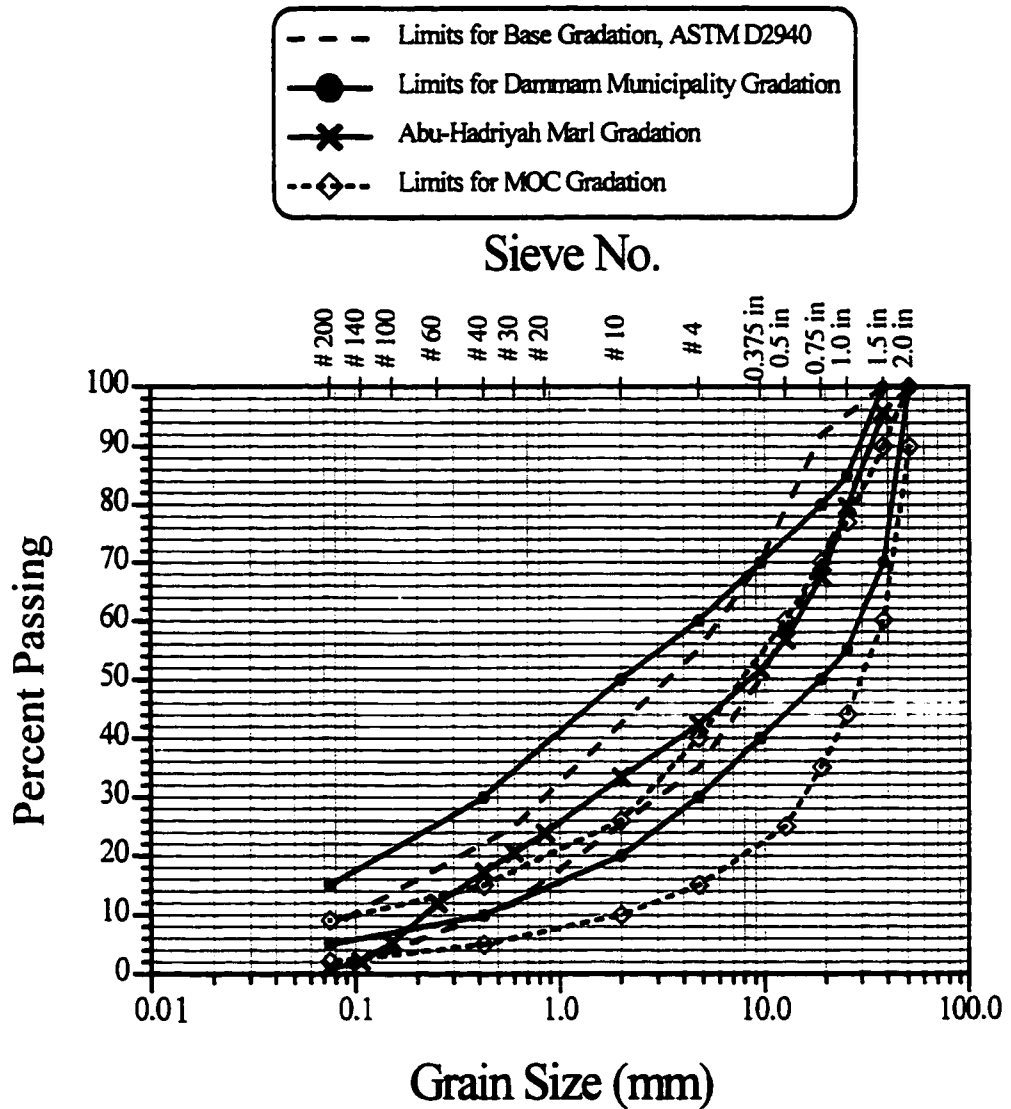


Figure 3.3: Base Course Gradation specified by various organizations

3.5.1 California Bearing Ratio (CBR) Test

California bearing ratio (CBR) test is commonly used in the structural design and evaluation of pavements. There is no direct relation between CBR value and the shear strength parameters, such as cohesion and angle of internal friction. It is basically a static test, while the loads generated by the traffic wheels are dynamic in nature. The results can only be used for the empirical design methods, for which the test was devised. In spite of these limitations, the test is recognized worldwide because of its simplicity and applicability. Therefore, it can easily be used to quantify the material for use in pavement construction. In addition, the test can be used to compare different materials and evaluate their water sensitivity.

The samples prepared for the moisture-density relationship were subjected to CBR testing procedure (ASTM D1883). Samples were tested immediately after preparation so that no moisture loss is permitted. For the preliminary characterization stage, only unsoaked CBR tests were performed. However, to simulate the field conditions in which the soil is flooded with water, soaked CBR test was adopted for the sand-marl mixes selected for detailed characterization. Flooding can either be from the ground water or from the rain water infiltrating the soil layers.

3.6 Stabilization of Sand-Marl Mixtures

Stabilization of sand-marl mixes is one of the primary objectives of this investigation. The stabilization techniques were employed on the basis of the behavior

of sand-marl mixture observed in the characterization stage. The sand-marl mixture selected for detailed stabilization was then optimized for the selected stabilization technique(s) taking into account certain parameters that are expected to affect the behavior of the stabilized soils.

In this study, portland cement and bitumen (emulsion/cutback) were used as the chemical additives for the selected sand-marl mixtures. The appropriate chemical additive was thereafter selected based on the results from CBR/Marshall tests. Moisture-CBR relationship was developed and compared for the 5% additive content in the case of cement. For bitumen, Marshall stability-bitumen relationship was refined and compared for 4% emulsion content as well as 4% cutback.

3.6.1 Optimization of Sand-Marl Mixtures Stabilization

The improvement of any soil does not only depend on the type and amount of stabilizer added but also on the environmental conditions associated with a particular site and the construction procedures. Considering all the conditions which contribute positively or negatively, an optimum level of stabilization should be determined which, besides being economical, should satisfy minimum requirements intended.

Since the intention in this study is to use sand-marl mixture as a construction material for bases and sub-bases of pavements and foundations; the strength, settlement and durability are the primary concerns. Strength of a stabilized material can be expressed in terms of the unconfined compressive strength, CBR or resilient modulus and

can be enhanced with certain chemical additives. There are, however, certain ranges of moisture and temperature for which the benefits are maximized.

The following parameters were considered for the optimization of sand-marl stabilization process:

1. Additives (cement and emulsion / Cutback) content,
2. Wetting and drying,
3. Curing conditions (period, temperature and exposure conditions).

The effects of these parameters on strength and durability were evaluated using the following parameters whenever applicable:

1. Unconfined compressive strength,
2. Marshal stability (dry and soaked),
3. Resilient modulus,
4. Durability.

These parameters along with their importance in the optimization process are discussed in the following paragraphs.

3.6.1.1 Cement Content

In this investigation, cement content is defined as the percentage of the weight of cement to that of the dry weight of soil plus cement. Because cement is an expensive material, there is a need to determine an optimum value, which depends on the soil type.

Granular soils are the most suitable one, as they do not require any pulverization. They mix well with cement and need less quantity of cement due to their relatively low specific surface area. On the other hand, fine-grained soils require high percentages of cement due to their large specific surface area. Normally, the cement content, for soil stabilization purposes, ranges from 4 to 16% by dry weight of soil (ACI Committee, 1990).

In this research, three different percentages of cement were used to optimize the cement content for cement-sand-marl mixture. These were 3, 5, and 7% in addition to the untreated soil (i.e. 0% cement). These percentages were initially used to get the moisture-density relationships of soil-cement mixes. The desired cement content, for detailed investigation, was selected based on the unconfined compressive strength (UCS) of cement-stabilized specimens.

3.6.1.2 Curing Conditions

Portland cement reacts with water to produce cementitious material, known as cementing gel. This reaction is called cement hydration. Cement needs certain amount of water for its complete hydration. The hydration reactions are initially fast, but its rate decreases with time. In general, the rate is very high during the first 7 days, and depends, among other factors, on the ambient temperature and curing conditions.

In the field, the moisture loss is usually prevented by different methods. The exposed surface of the stabilized soil is either covered with a wet layer of sand, burlap,

etc., or the surface is sealed by spraying curing compounds such as emulsified asphalt. Water is regularly sprinkled to substitute for the moisture loss. To simulate field conditions, the samples for unconfined compressive testing were subjected to various curing regimes. The moisture conditions were controlled either by sealing the sample in plastic wraps or by keeping the specimens uncovered (exposed conditions) in the laboratory environment. The sealed samples were wrapped with 5 layers of plastic sheet, to ensure that no moisture loss took place. The samples for the exposed conditions were not sealed at all.

Due to the variations in the air temperature on daily and seasonal basis, it was hard to simulate the field conditions accurately. According to Al-Abdul Wahhab et al. (1994), the average seven day maximum temperature in Dhahran and Al-Hasa area of the Eastern Province has been recorded to be 46.3°C and 47.6°C, respectively. Therefore, to get an idea about the strength gain of the soil-cement specimens with the variation in temperatures, the samples for unconfined compressive testing were subjected to different curing temperatures including room temperature (21°C), 35 and 50°C. The samples were put in the oven to cure at constant temperatures of 35 and 50°C. The samples were allowed to cure at these temperatures under both the exposed and sealed conditions.

3.6.2 Unconfined Compressive Strength

Unconfined compressive strength test is commonly used for the evaluation of chemically-stabilized soils. In the structural design of pavements, the unconfined

compressive strength (UCS) is used as the strength criterion for the base and the sub-base layers. In addition to the UCS, the resilient modulus test was adopted for the evaluation of pavement layers and the test relates more closely the response of pavement material to impulse loading. Nevertheless, the UCS is used in many standards and codes for the stabilized materials. Usually, a minimum UCS value is specified for different layers of a pavement system. It was, therefore, adopted as a basic test in this research where a comparative study of the effects of different parameters such as curing regime, curing temperature, curing period on the strength gain of the treated marls were investigated. The unconfined compressive strength test (UCS) was also used for evaluating the strength characteristics of both untreated soil and soil-cement mixtures.

In this research, cylindrical specimens with h/d ratio of 2.0 were prepared for all unconfined compressive testing. All the samples were reconstituted to meet the gradation requirement. The required cement and the corresponding (optimum) moisture were added to the sand-marl mixture and mixed in a mechanical mixer for 4 minutes. The mix was then, compacted in a mold having a diameter of 100 mm and a height of 200 mm ($h/d = 2.0$) to the maximum dry density of the treated sand-marl mixture according to the modified Proctor compaction test. The mix was compacted in 5 equal layers. The number of blows was adjusted to attain the maximum dry density at the optimum moisture content. The mold used is of a split type with a longitudinal slit along its axis. The slit is tightened and opened with the help of bolts. After compaction, the specimens were taken out of the molds by loosening the bolts.

3.6.3 Modulus of Resilience

The resilient modulus M_R is a dynamic test response defined as the ratio of the repeated axial deviator stress σ_d to the recoverable axial strain ϵ_a (Yoder and Witczak, 1975).

$$M_R = \sigma_d / \epsilon_a \quad (3.3)$$

Since the inclusion of resilient modulus test in the new AASHTO design guide for pavements (AASHTO Design Guide, 1986) for the characterization of material properties, it has started replacing the static strength tests. The resilient modulus test is dynamic in nature, and simulates the response of the pavement layers to the impulse loading produced by the moving wheels of vehicles. Moreover, the conventional strength tests determine only the relative suitability of the material based on their ultimate “static” strength, while the properly-designed pavement layers seldom receive loads that are close to failure. The performance of a material at low impulse load levels is quite different from that at higher load levels.

The impulse load causes deformation in pavement layers consisting of plastic and elastic components. The elastic deformation is termed recoverable or resilient. Before 1986, studies of resilient modulus of various types of aggregates and soils were mainly motivated by the fact that the failure of highway pavement could result not only from excessive accumulated permanent deformation of subgrade under repeated traffic loading, but also from the fatigue cracking of the asphalt concrete surface caused by the repeated resilient deformation (Li and Selig, 1994). Later on, the resilient modulus study

of subgrade soils and other pavements layers has become essential in pavement design procedures.

In this investigation, the test adopted for the stabilized materials was the diametral resilient modulus. Stabilized soil samples of 101.6 mm (4 in.) in diameter and 63.5 mm (2.5 in.) in length were prepared. The samples were compacted in one layer. Blows were adjusted to achieve the required maximum dry density based on the modified Proctor test of the treated soil. After compaction, the samples were extruded from the mold and subjected to different curing regimes similar to those for the unconfined compressive test. At the end of the curing period, these samples were exposed to diametral resilient modulus testing.

For testing, each stabilized sample was fixed in the testing yoke. The yoke containing the sample was fixed-in position in the resilient modulus device, by applying a seating stress of 4 psi (27.6 kPa). The yoke has two LVDTs on the two opposite ends of the sample. The LVDTs were used to measure the diametrical resilient tensile strain produced upon the application of impulse load. An impulse load of 150 lb. (68.2 kg) was applied normal to the direction of the installed LVDTs. The resilient strain at the end of the 100th cycle of impulse load was used in calculating the resilient modulus (M_R) of the stabilized soil. Both the applied load and the resulting strain were automatically measured by electronic devices and transferred to a resilient modulus computing system.

3.6.4 Durability

Stabilized soils need to be strong and durable so as to resist all types of loads under different exposure conditions. Variation of moisture and temperature, can produce wet and dry or freeze and thaw cycles. Also, the stabilized soils should maintain stability under the cyclic environmental loadings. These conditions cause weight loss and/or volume change to the soil-cement. The volume change induces stresses in the soil-cement and contributes directly to the loss of serviceability and strength.

Durability of soil-cement specimens was evaluated using two different procedures, the standard ASTM D 559 method and the proposed slake durability test (Aiban et al., 1999). The latter is originally used for rocks and was modified to accommodate cement-stabilized soils with certain sizes.

a) Standard Durability Test (ASTM D 559)

Soil-cement specimens were prepared with different percentages (3, 5 and 7%) of cement. The molds used to prepare specimens were 101.6 mm (4 in.) in diameter and 116.8 mm (4.6 in.) in height. Each sample was compacted to the maximum dry density obtained from the modified Proctor of the treated soil. The number of blows was adjusted and the samples were compacted in three layers. After compaction, the sample was extruded from the mold. Four samples were prepared for each cement content. Two of these samples were designated as weight loss samples, while the other two were designated as volume change samples. The exact height and diameter of the volume change samples were measured.

All the samples were allowed to cure under 100% relative humidity and a temperature of 21 ± 2 °C for 7 days. The samples were then placed in a water tank for 5 hours at room temperature and thereafter transferred to an oven. Samples were kept in the oven at 71°C for 42 hours. This is what constitutes one wet-dry cycle for the soil-cement samples.

At the end of the cycle, the samples designated as volume change samples were dimensioned with the help of a vernier caliper, and were weighed. The other two samples were brushed, using a standard brush, with two strokes on the whole surface with a force of about 3 lb. (1.36 kg). To apply the 3 lb. (1.36 kg.) force, each sample was placed on a balance, and was then brushed while observing the specified force on the scale of the balance. The weight of the samples before and after brushing was measured.

Similar procedures were repeated for the remaining 11 cycles, thus subjecting the samples to a total of 12 cycles. At the end of each cycle, the weight loss and volume change were noted for the respective samples. At the end of the 12th cycle, the samples were dried to constant weight at 110 °C.

b) Slake Durability Test

This test is based on the slake durability of rocks (Goodman, 1980). A certain specific weight (500 gm) of rock pieces is put in a drum made of 2.00 mm mesh. The drum is 100 mm in length and 140 mm in diameter. The drum is rotated at a speed of 20

rpm while being partially submerged in water. The weight loss after 10 minutes of rotation is a measure of the durability of the rock.

The above test was adopted for the soil-cement samples with some modifications (Aiban et al., 1999). The diameter and length of the drum was changed to 152.4 mm (6 in.) long and 304.8 mm (12 in.) in diameter. To allow the soil-cement specimens to travel the same distance as that of the rock pieces in the original test, the number of revolutions was adjusted to account for the change in dimensions. The revolution time was reduced from 10 to 4.6 minutes. This will give a total travel distance of 88 m. The set up for slake durability testing is shown in Figure 3.4.

Two additional samples were compacted for each percentage of cement. These samples were subjected to the same wet and dry cycles as for the samples tested using ASTM D 559 durability test. At the end of each cycle, each sample was tested for durability using the modified drum "Slake durability". After slaking, the surface of the sample was cleaned with a dry absorbent cloth and then weighed. For each cycle, weight loss for each sample was noted by taking the weight before and after the slaking test. After 12 cycles, the samples were oven dried at 110°C.

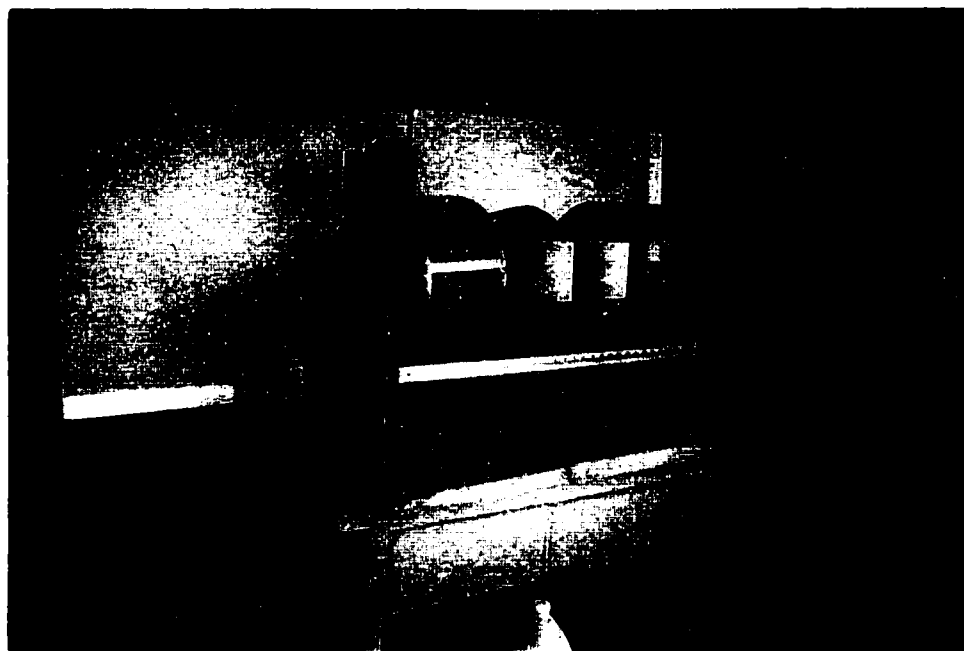


Figure 3.4: The setup for Slake Durability Testing (Aiban et al., 1999)

3.6.5 Emulsified and Cutback Asphalts Mix Design

The development of a standardized procedure for the design of mixtures containing emulsified asphalt and aggregate presents a significant challenge to the highway industry. In spite of the considerable research that has been done in this area, unanimous agreement has not been reached and much research remains to be done (Puzinauskas and Jester, 1983). The design of emulsified asphalt mixes (EAM) is considerably complicated because of the difficulty of duplicating the field curing of EAM in the laboratory. Furthermore, the ultimate stability and the related properties are not reached until virtually all the water and kerosene in the emulsion and cutback mixes, respectively, have evaporated. Under field conditions, this evaporation may require, especially in emulsion mixtures, several months.

In virtually all EAM, it is necessary that some percentage of mixing water be added to the aggregate to facilitate coating and workability of the mixture. This is largely a trial-and-error method. The amount of mixing water must be sufficient to aid in an even distribution of the emulsified asphalt over the aggregate surface, yet water should not be used to the extent that the emulsified asphalt will drain from the aggregate particles. Complete asphalt coating of the soil particles is not necessarily required in order to produce a satisfactory mix. It must be understood that the emulsified asphalt contains about one-third water, which is lost in the curing process.

The asphalt mixtures must possess sufficient stability or resistance to deformation to support the anticipated traffic loads without cracking, rutting, or distorting. In many EAM, stability continues to increase over time as water evaporates.

Several different approaches are currently being used to get the optimum mixes. Procedures using the stabilometer and cohesiometer include: the Asphalt Institute procedure, California procedure, and Chevron procedure. Procedures using the modified Marshall test method are adopted by some agencies such as Aramak Co. and Illinois Department of Transportation. Since the Marshall method of mix design is commonly adopted for hot mix asphalt concrete and the needed equipment for mix design are routinely available in Saudi Arabia with all contracting agencies, the modified Illinois method was adopted for the design of EAM mixtures. Certain modifications with respect to curing were incorporated in the design procedure.

Two types of bitumen were used in this study namely: cationic slow setting emulsified asphalt, CSS-1h, and medium curing cutback asphalt, MC-70, which are locally produced by Sandstill, Dammam, Saudi Arabia. Illinois method of mix design was adopted to determine the optimum emulsion content. The method involves: calculating the surface area of the mix; performing Centrifuge Kerosene Equivalent (CKE) and oil ratio tests to determine the range of trial asphalt content. The range asphalt contents were then checked to determine the optimum asphalt mixes.

3.6.5.1 Specimen Preparation

The following steps were used to determine the optimum emulsified asphalt sand-marl mixtures:

- (i) 1000 g. of mixture of sand and marl was mixed with premixing water, the amount of water depends on the surface area of the aggregate. Water is added in a thin stream until the color of the aggregate started to change. At this point no more water is added and the amount of water is called the premixing water.
- (ii) A spectrum of emulsified asphalt percentages (2% to 6%) was evaluated, where the initial premixing water was fixed at constant value and the added water varied from 0% to 3%.
- (iii) To achieve good dispersion and satisfactory coating, in addition to the determination of optimum asphalt content, water was added in different percentage to the consecutive emulsion contents.
- (iv) After the addition of premixing water, mixing was initially done by hand followed by mechanical mixing in a Hobart mixer. A reasonable mixing time of 30 seconds was found necessary to ensure proper coating of the aggregate. The mixing was carried out for another 30 seconds after the addition of the required amount of emulsified asphalt and the added water.
- (v) The mixtures were then compacted into Marshall specimens (4-in diameter and 2.5-in high), using 75 blows on each side of the specimen.
- (vi) The compacted specimens were cured and tested for stability.

- (vii) Five specimens were made for each emulsified asphalt content, three of them were tested for dry stability and two for soaked stability.
- (viii) The emulsified asphalt content was plotted against stability, and the mix that gave maximum dry stability was selected.

In cutback mixes, kerosene is the solvent liquid in the asphalt, accordingly, no premixing water nor added water were required. A range of cutback percentage (1% to 7%) was used. The mixing was carried out mechanically for 60 seconds. Afterward, the mixtures were then compacted into Marshall specimens, using 75 blows on each side of the specimen. The compacted specimens were allowed to cure and then tested for stability in a way similar to the emulsified asphalt discussed above.

3.6.5.2 Curing Conditions

The emulsified asphalt depends on the evaporation of water (kerosene in cutback) for development of their adhesion characteristics. Water evaporation can be fairly rapid under favorable weather conditions but low temperature or rainfall soon after construction can retard proper curing of in-service pavements. Laboratory curing conditions must reflect, in a comparatively short period of time, the long-term environmental influences to which the compacted mix will be subjected. Such curing will, in turn, influence the physical properties of the mix and thus the pavement performance. In this study, the specimens with emulsion were allowed to cure in the

oven at 100°F (38°C) inside the mold for 24 hours followed by 48 hours of curing outside the mold.

The specimens with cutback were cured in the oven at 140°F (60°C) inside the mold for 24 hours followed by oven curing at 122°F (50°C) outside the mold for another 48 hours (The Asphalt Institute, 1977). Finally, all samples were vacuum desiccated at 3.9 in. Hg for 24 hours as per the recommendations of the Illinois method of design. ASTM D2726 was used to obtain the bulk specific gravity of the cured specimens. The specimens were then tested for Marshall stability at a room temperature of 73°F (23°C).

3.6.5.3 Marshall Stability Test

After determining the bulk specific gravity, Marshall stability test was carried out using the Marshall stability testing machine, at room temperature. The specimens were loaded at a constant rate of deformation of 2 in. per minute until failure occurred. The maximum load at failure was recorded as the Marshall stability, which was corrected for standard specimen thickness, if necessary.

3.6.5.4 Creep Test

Creep test was performed, as per the Shell procedure, on emulsion and cutback asphalt specimens prepared by Marshall compactor following the procedure presented in section 3.6.6.1. The test was carried out at temperatures of 21°C, 35°C, and 50 °C.

Each specimen was tested at a stress level of 414 kPa (60 psi) for 2 hours, one hour loading and one hour unloading. The measured vertical deformations at different loading and unloading periods were recorded using a data logger. The data logger was set to take the readings of vertical deformations every minute and was connected to a computer where all the data were stored for further analysis.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Characterization of Collected Samples

The sand samples were collected from ten (10) different sites distributed all over eastern Saudi Arabia. Sampling areas for sand included Dammam, Baggah, Dhahran, Abqaiq, Dahana, Jubail, and Al-Qurayyah. In addition, the marl used was obtained from Abu Hadriyah area, eastern Saudi Arabia. Some of these sand sites are being used as a borrow area by contractors, while the marl is heavily used in roads construction and as a foundation material.

The surface deposits or formations in the Eastern Province belong to different geological periods as shown in Figure 2.2. Thus, the soils from these formations contain specific minerals reflecting their origin. The behavior of these materials is expected to vary.

Collected sand samples were subjected to a preliminary testing program to explore their general properties including specific gravity, grain-size distribution, classification, maximum and minimum density values, peak friction angle and mineralogical composition using the X-Ray Diffraction.

The soils were classified according to AASHTO and USCS. The physical properties of sands are shown in Table 4.1. The grain size distribution for the collected samples are plotted in Figures 4.1 to 4.12. In these Figures, there are no large variations neither in the sand sample types nor in the grain size distribution for both dry and wet sievings. This similarity is mainly occurring because sand is made up of quartz under similar formation and weathering conditions.

The result revealed that the collected sands samples are classified as A-1-b and A-3 according to the AASHTO and SP according to the USCS. The specific gravity values are ranging from 2.62 to 2.70 whereas the minimum dry density values were ranging from 1.52 to 1.64 g/cm³ and the maximum dry density values from 1.79 to 1.91 g/cm³. The coefficient of uniformity, C_u using the dry and washed sieving is ranging from 1.54 to 3.19 and from 1.88 to 3.89, respectively. The corresponding values for the coefficient of curvatures, C_c , are ranging from 0.94 to 1.15 and 0.94 to 1.27. The peak friction angles, ϕ , is ranging from 40 to 44 degrees as an average value of two tests for each sand.

Mineralogical analyses were also performed for some of the collected sand samples and the results are shown in Table 4.2. These results indicate that the silica content in the Eastern Province sands ranged from 83% to 100%. In addition to quartz, calcite and gypsum were also present in trace.

Table 4.1: Physical properties of the collected sands

Sand Name & Place	Abbreviation Symbol	Classification [§]		Coefficient of Uniformity, C _u		Coefficient of Curvature, C _c		Specific Gravity, G _s	Peak* Friction Angle (deg.)	Dry Density (g/cm ³)	
		USCS	AASHTO	Dry	Washed	Dry	Washed			Min	Max
Dammam Sand-1	S-Dam1-D	SP	A-1-b	3.19	3.57	0.96	1.12	2.69	40	1.63	1.91
Bagah Sand-1	S-BAG1-D	SP	A-3	2.06	2.29	0.94	0.94	2.66	44	1.61	1.86
Abqaiq Sand-1	S-ABQ1-D	SP	A-3	2.5	2.91	0.99	1.14	2.67	40	1.61	1.87
Dahran Sand-1	S-DHA1-D/B	SP	A-3	2.07	2.5	1.05	1.18	2.66	41	1.53	1.82
Dahran Sand-2	S-DHA2-D/B	SP	A-3	2.07	2.55	1.11	1.3	2.67	42	1.59	1.83
Dahran Sand-3	S-DHA3-D/B	SP	A-3**	3.13	3.68	1.05	1.09	2.68	42	1.64	1.91
Qurayyah Sand-1	S-Qur1-D	SP	A-3	3.18	3.89	1.15	1.27	2.65	43	1.57	1.84
Qurayyah Sand-2	S-Qur2-D/B	SP	A-3	1.94	2.58	0.94	1.19	2.65	43	1.57	1.83
Dahna Sand-1	S-DAH1-D	SP	A-3	1.6	1.9	0.96	1.17	2.70	42	1.52	1.79
Jubail Sand-1	S-JUB1-D	SP	A-1-b	1.54	1.88	1.02	1.19	2.62	40	1.57	1.82

* Conventional Triaxial Test at 75% relative density at confining pressure of 69 kPa

§ Based on both dry and washed sieve results

** Samples classification is A-1-b based on washed sieve

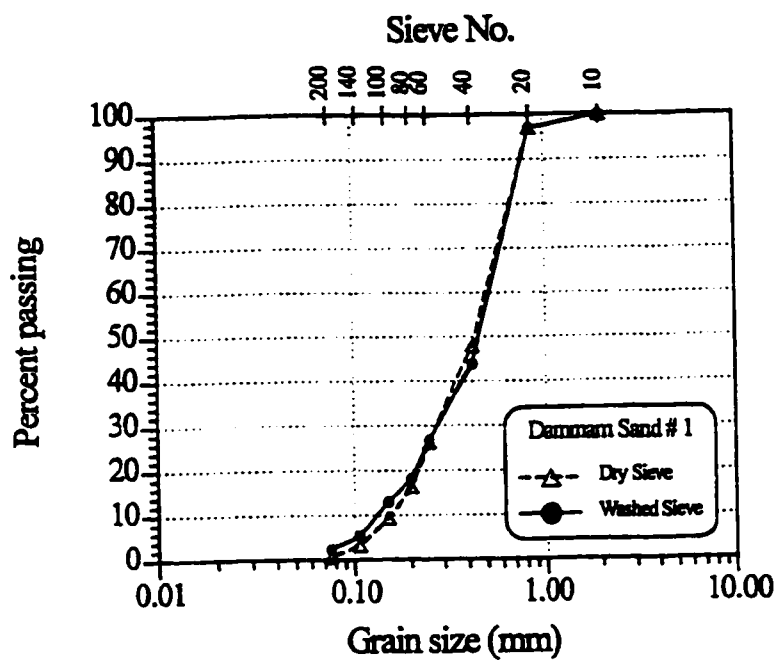


Fig. 4.1: Dry and washed grain size distribution for Dammam Sand # 1

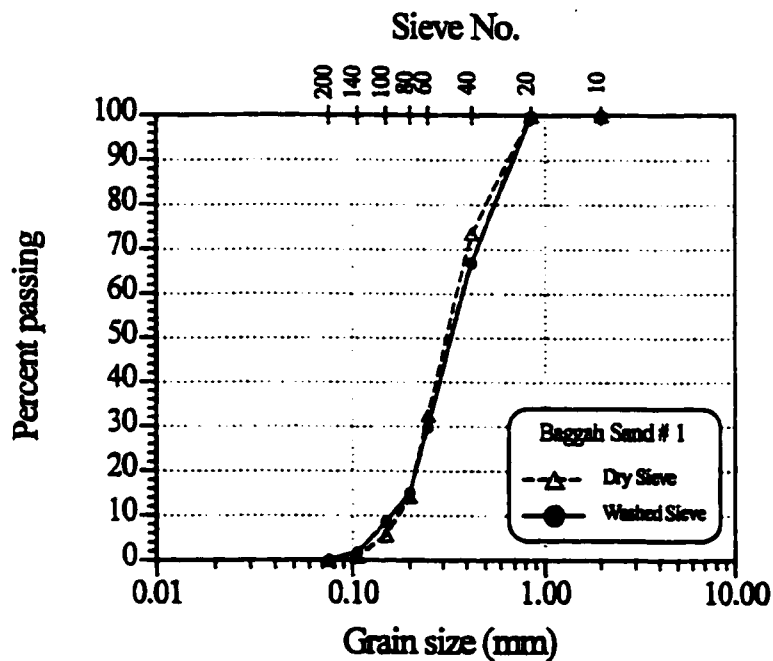


Fig. 4.2: Dry and washed grain size distribution for Baggah Sand # 1

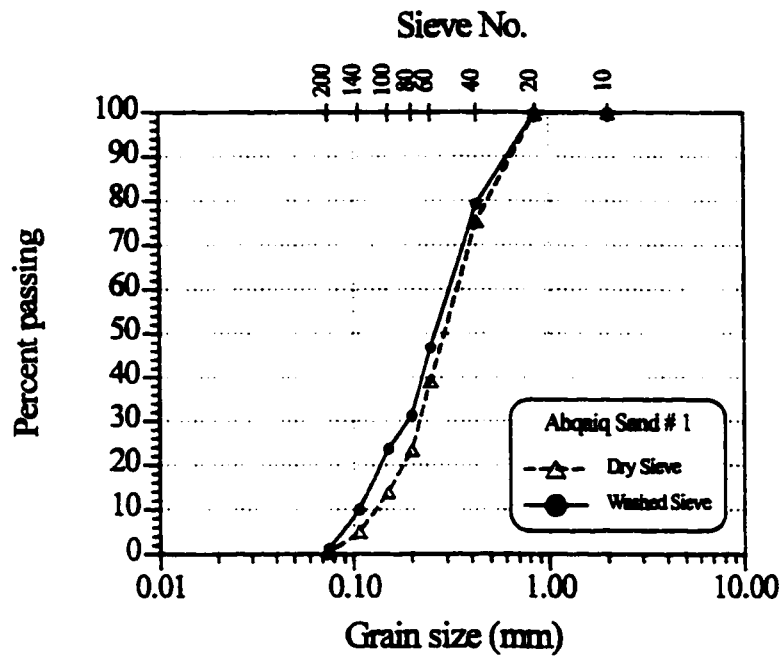


Fig. 4.3: Dry and washed grain size distribution for Abqaiq Sand # 1

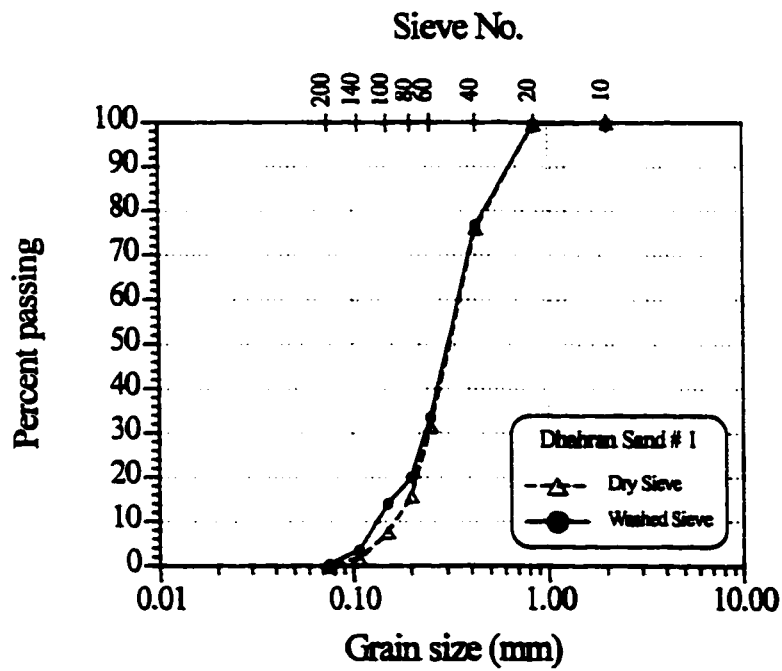


Fig. 4.4: Dry and washed grain size distribution for Dhahran Sand # 1

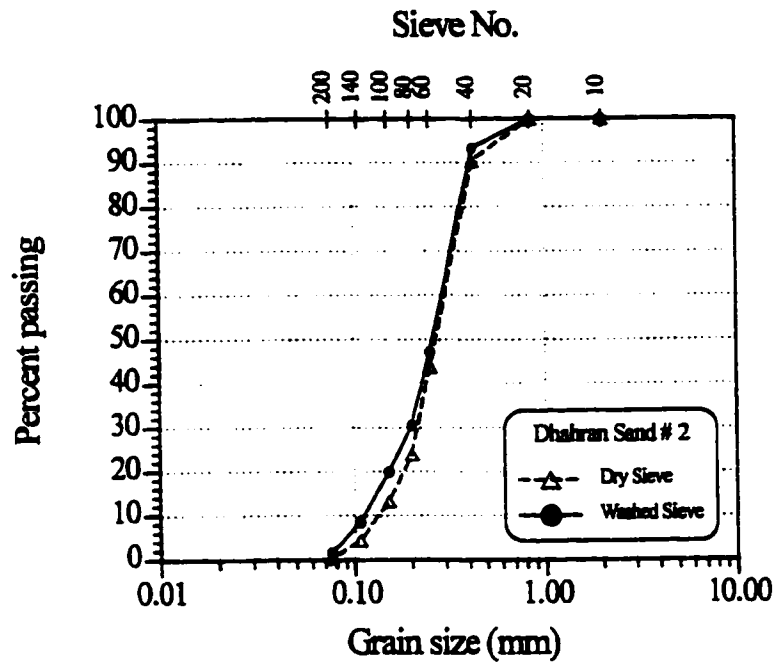


Fig. 4.5: Dry and washed grain size distribution for Dhahran Sand # 2

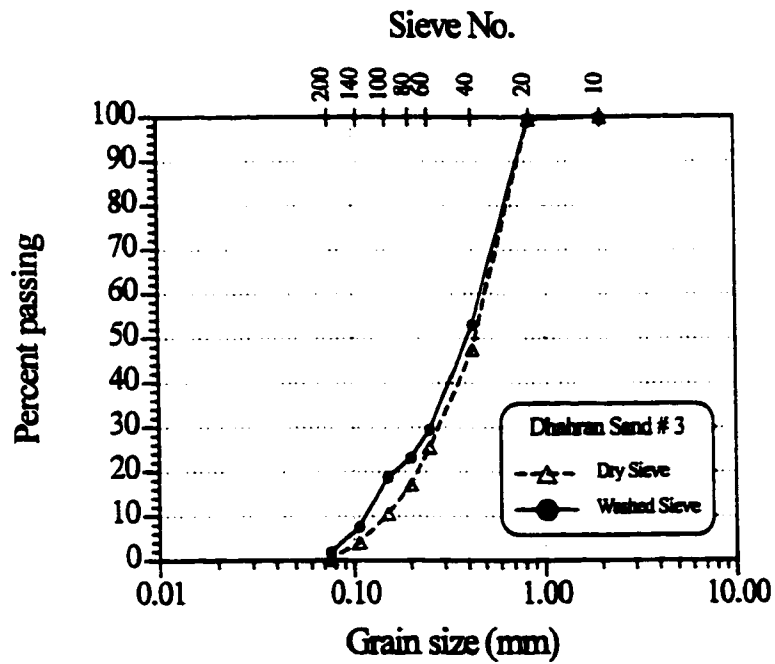


Fig. 4.6: Dry and washed grain size distribution for Dhahran Sand # 3

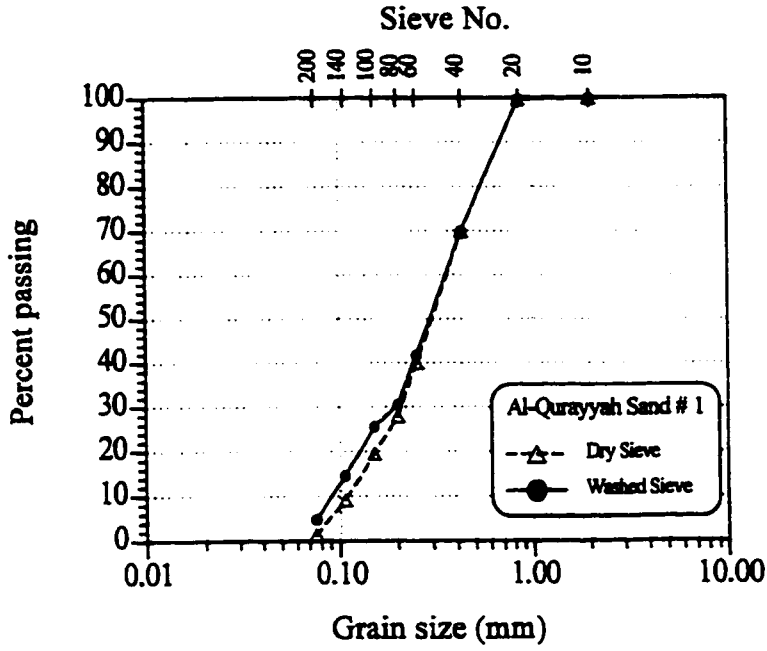


Fig. 4.7: Dry and washed grain size distribution for Al-Qurayyah Sand # 1

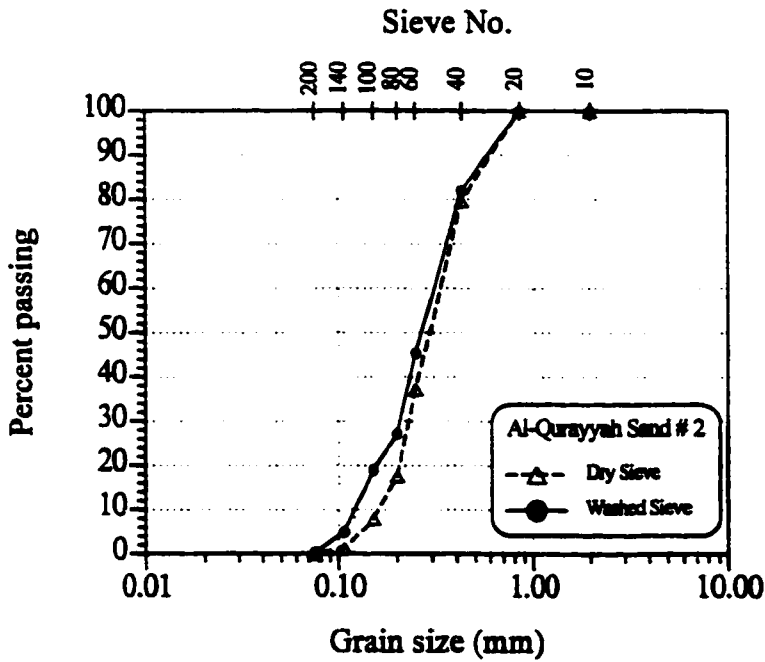


Fig. 4.8: Dry and washed grain size distribution for Al-Qurayyah Sand # 2

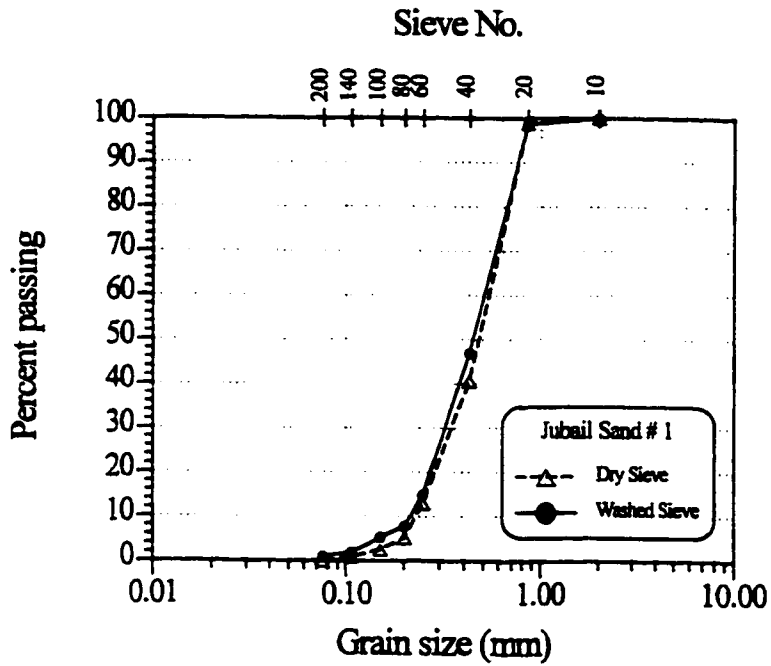


Fig. 4.9: Dry and washed grain size distribution for Jubail Sand # 1

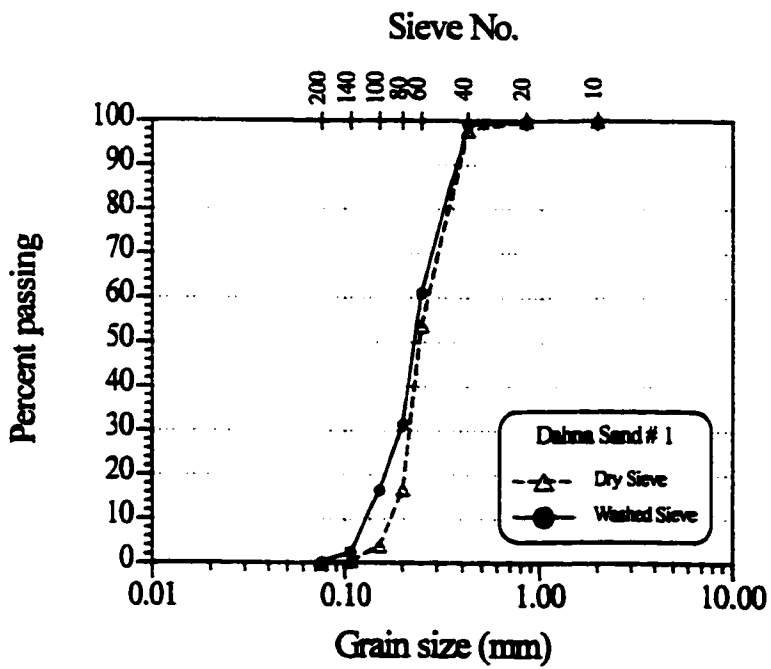


Fig. 4.10: Dry and washed grain size distribution for Dahna Sand # 1

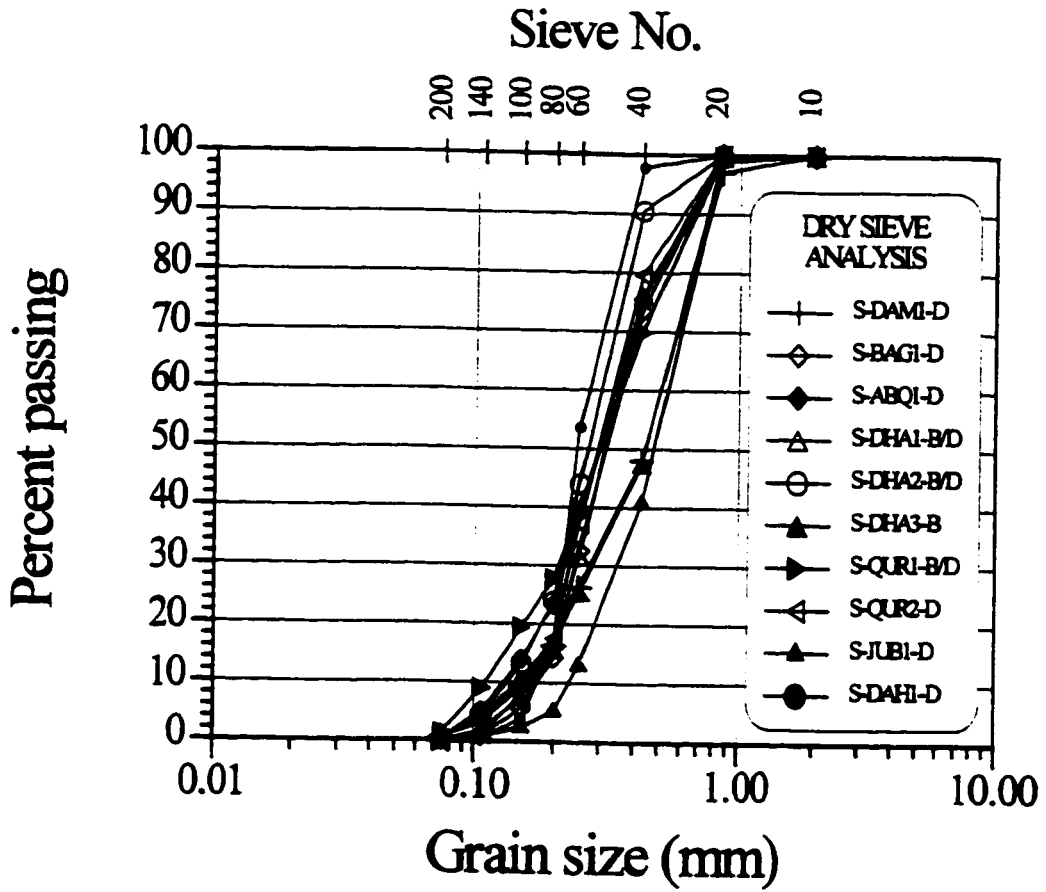


Fig. 4.11: Dry grain size distribution for sand

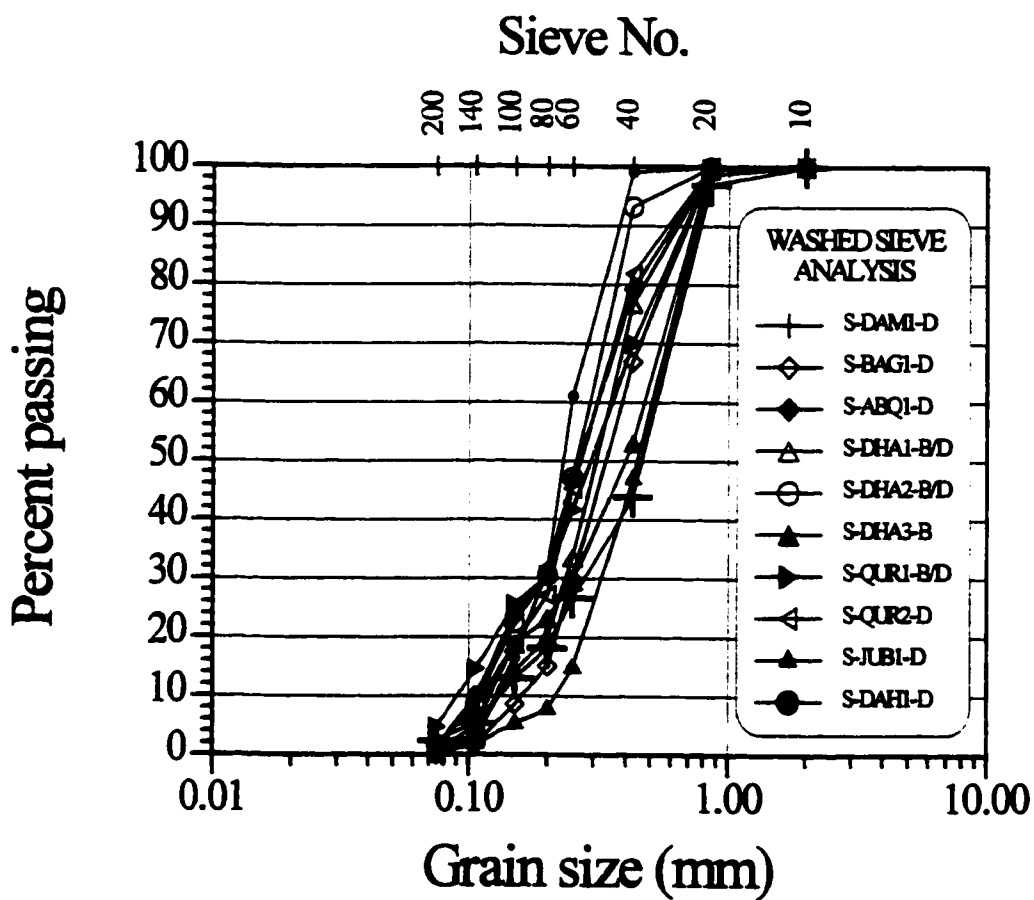


Fig. 4.12: Washed grain size distribution for sand

Table 4.2: Approximate crystalline minerals (mineralogical composition) of some of collected dune sands

Sand \ Mineralogy	Dhahran Sand-1	Al-Qurayyah Sand-1	Baggah Sand	Dahna Sand
SiO ₂ (quartz)	84	83	89	100
CaSO ₄ .2H ₂ O (gypsum)	4	14	4	–
CaCO ₃ (calcite)	12	3	7	–

4.2 Characterization of Sand-Marl Mixes

4.2.1 General

It was noticed that the physical and chemical properties of the different sands were similar, except for Al-Qurayyah Sand-1, which does not exist in commercial quantities. This encouraged the author to select only two sands from two different environments, namely desert and beach, for the detailed stabilization study. Therefore, Baggah Sand-1 (S-BAG1-D) and Dhahran Sand-2 (S-DHA2-D/B) were selected for the detailed study. Fine gradation clearly revealed that these sands can not be used as a foundation material in their natural condition. However, when improved, they can be utilized in many applications (Al-Sanad et al., 1993). The selected dune sands were blended with Abu Hadriyah marl to determine the geotechnical properties of such mixes and come up with a mix which is a suitable construction material.

4.2.2 Modified Proctor Compaction Tests

The moisture-density relationship reflects the behavior of soils during compaction. The soils to be used as construction materials need to be compacted to a certain dry density at a specific moisture content. These two parameters control the structure of the soil which directly relates to the properties of the soil such as strength, compressibility, and permeability. The compaction curves for the different sand-marl mixes are presented in Figure 4.13. It is noticed that the maximum dry density varied from 1.68 to 2.20 g/cm³, and the optimum moisture content (OMC) varied from 6 to

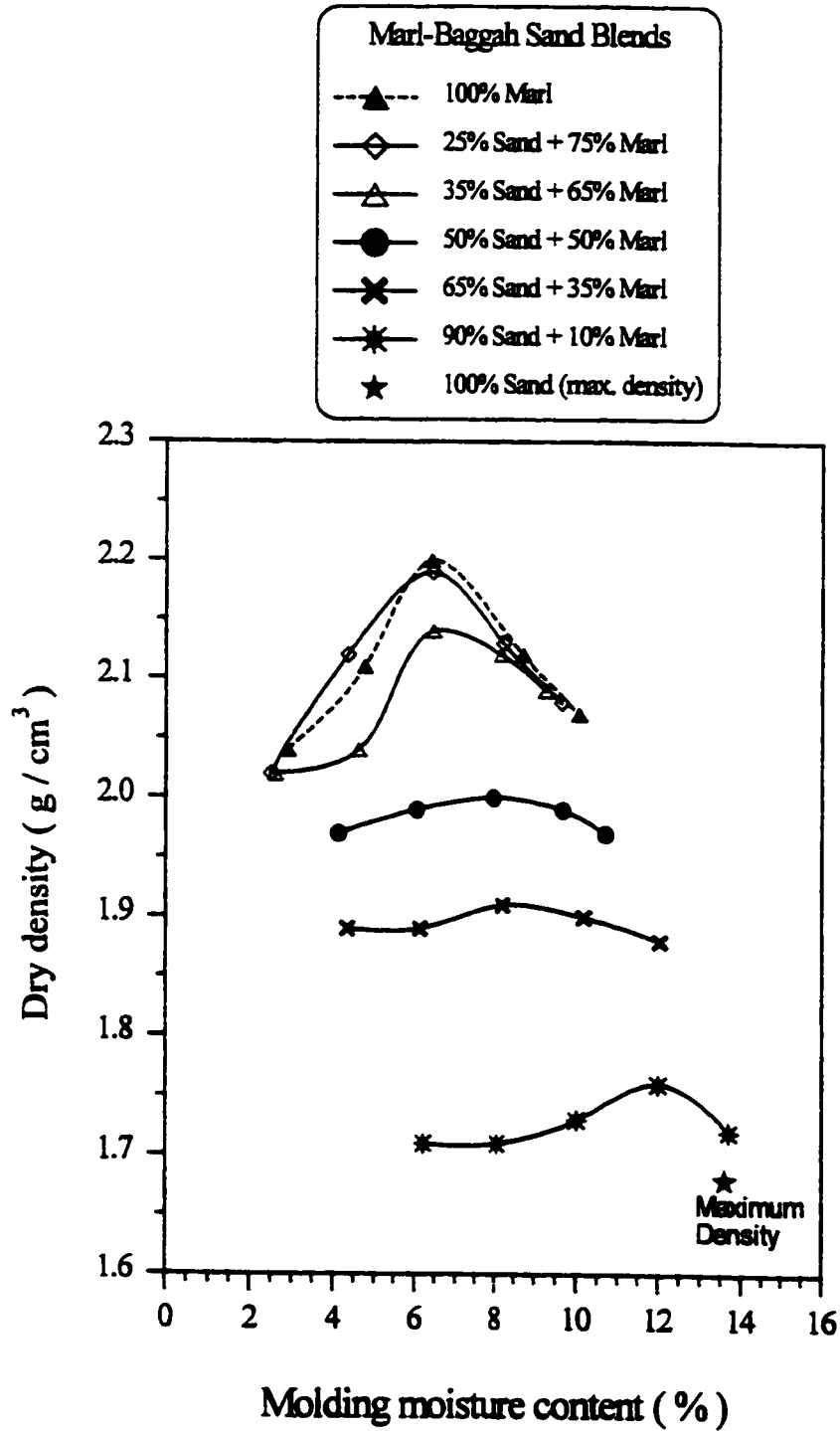


Fig. 4.13: Moisture-density relationships for sand/marl mixes at different ratios

13.6%. The maximum dry density, $\gamma_{d \max}$, increases as the marl content increases. This increase in $\gamma_{d \max}$ is associated with a decrease in the corresponding OMC. This is mainly attributed to: (1) the improved gradation of the mix, (2) the cohesive nature of the marl and (3) the presence of large size aggregate in the marl (up to 19 mm), which reduces the surface area. The variability in these parameters reflect the variability of the mineral constituents, grain-size and their relative percentage. In general, higher OMC and lower dry density are associated with poorly-graded soils. This is the case with increasing sand content.

4.2.3 California Bearing Ratio (CBR) Test

In general, soils compacted on the dry side of optimum moisture content (OMC) bear higher strength than those compacted on the wet side of optimum. The higher strength for samples compacted on the dry side of optimum moisture content is attributed to the formation of large-sized strong clods (macropeds) which provide high frictional resistance and to the small increase in effective stresses due to suction. In addition, the dry cohesive samples have higher strength compared to those with higher moisture content. On the wet side of optimum, the macropeds got smaller and weaker causing a reduction in the cohesion and thus in the strength. However, the higher density at the optimum moisture content and the intermediate macropore and macropeds result in a higher strength which is partially attributed to the relatively low moisture content (Ahmed, 1995).

The CBR results are shown in Figure 4.14 and they indicated that when the marl content in the mix is equal to or higher than 50%, the maximum CBR values are obtained either at the optimum or very close to the optimum on the dry side. Further reduction in the moisture content below the optimum causes a strength decrease. This behavior of marls on the dry side of optimum may be attributed to the absence of cohesive material and the large macropores, and therefore the macropeds become unstable, friable and relatively weak. As the moisture content decreases below the optimum, the interlocking decreases due to the reduction in dry density. However, at a moisture content at or near the OMC, denser macropeds give high interlocking, which is responsible for the strong and stable soil mass. Since there are no cohesive fines in the soil, the strength is derived only through partial interlocking. Similarly on the wet side of optimum, the non-cohesive carbonate fines form loose lumps and the excess water results in a loss of cementation near the contact points. The soil mass becomes mud-like lumps with no bearing strength. The gravel-sized aggregate will just float in the loose matrix (lumps) of the fine particles.

The strength (CBR) of soil mixes varies widely over a narrow range of moisture content. At or near the optimum moisture content on the dry side, the CBR values are well above 90%, indicating a quit good strength. However, in most cases, increasing the moisture content above the optimum by 1% causes complete loss of the soil bearing strength.

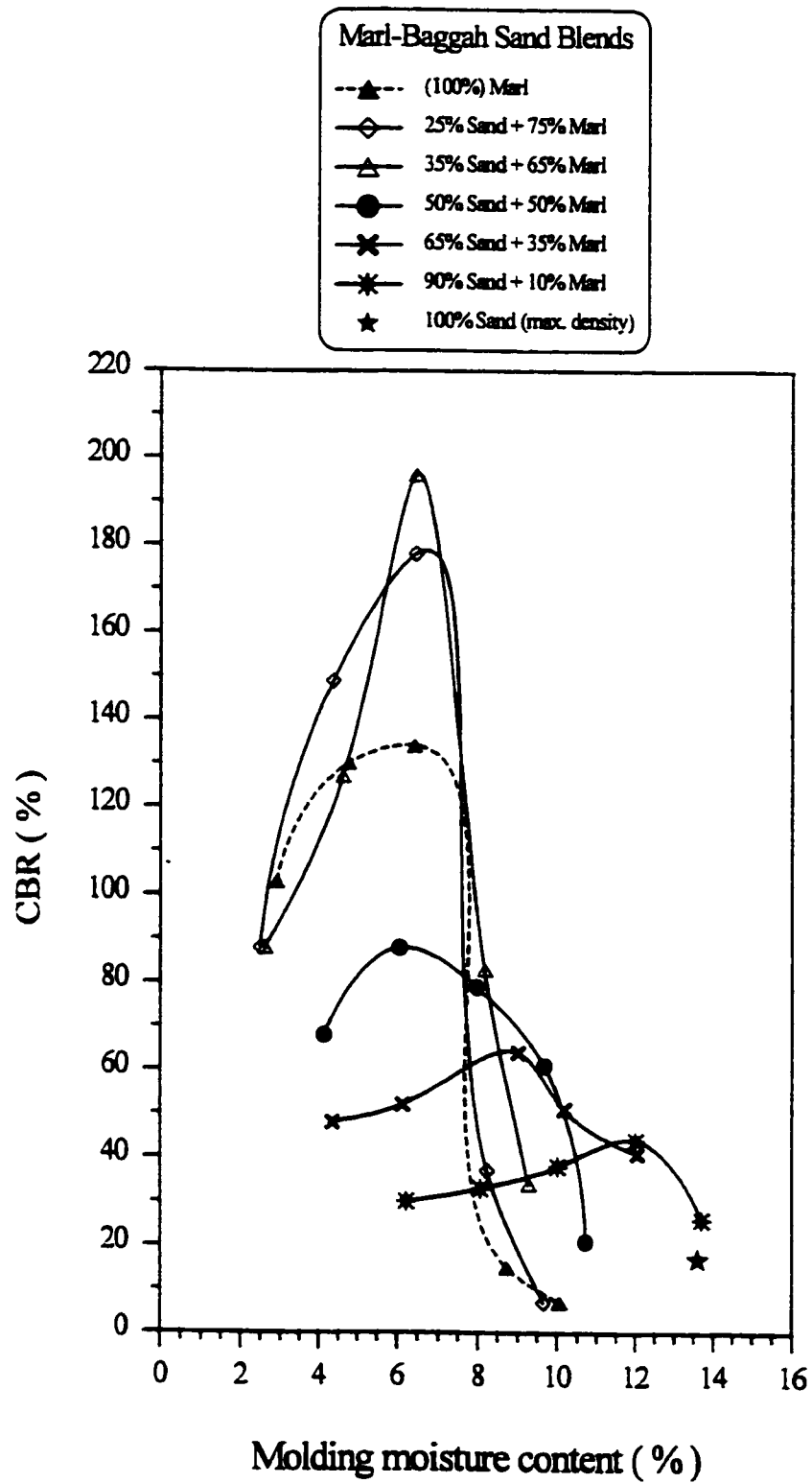


Fig. 4.14: CBR-moisture content variations of sand/marl mixes at different ratios

In the characterization stage, the variation of CBR with the molding moisture content was taken as an indication of the strength behavior for the sand-marl mixes as shown in Figure 4.14. Baggah dune sand and Dhahran sand-2 have been mixed separately with Abu Hadriyah marl at different ratios and optimized based on CBR (dry and soaked) values as shown in Figures 4.15 and 4.16 for the two sands, respectively. The results reveal that the CBR value changes significantly with sand-marl mix ratios. It shows that a blend consisting of 35% Baggah sand and 65% Abu Hadriyah marl at the optimum moisture content (OMC) gives the highest CBR value. A similar behavior was noticed for Dhahran sand-2. It was therefore decided to continue the investigation using Baggah dune sand only. The gradation and classification for different sand-marl blends are shown in Table 4.3 and Figure 4.17. All sand-marl blends (except for pure marl) are classified either as A-1-b or A-3 according to AASHTO and as SP according to USCS.

4.3 Stabilization of Sand-Marl Mixtures

Mineralogically, sands are mostly quartz, which is considered the most resistant mineral to chemical and mechanical degradation. Thus, the water sensitivity of the soil is low. On the other hand, marls exhibit wide variations in terms of origin, color, mineral composition, plasticity and other engineering properties. Almost all types of marl are lacking hardness and resistance to disintegration in water (Bayomy, 1988). The primary constituents of marls are calcite and dolomite. Therefore, the use of marls in construction should consider their poor performance under harsh environmental and loading conditions.

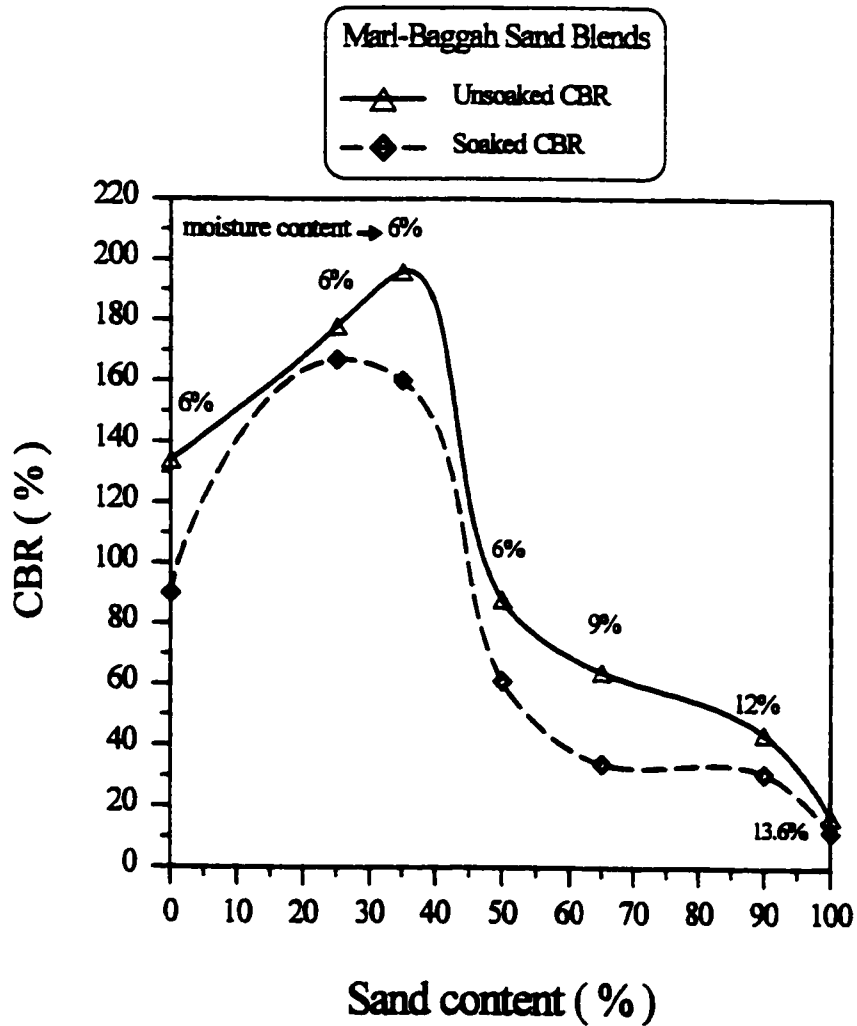


Fig. 4.15: Variation of the maximum CBR values with Baggah sand content (the molding moisture content shown corresponds to the maximum CBR value)

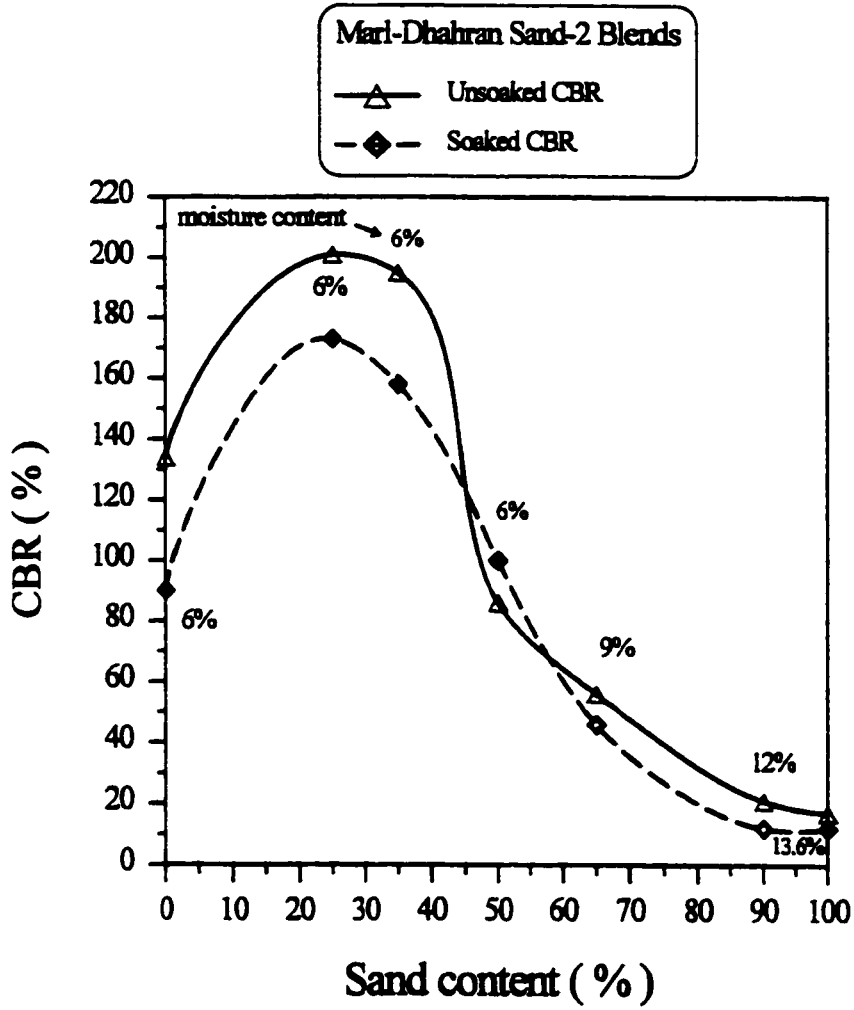


Fig. 4.16: Variation of the maximum CBR values with Dhahran sand-2 content (the molding moisture content shown corresponds to the maximum CBR value)

Table 4.3: Classification of the sand/marl mixes (Baggah sand mixed with Adu Hadriyah marl)

Mix Type (%)	Classification	
	AASHTO	USCS
100% Marl + 0% Sand	A-1-a	GP
75% Marl + 25% Sand	A-1-b	SP
65% Marl + 35% Sand	A-1-b	SP
50% Marl + 50% Sand	A-1-b	SP
35% Marl + 65% Sand	A-3	SP
10% Marl + 90% Sand	A-3	SP
0% Marl + 100% Sand	A-3	SP

The engineering properties of sands as well as marls can be improved using chemical admixtures such as portland cement and bitumen. Researches have shown that the incorporation of small amount of additives to these soils results in a high strength gain of the mix. The strength gain is highly dependent on the type of soil and the type and amount of additive. The success of a certain additive is reflected by the strength gain and the durability of the material. It was therefore decided to investigate the effect of different additives on the performance of the stabilized material using both strength and durability tests.

4.3.1 Stabilization using Portland Cement

Optimization of cement-soils mixes involves mixing of pulverized soil, cement and water, and compacting this mixture to a high density, which renders the material resistant to various physical, and chemical stresses. As the cement hydrates, a gel is formed which, upon hardening, forms a cellular matrix that encapsulates the soil particles or forms strong bridges between the aggregate, thus producing a hard, durable structural material.

Sulphate resisting (Type V) cement, which is commonly used for maximum sulfate attack resistance, was selected as the stabilizer. In the sand-marl mixes, additive contents of 3, 5, and 7% cement were used. The effect of different parameters on the performance of the stabilized sand-marl mixes are discussed in the following sections.

a) Cement Content for Strength Requirements

For each cement content, the modified Proctor compaction test was performed and the results are shown in Figure 4.18. It is clearly seen that the maximum dry density for these mixes ranges from 2.12 to 2.18 g/cm³ and the optimum moisture content is almost the same for all mixes (6.5~7%). The variation of the CBR values with cement content for samples prepared at the optimum moisture contents is plotted in Figure 4.19. Samples for the unconfined compressive strength (UCS) were prepared at different moisture content for 0, 3, 5, and 7% cement contents. The variations of UCS values, after 7 days of exposed curing, are plotted in Fig. 4.20. Samples were also compacted at the optimum moisture content with 0, 3, 5, and 7% cement contents. Some of the compacted samples were sealed in plastic sheets, while others were left exposed. All samples were allowed to cure for 7 days at room temperature (21±2 °C). The variation of unconfined compressive strength (UCS) with cement content is plotted against the cement content in Figures 4.21.

The results reveal that the strength increases as the cement content increases from 0 to 7% in both CBR and UCS tests. Moreover, the strength of the exposed samples is higher than that of the sealed ones, as the cement content increases from 0 to 5%. Further increase in cement content to 7% and while exposed and sealed samples continued gaining strength, sealed samples exhibited higher strength than exposed ones. The UCS at a cement content of 5%, was 6668 and 6600 kPa for exposed and sealed conditions,

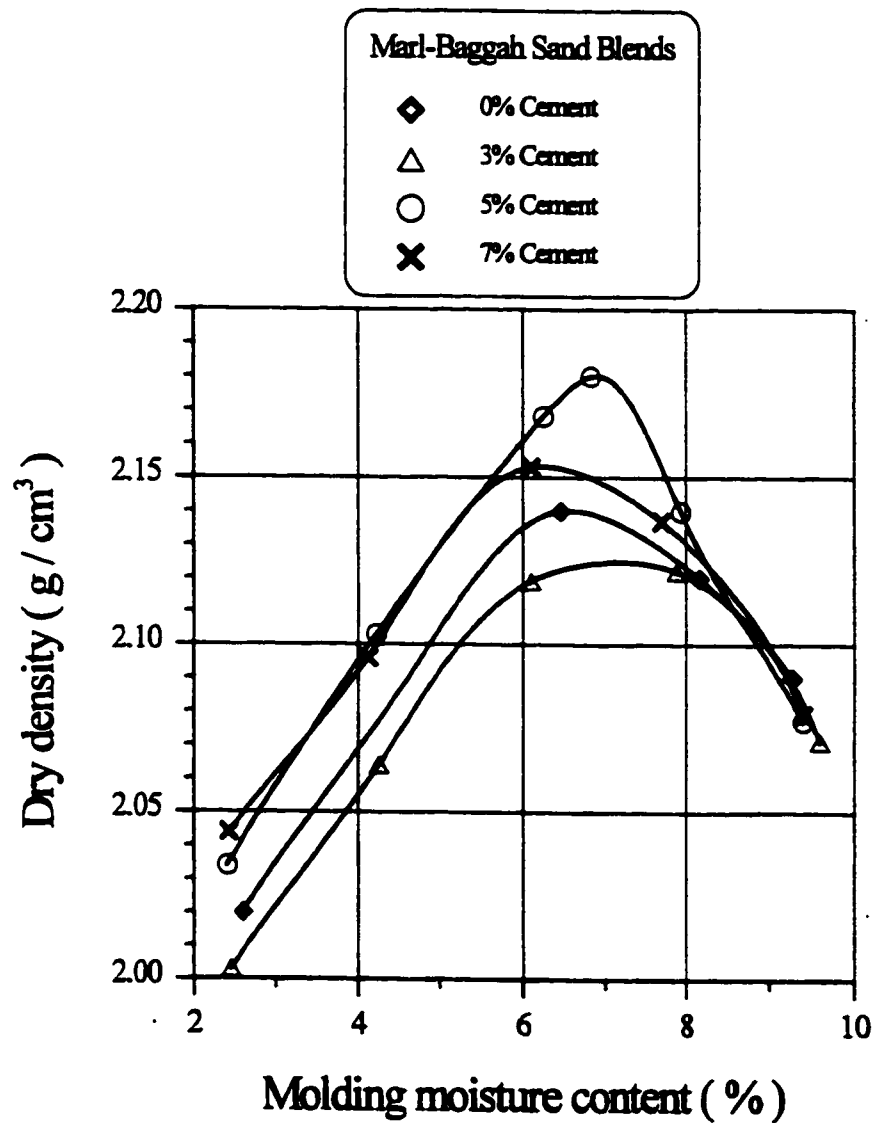


Fig. 4.18: Variation of the dry density with moisture content for Baggah sand/Abu Hadriyah marl mixes treated with different percentages of cement content

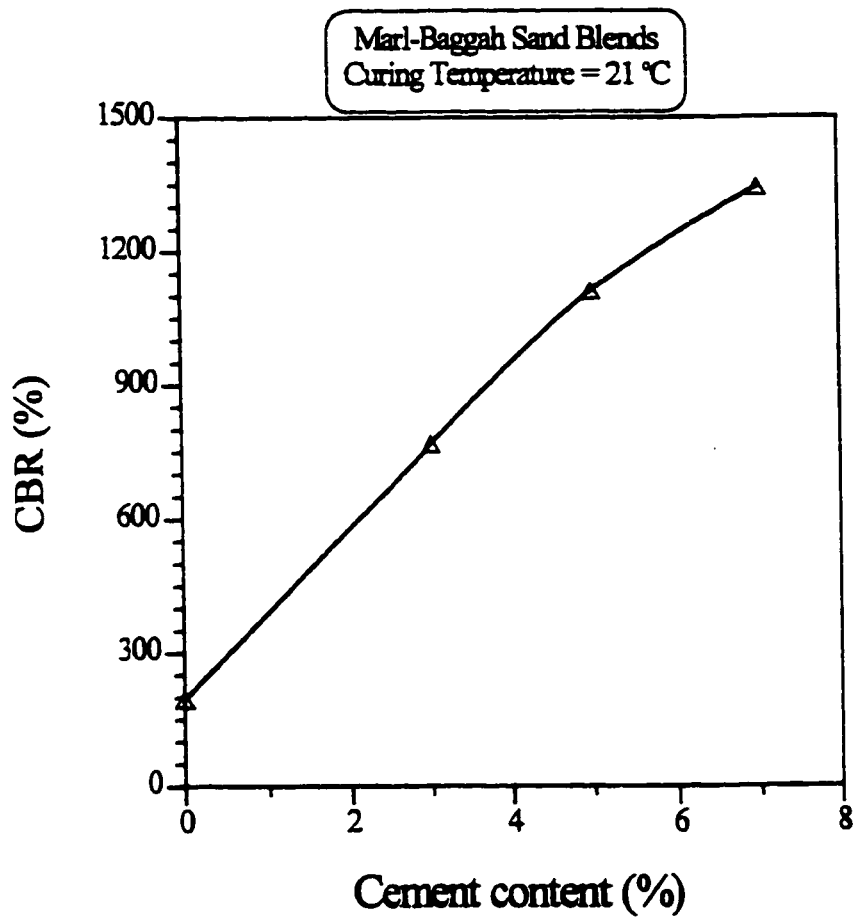


Fig. 4.19: Variation of CBR with varied cement contents

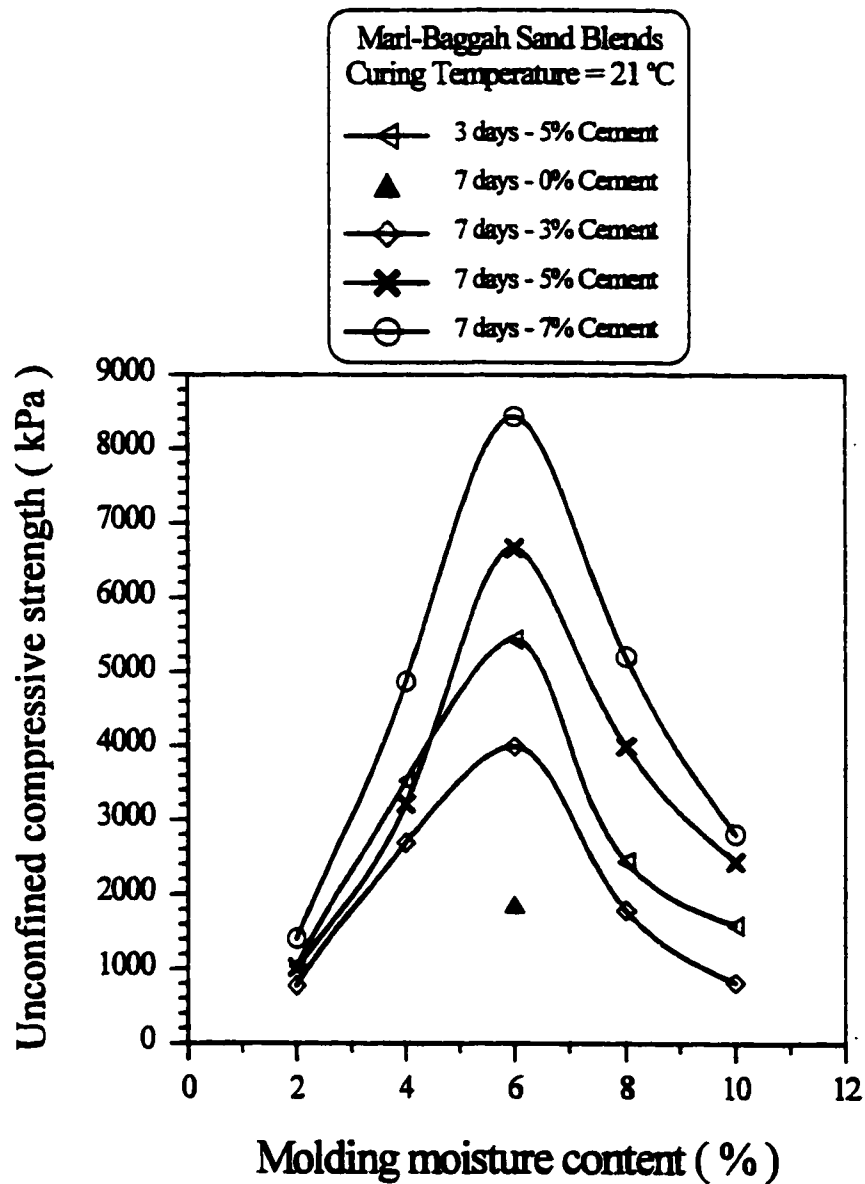


Fig. 4.20: Variation of the UCS with moisture content for Baggah sand/Abu Hadriyah marl mixes treated with different percentages of cement content, and allowed to cure under exposed condition

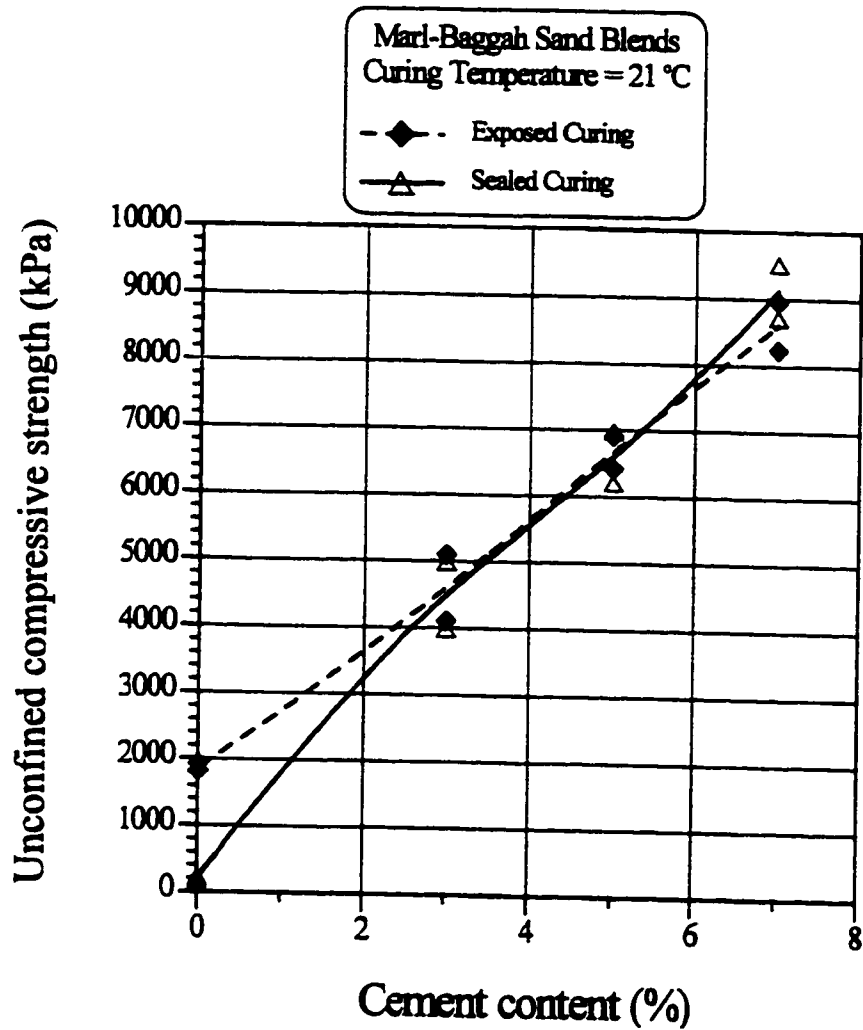


Fig. 4.21: Variation of the UCS with cement content for sand/marl mixes samples cured for 7 days

respectively. Since the field curing conditions can be considered somewhat between these two extremes (exposed and sealed), it seems appropriate to take the UCS value at any cement content as the average of two extreme values. Based on the above perception, the UCS value is 6634 kPa. Further, a factor of 1.10 is to be used to get the UCS corrected for the height to diameter (h/d) ratio of 1.15 as recommended in the ASTM standards. It results in UCS value of 7300 kPa (1060 psi). According to the ACI Committee 230 report (1990), the minimum 7-day UCS specified for the base course construction by the US Army Corps of Engineers (USACE) is 5175 kPa (750 psi). The requirement for UCS is fulfilled at a cement content of 5% for sand-marl mix and the strength is higher than the required value by more than 40%. Therefore, the required cement content, based on UCS was chosen to be 5% for the sand-marl mix.

b) Cement Content for Durability Requirements

The soil-cement fulfilling the minimum strength requirement should also be durable against wetting and drying and freeze and thaw cycles. The durability against freezing and thawing need not be investigated because the seasonal and daily temperature variations of eastern Saudi Arabia are not simulated by such test.

The standard durability test (ASTM-D 559) and modified slake durability test were conducted on soil-cement having 3-5-7% cement contents. The results obtained from the durability tests are shown in Figure 4.22. The data is presented in terms of the

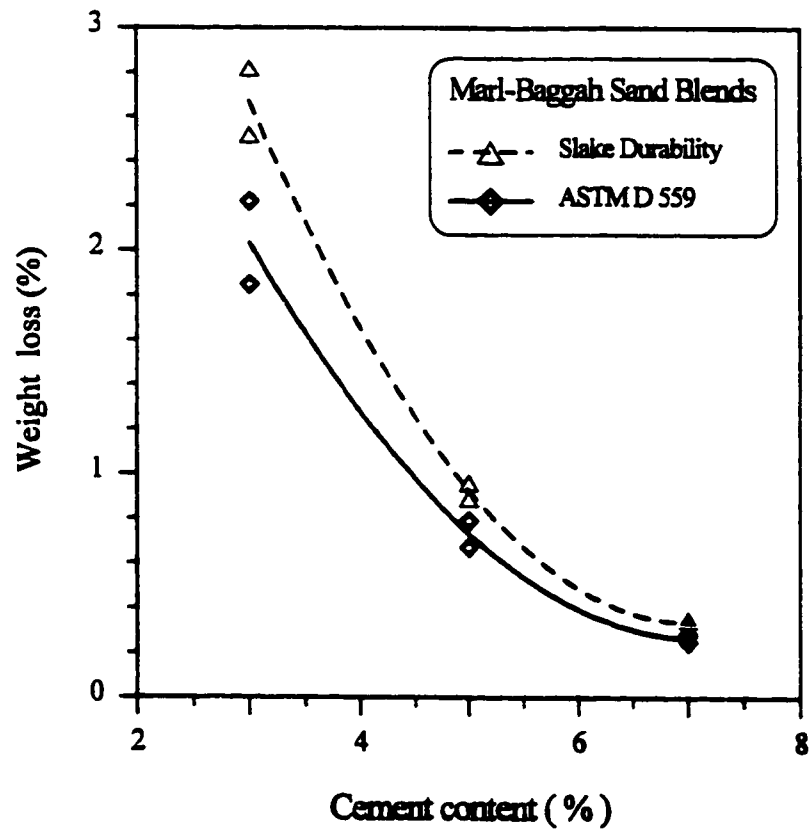


Fig. 4.22: Variation of the weight loss with cement content for marl/Baggah sand Blends after 12 cycles of repeated wetting and drying

weight loss of soil-cement specimens for different cement contents. The maximum weight loss occurs at a cement content of 3% and it decreases as the cement content increases. The weight loss for a cement content of 5% is 0.9 and 0.7% in the Slake test and ASTM test, respectively. This is far below the maximum allowable weight loss of 14%, according to the Portland Cement Association (PCA) and 11% according to US Army Corp of Engineers (USACE) for soils classified as SP and soils having $PI < 10$, respectively (ACI Committee, 1990). Therefore, a cement content of 5% is considered appropriate not only from strength perspective, but also from durability and economy standpoints. It is worth mentioning that hairline cracks have been observed in almost all samples for durability tests as shown in Figure 4.23. This is probably due to the cohesionless nature of the mix because of the high sand content compared to pure marl. These cracks, however, were not deep and seem to be very surficial.

In addition, weight loss of soil-cement specimens for 5% cement content at different sand-marl ratios is presented in Figure 4.24. The weight loss seems to be independent of the sand content however, the slake durability test showed slight increase in the weight loss with the increase in the sand content. The maximum weight loss occurs at a sand content of 35% (maximum ratio used) in the slake test and it decreases as the sand content decreases. However in the standard test, there was no change in weight loss with varying the sand contents. This small weight loss indicates that the 5% cement provided strong bonding between the soil particles regardless of the sand contents.

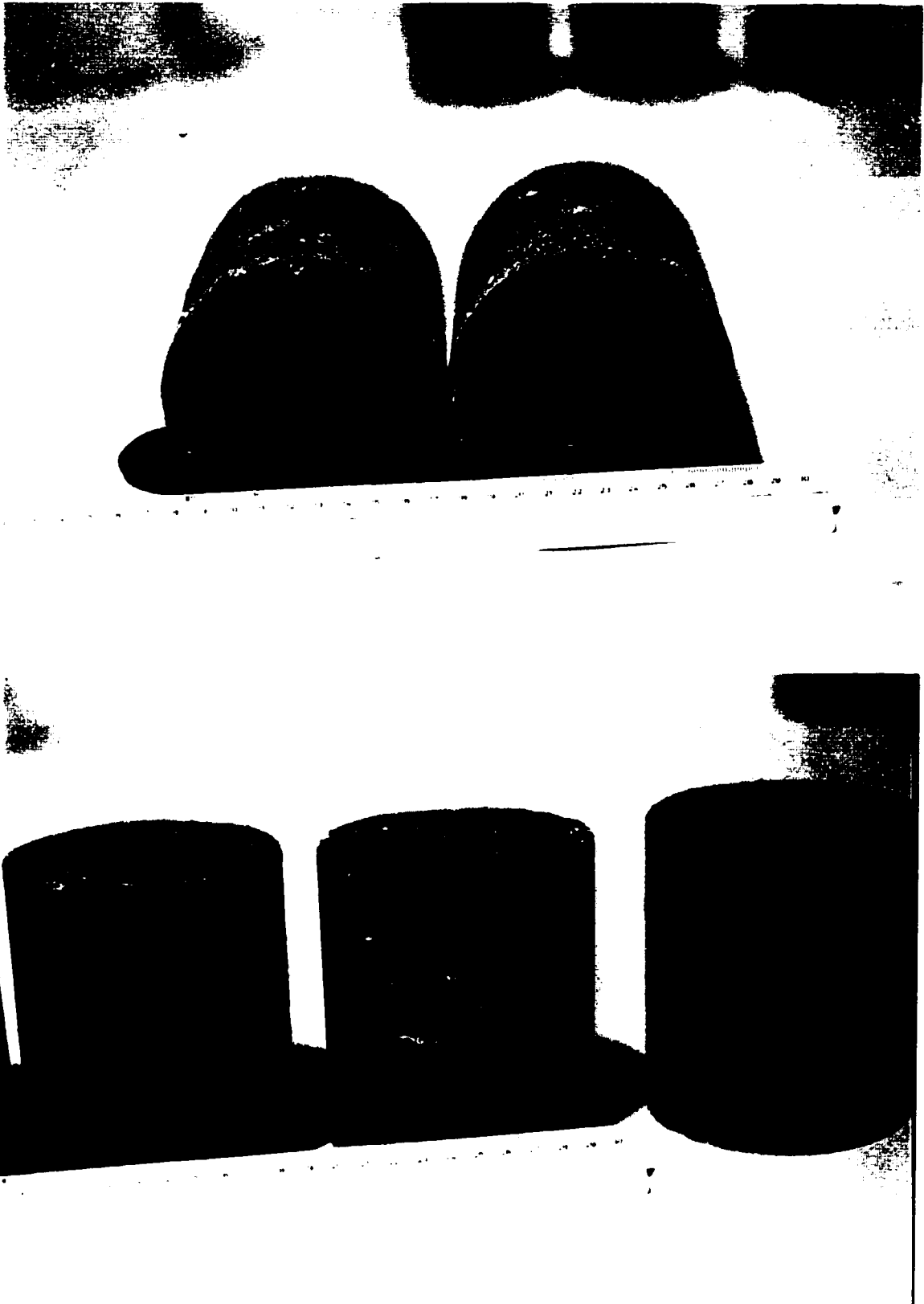


Fig. 4.23: Durability samples with hairline cracks for marl/Baggah sand mixes treated with 5% cement

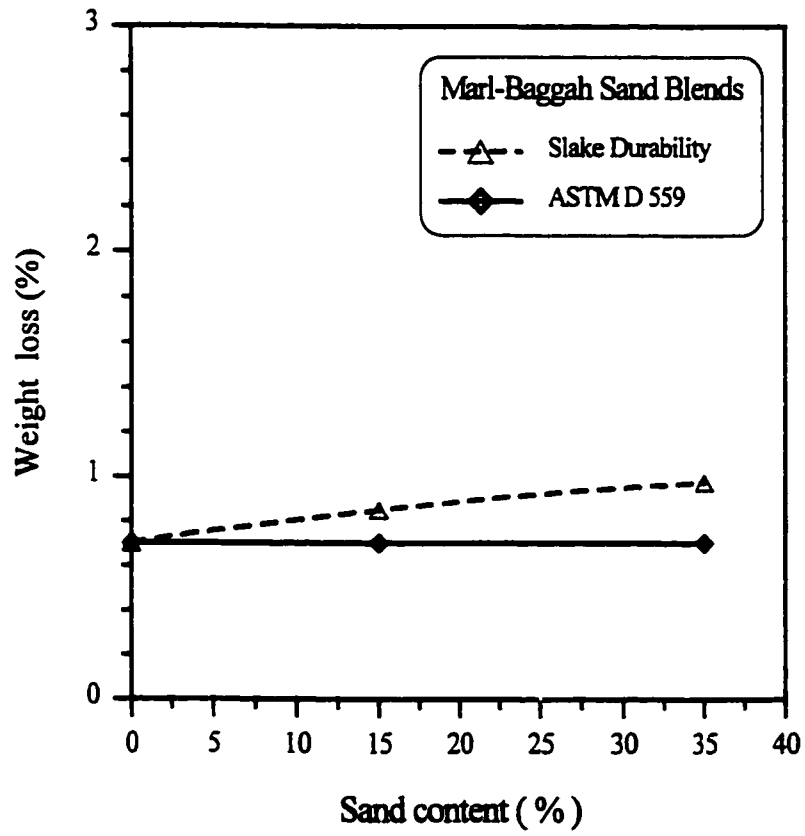


Fig. 4.24: Variation of the weight loss with sand contents for sand/marl mixes treated with 5% cement.

4.4 Cement Stabilized Sand-Marl Mixtures

Further investigation was conducted on the stabilized sand-marl mixes to study the effect of other parameters that control the behavior of these soils. In particular, the effects of curing regime, and age on the unconfined compressive strength and the modulus of resilience were investigated and the results are summarized below.

4.4.1 Unconfined Compressive Strength

The UCS was adopted to study the effect of different parameters such as curing temperature, exposure conditions, and age on the strength gain of the treated mixes.

a) Effect of Exposure Conditions

Sealed and exposed regimes were adopted in the laboratory and chosen to simulate the two extreme curing conditions in the field. In the sealed curing, samples were completely wrapped with several layers of plastic sheets right after their extrusion from the mold; whereas in the exposed curing, samples were kept exposed to the laboratory environment. The sealed curing condition adopted in the laboratory simulates the proper curing procedures in the field where moisture loss is minimized, whereas the exposed curing simulates the field conditions when no curing method is used to prevent the moisture loss. This is usually the case when the importance of curing is overlooked by some contractors.

For complete cement hydration, the water required is about one fourth the cement weight (Ingles and Metcalf, 1972). For instance, for 5% cement content, a water content of 1.25% of the soils' weight is required for complete cement hydration. In the case of the sand-marl mixes, the optimum moisture content used in the preparation of soil-cement mixtures is well above the required amount for cement hydration. The extra water is needed for lubrication during compaction. However, the exposed curing conditions do not prevent water evaporation from the samples and thus results in lower moisture contents during the curing period.

The loss of water from the soil-cement samples has both negative and positive effects. The negative effect is the unavailability of sufficient water for the process of cement hydration, while the positive effect is due to the carbonate cementation upon evaporation of water and the increase in suction which will increase the effective stress. It is worth to mention that sealed curing provides a strong and staple bonding that will insure proper long term performance of the mix. Therefore, exposed curing conditions can be helpful if enough moisture remains available for cement hydration. The moisture loss during exposed curing depends mainly on temperature and relative humidity. The relative humidity in the laboratory is usually around 50% under air-conditioned environment, while in the field, it varies from 30% up to 90%. The relative humidity in eastern Saudi Arabia can easily reach 90% in the summer. A relative humidity of 100% at any temperature has the same effect as that of the sealed conditions, where negligible evaporation of water occurs. On the other hand, lower relative humidity results in greater

loss of moisture. Therefore, the results obtained under the laboratory curing conditions should be interpreted after considering the conditions prevailing in the field (Ahmed, 1995).

b) Effect of Curing Temperature

The relative effects of various curing temperatures (21°, 35° and 50°C) on the UCS of the stabilized sand-marl mix were investigated. The results shown in Figure 4.25, for both sealed and exposed curing, clearly show that in the case of sealed curing, the higher the curing temperature, the higher the UCS. This is mainly because cement setting and hardening rates tend to increase with an increase in temperature provided that enough moisture is available for hydration. On the other hand, the strength attained at different temperatures is somewhat the same for exposed curing. This is attributed to the deficiency in water needed for cement hydration.

c) Effect of Curing Time

The variation in UCS strength of sand-marl mixtures with curing time was studied for samples treated with 5% cement and the results are shown in Fig. 4.26. Sealed as well as exposed samples were prepared and tested after curing periods of 1, 3, 7, 14, and 28 days at room temperature (21±2°C). The results indicate that the gain in strength was rapid in the first seven days and gradual thereafter. There was no significant difference in strength between exposed and sealed samples for the first seven days. However, after the 7 days, the exposed samples exhibited very slight gain in strength

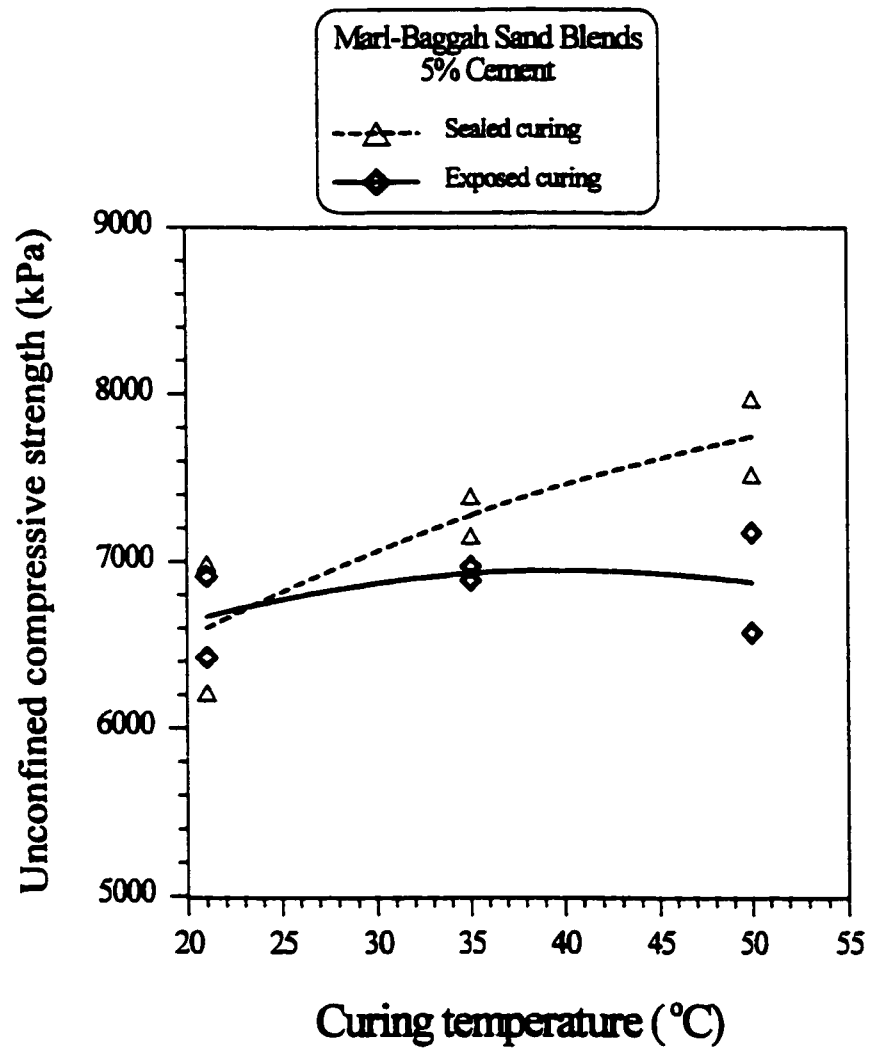


Fig. 4.25: Variation of the UCS with curing temperature for samples cured for 7 days

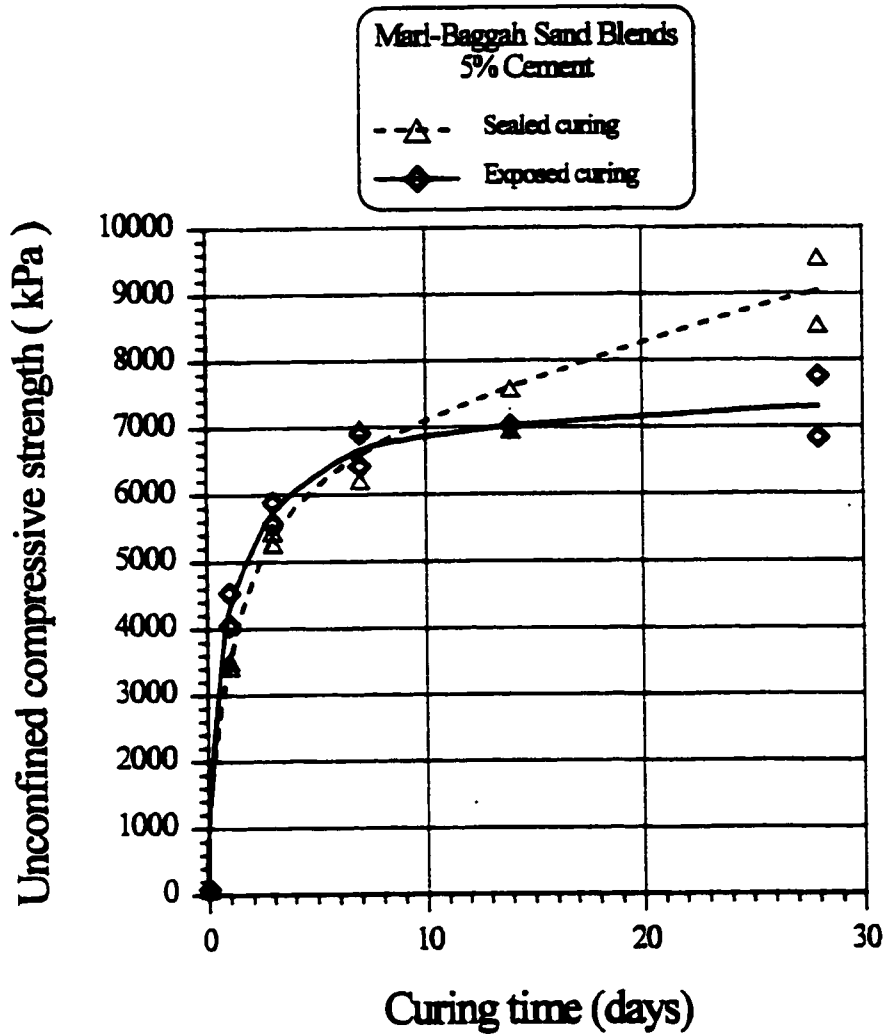


Fig. 4.26: Variation of the UCS with curing period for sand/marl mixes treated with 5% cement

while the sealed ones continued gaining strength with time. The slow gain in strength for the exposed samples was due to the loss of moisture during the first 7 days before a complete cement hydration took place. However, the gain in strength with time for the sealed samples was mainly due to the availability of sufficient moisture during the course of cement hydration.

4.4.2 Modulus of Resilience

The diametral resilient modulus (M_R) test was used to investigate the behavior of stabilized sand-marl mixtures under dynamic loading. M_R values of sealed as well as exposed samples at different temperatures is shown in Figure 4.27. The sealed samples exhibited consistently higher modulus values than the exposed ones. This is attributed to the availability of moisture during the curing period of the selected samples. Increasing the curing temperature resulted in a very slight decrease in the M_R values for both sealed and exposed samples. This slight decrease in M_R is possibly due to loss of moisture at higher temperatures, resulting in deficiency of water needed for further cement hydration.

The M_R tests were performed on the same specimens after 7, 14, 28, and 56 days of curing. This was done to investigate the effect curing period on the exposed specimens at laboratory temperature of 21 °C. The results reveal that the curing period

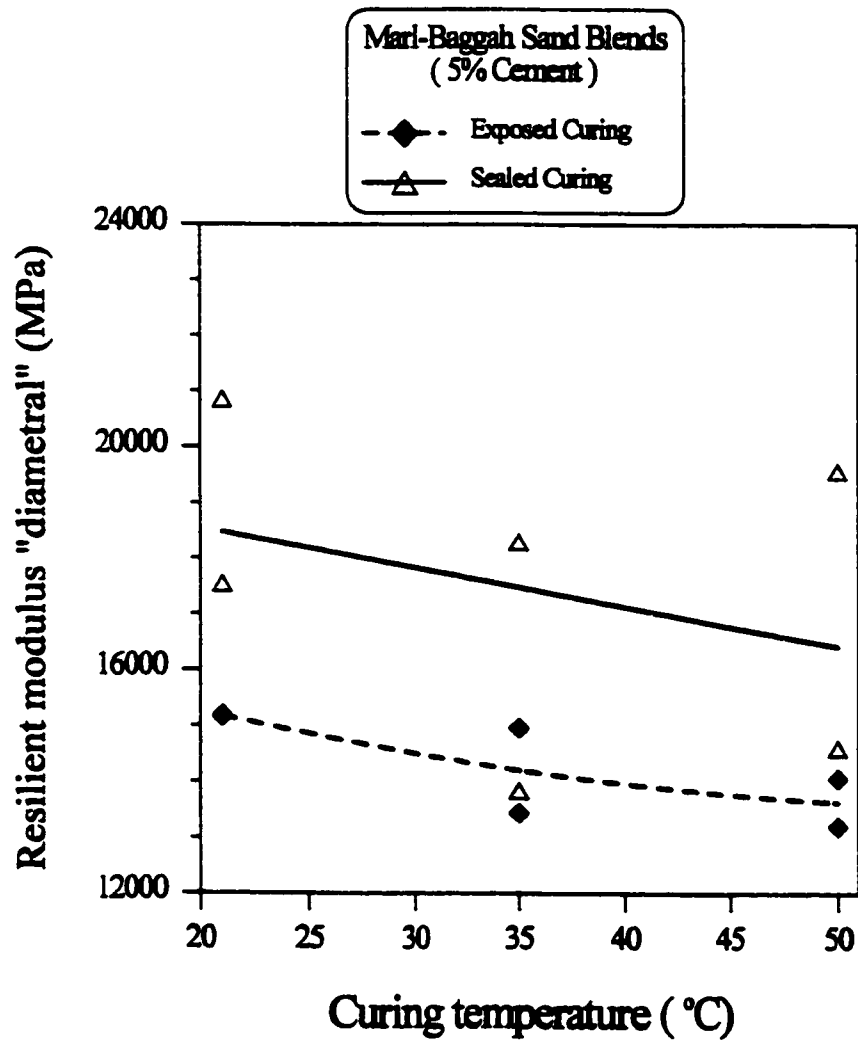


Fig. 4.27: Variation of modulus of resilience with curing temperature for samples cured for 7 days

does not affect the M_R values and were almost the same after 7 days of curing. Therefore, the 7 days M_R value can be considered an ultimate value for these curing conditions. This particular behavior is attributed to the low stress level; in which the portion of the stress-strain curve used in M_R testing develops after a few days of curing. Further increase in curing time does not affect this portion of stress-strain curve.

4.5 Bitumen Stabilized Sand-Marl Mixtures

Design of emulsified asphalt paving mixtures presents a significant challenge because such mixtures are considerably more complex than paving mixtures utilizing cutback or asphalt cement. Emulsified asphalt mixtures are three-component systems containing mineral aggregate, asphalt, and water. On the other hand, asphalt concrete and cutback mixtures contain aggregate and organic binder, and are considered as two component system.

Specifications and criteria for bituminous stabilized soils and aggregates are almost exclusively based on stability, durability and gradation requirements. The durability is not required by some agencies thus stability (i.e. Marshal test) becomes the only laboratory test specified for mixture design (Puzinauskas and Jester, 1983).

Various tests were performed on sand-marl mixtures as per the test designations from ASTM specifications. The tests are namely, sieve analysis, specific gravity, sand equivalent and centrifuge kerosene equivalent (CKE). These tests were conducted for the identification of the basic physical properties of the selected mix. The test results shown in Table 4.4 are within specification limits.

Since this investigation is primarily of geotechnical orientation rather than pavement construction, the tests used were stability, California bearing ratio (CBR), modulus of resilience (M_R), and creep.

4.5.1 Optimum Mix Design

The amount of asphalt required was estimated for trial sand-marl mixes using the centrifuge kerosene equivalent test (CKE) and oil ratio tests. These tests were used to determine the surface capacity of fine and coarse aggregates. The results indicated that the initial oil ratio is 4.5 and therefore, a range of percentages of liquid asphalt bitumen (emulsion and cutback) was used. In the case of emulsion, 2, 3, 4, 5, and 6% emulsion contents were used with three different added water ratios (1, 2, 3%). Prior to the determination of emulsion contents, premixing water of 2% was found sufficient. The percentages (or content) of the used cutback asphalt were 1, 2, 3, 4, 5, 6, and 7%.

Table 4.4: Physical properties for sand-marl mixtures

Test	Sand-Marl Mixture	Limiting Criteria (Terrel et al., 1984)
Passing No. 4 sieve	59 %	50-100%
Passing No. 40 sieve	38 %	35-100%
Passing No. 200 sieve	1.2 %	0-30%
Plasticity Index	NP	<6
Specific Gravity	2.7	-
Sand Equivalent Value	30	>25
CKE	1.10	-

4.5.2 Results and Discussion

Most of the literatures agree that the stability of water-exposed samples has more significance than that of dry-cured ones. There is normally a characteristic peak in soaked stabilities indicating that there are optimum water content and asphalt content at which maximum stability is achieved. One explanation for this is that while stabilities of samples cured at different asphalt contents are affected by the amount of water, the water contents of soaked samples are essentially the same regardless of asphalt contents. This allows selection of optimum asphalt content on the basis of soaked stabilities (Puzinauskas and Jester, 1983). Test results are shown in Table 4.5 and explained thoroughly in the following paragraphs.

a) Marshal Stability Test

Figures 4.28 to 4.30 illustrate the optimum mix design for the emulsion and cutback asphalts using the Marshal stability tests. For the added water, as well as the emulsified asphalt contents, it was observed that with the increase of added water for each increment of emulsion content, dry stability increases up to a peak, then it decreases thereafter. Similar trend was noted in the case of cutback asphalt. This peak in stabilities is a result of maximum dry densities achieved during compaction at the optimum residual asphalt content. The reduction in stability beyond the peak is believed to be a result of asphalt buildup in voids (between soil particles) which tends to push particles apart,

Table 4.5: Sand-Marl Bitumen Mix

Mix Design Criteria	Emulsion Mix	Cutback Mix	Limiting Criteria (Puzinauskas and Jester, 1983)	
			Min.	Max.
Opt. Pre-mixing water (%)	2	--	--	--
Opt. Added water (%)	2	--	--	--
Opt. Asphalt Content (%)	4	4	--	--
Dry Stability (kN)	27	11	2.224	--
Soaked Stability (kN)	6	4	2.224	--
Stability Loss (%)	77	64	--	50
M_R (ksi)	800	150	--	--
CBR (%)	Dry	170	--	--
	Soaked	120	--	--

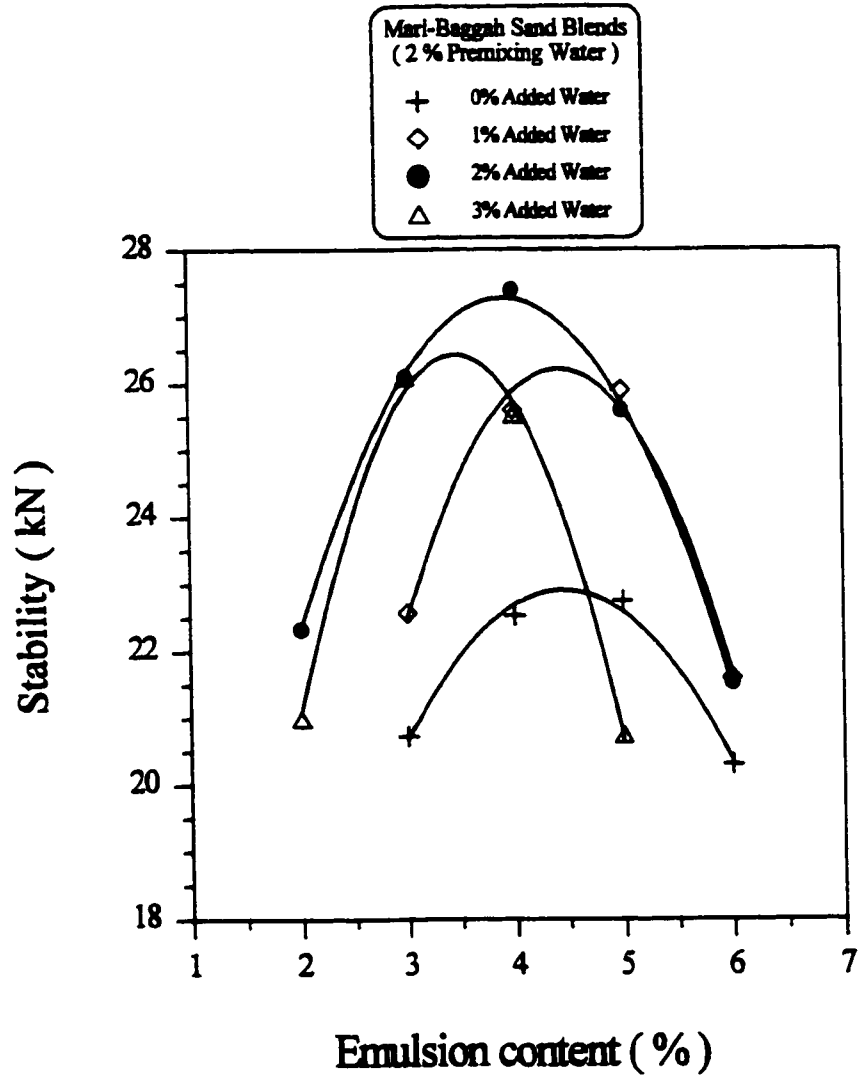


Fig. 4.28: Variation of the dry stability with emulsion and added water contents

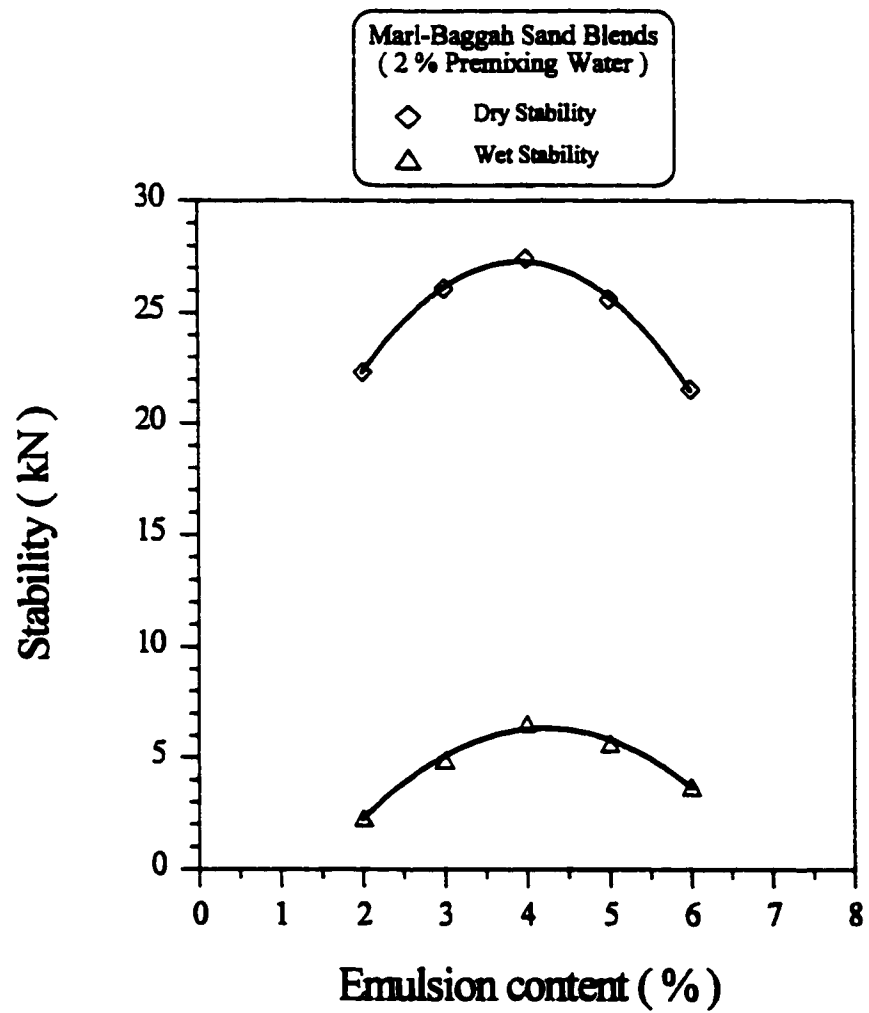


Fig. 4.29: Variation of the dry and wet stability with emulsion content

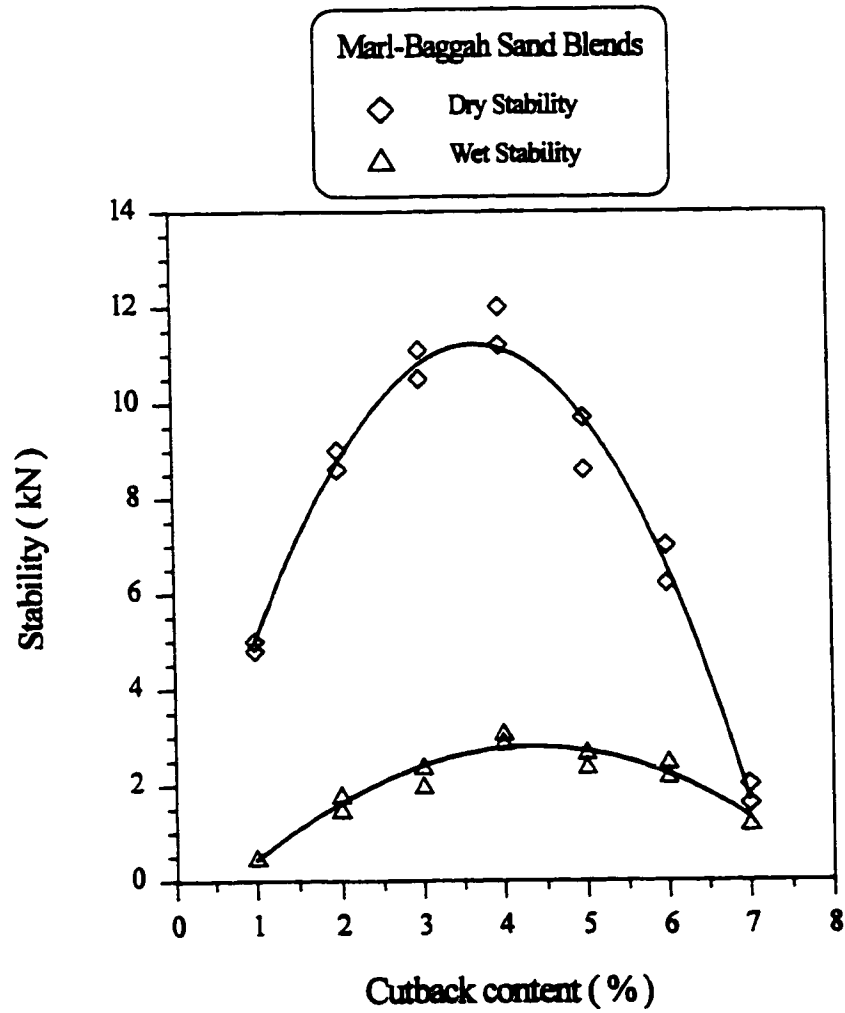


Fig. 4.30: Variation of the dry and soaked stability with cutback content

reducing granular contact and increasing lubrication. The percent loss of stability of the mixture when tested “soaked” in comparison to “dry” is shown to be excessive for both emulsion and cutback mixes. The loss amounts to 77% and 64% for emulsion and cutback mixes, respectively. This high loss is indicative of mixture having high moisture susceptibility and may cause disintegration during wet seasons. Since the stability loss criteria (Table 4.5) have not been met, the mix was considered inadequate.

b) Modulus of Resilience

Figure 4.31 shows that M_R tends to decrease with the increase in testing temperature due to the reduction of asphalt stiffness with temperature. The gain in strength (and modulus) is expected to continue up to 3 to 6 months due to water evaporation and asphalt hardening.

In the case of cutback asphalt, the specimens could not withstand the static load of 10 lb, which was intended to hold the sample in place. The low strength is attributed to the lubrication of soil particles. In addition, the cutback asphalt does not provide strong bonds between soil particles. Therefore, the resilient modulus test was omitted for all prepared samples.

The strong bonding in the case of emulsified asphalt is due to the efficiency of aggregate coating and the quick regain of viscosity upon water evaporation. While the low strength in cutback asphalt is attributed to high lubrication of soil particles due to gradual regain of viscosity with cutback evaporation.

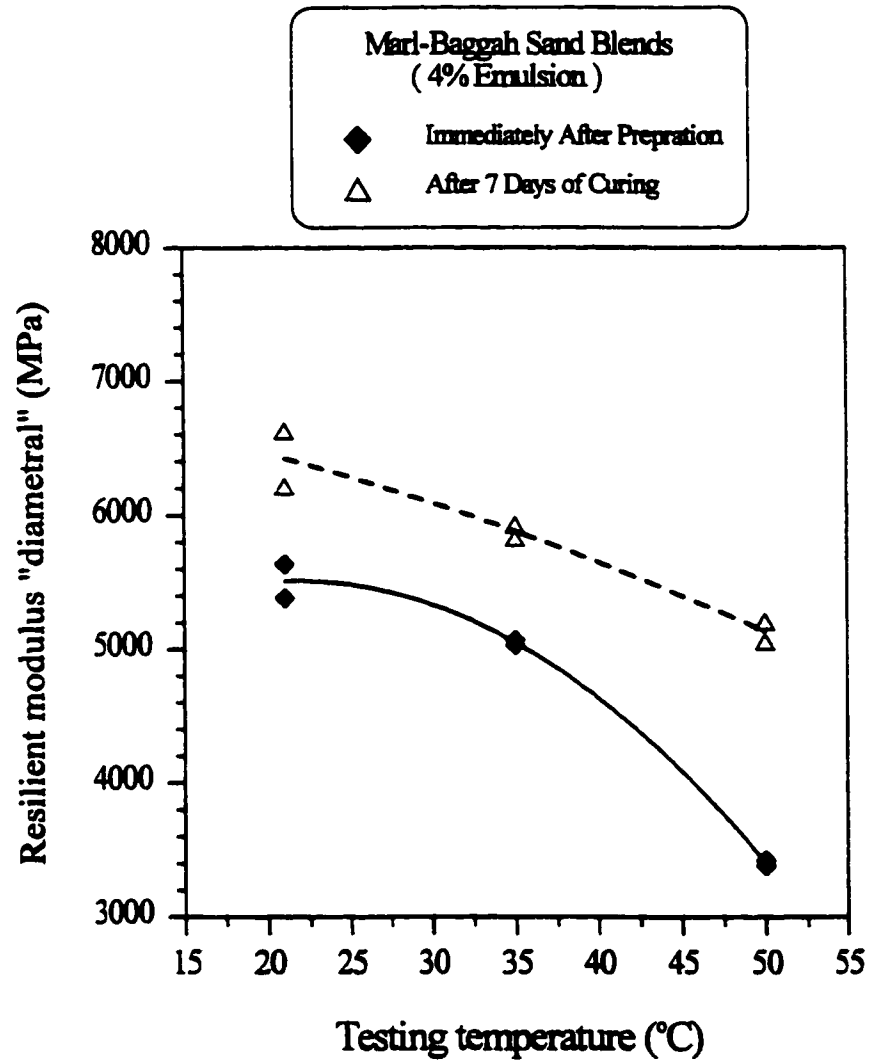


Fig. 4.31: Variation of the resilient modulus with testing temperature for samples tested after different curing periods

c) California Bearing Ratio (CBR)

Although the CBR test is not standard for stabilized materials, the test was conducted only for emulsion to assess the bearing capacity of the mix. For 4% emulsion, the CBR values of 170 and 120 were obtained for dry and soaked samples, respectively. The corresponding CBR values for the same mix with no additives are 195 and 165 for dry and soaked testing, respectively (see Figure 4.33). The results are presented in Figures 4.32 to 4.34. It is clearly seen that the addition of emulsified asphalt to sand-marl mix reduces both the dry density and the CBR values. The CBR decreased from 195 for sand-marl mix with 0% emulsion to 170 at 4% emulsion content at an optimum moisture content of 6% (Figure 4.34). Reduction in strength when adding emulsion is expected since CBR depends mostly on friction. Emulsion tends to lubricate the soil particles and this resulted in reduction of frictional resistance. Despite the fact that emulsion increases the flexibility of the mix, the results reveal that addition of emulsion is not improving the load carrying capacity of the mix. The results also indicate that soaking reduces the CBR regardless of the addition of 4% emulsion.

d) Creep

The creep of the sand-marl mix was evaluated under three testing temperatures (21, 35, and 50°C), as shown in Table 4.6. Creep curves for emulsion and cutback asphalts are shown in Figure 4.35 and Figure 4.36. These results reveal that the total deformation, elastic strain, viscoelastic properties as well as the permanent strain increase with the increase in temperature for emulsion as well as cutback.

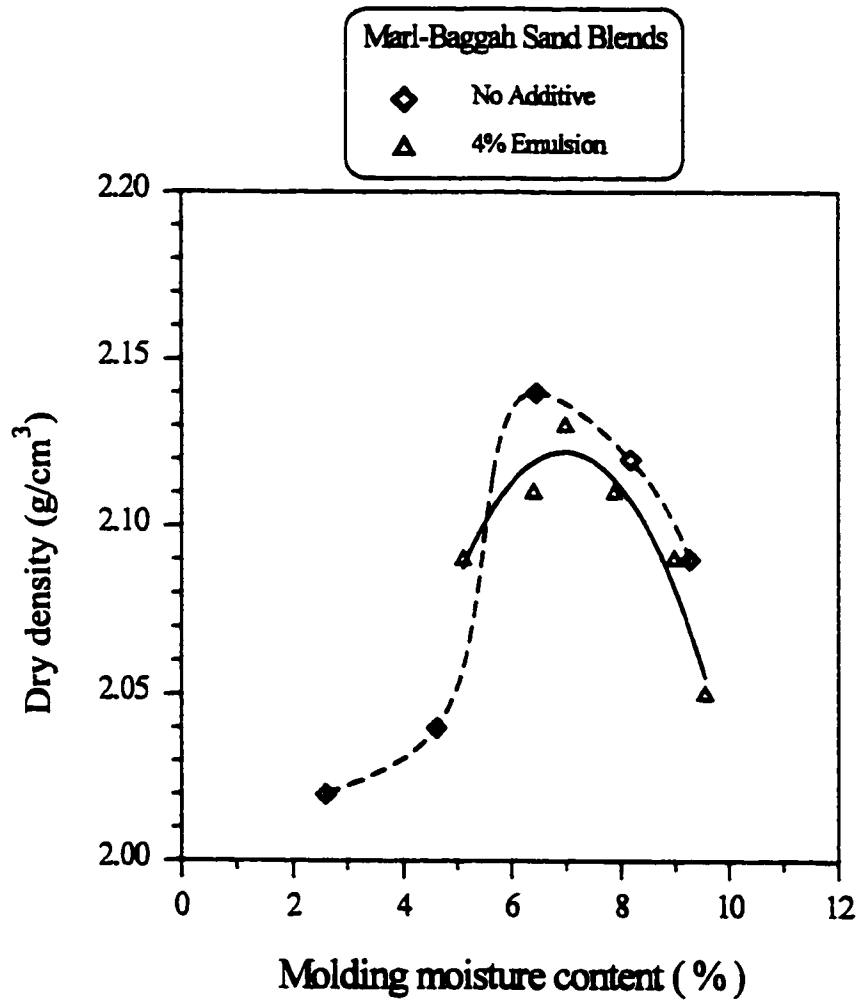


Fig. 4.32: Density of sand-marl mix with and without additives

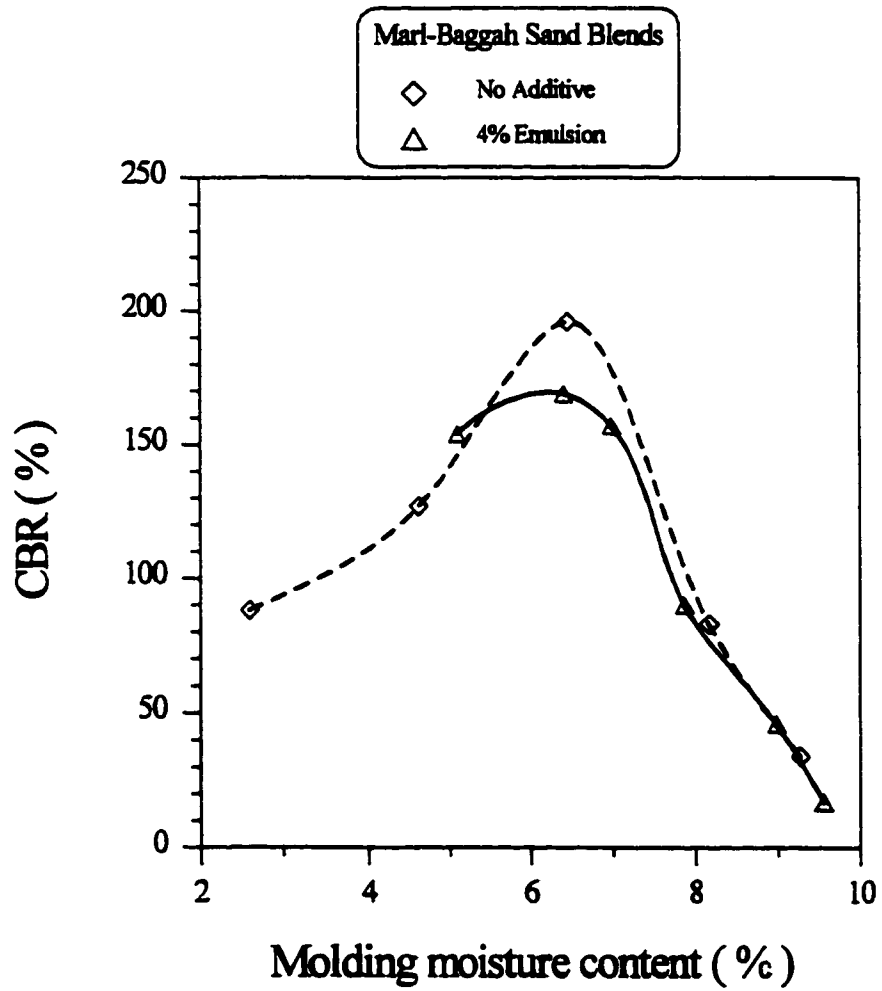


Fig. 4.33: Variation of the unsoaked CBR with molding moisture content

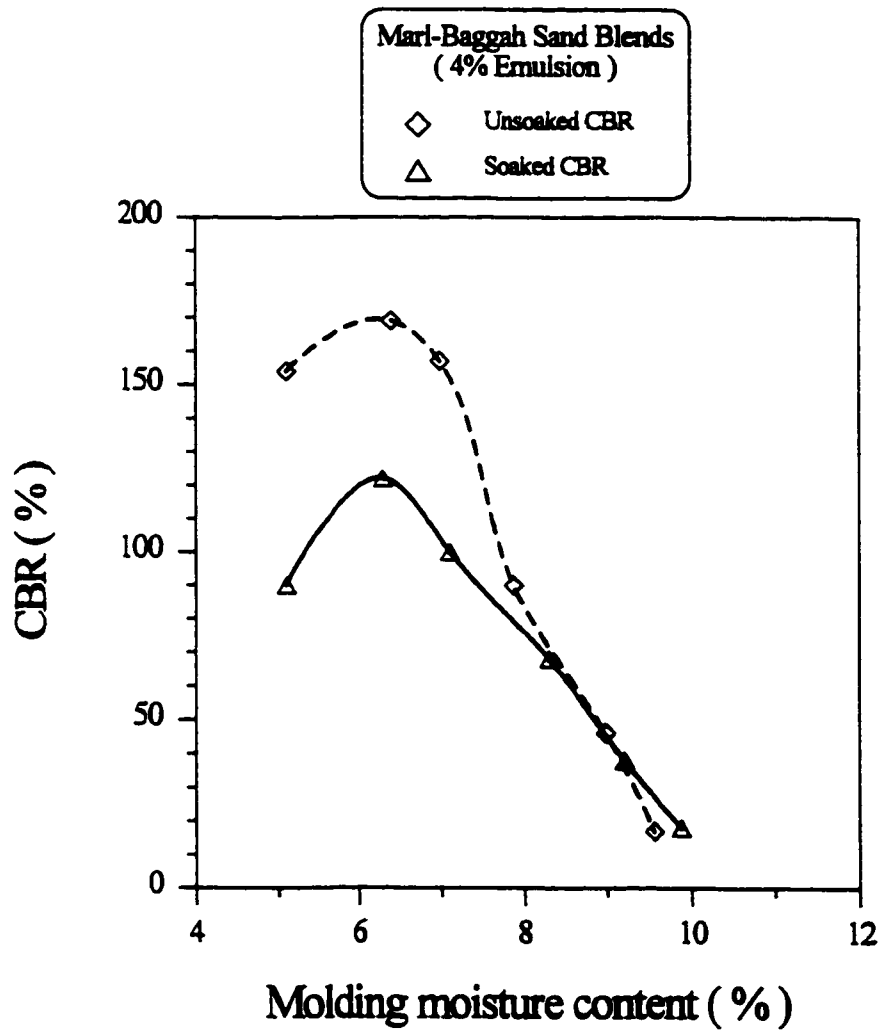


Fig. 4.34: Variation of the CBR with molding moisture content

Table 4.6: Results of Creep curves for bitumen-sand-marl mixes

Testing Temp. (°C)	Bitumen Type	Total Deformation ΔT (mm)	Elastic Strain $\Delta \epsilon$ (mm)	Viscoelastic ΔV (mm)	Permanent Strain ΔP (mm)
21	Emulsion	0.95	0.50	0.12	0.33
	Cutback	1.10	0.53	0.11	0.52
35	Emulsion	1.03	0.54	0.15	0.34
	Cutback	1.22	0.52	0.16	0.55
50	Emulsion	1.25	0.61	0.13	0.49
	Cutback	1.40	0.54	0.13	0.75

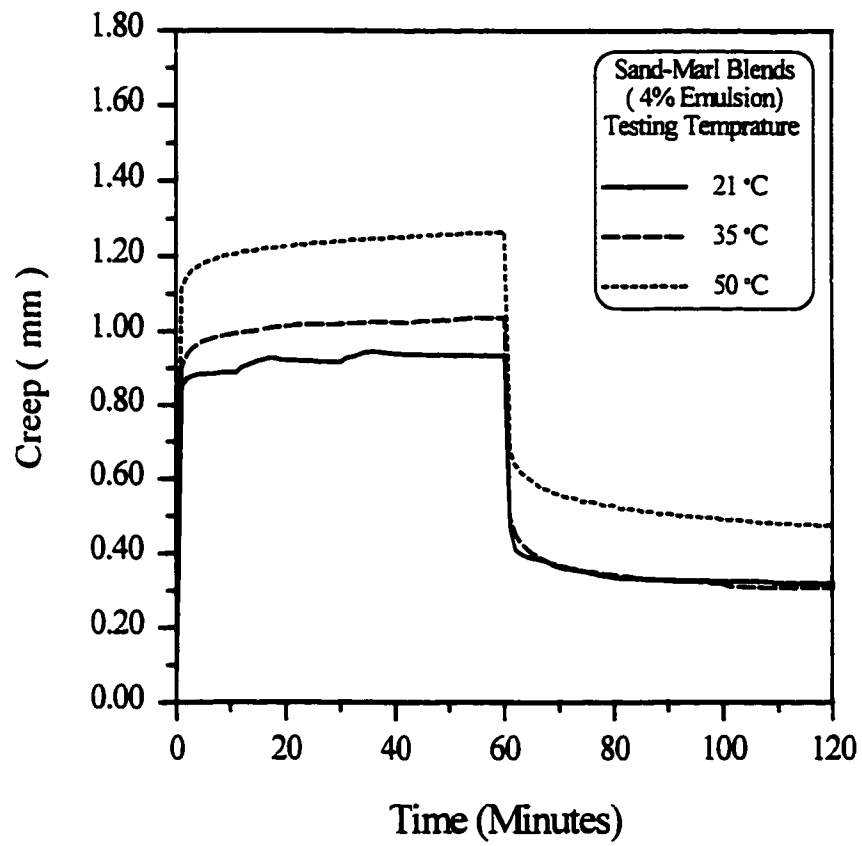


Fig. 4.35: Creep curves for sand/marl mixes treated with 4% emulsion

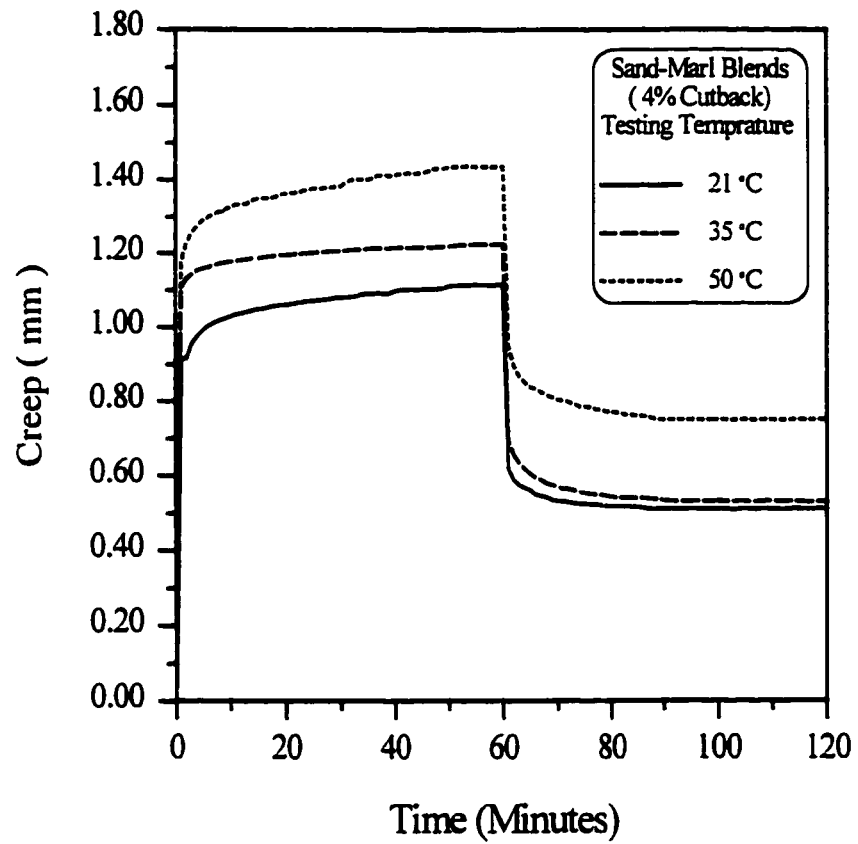


Fig. 4.36: Creep curves for sand/marl mixes treated with 4% cutback

The results also indicated that emulsion mixes: 1) have higher resistance to rutting than cutback, and 2) are more elastic than cutback mixes especially at higher temperatures. Therefore, the 4-in diameter and 2.5-in high specimen which is specified by Shell group, was sufficient to display the correct behavior of the mixture under static loading, for the particular aggregate gradation used here.

CHAPTER 5

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary

The use of abundant eastern Saudi sands in construction helps control sand movements which are known to result in many hazards, especially in windy seasons. Therefore, there is a need to conduct an investigation on sands to help delineate their relevant engineering properties and establish a data base for their practical uses and eventual classification, as part of regional code. Furthermore, such information is considered beneficial for soil stabilization and ground improvement.

In addition, marls are considered the best construction material in eastern Saudi Arabia. However, the extensive use of marl has depleted local supplies. Such a situation has necessitated the search for alternative material sources. Therefore, blending sand with marl will result in an alternative construction material, that can be treated or stabilized to improve its engineering properties at reasonable costs.

Results have revealed that most of eastern Saudi sands are classified as poorly graded and all sands show somewhat similar behavior. It also showed that optimum blends consist of 35% sand and 65% marl. The engineering properties of such blends are superior when compared to marl alone. Furthermore, stabilizing sand-marl mixes using

Portland cement has brought up a material with the most acceptable engineering characteristics. The mix meets the strength and durability requirements and it is economically advantageous. On the other hand, stabilizing the mixes with bitumen resulted in a poor quality material and thus the bitumen stabilization was rejected.

5.2 Conclusions

Based on the results of the experimental program conducted in this investigation on sands and sand-marl mixtures, the following conclusions can be drawn:

1. Most of eastern Saudi sands are classified as poorly graded according to the unified soil classification system (USCS) and all sands show somewhat similar behavior.
2. Sand-Marl mix at a ratio of 35% and 65% gives the highest CBR values when compacted at the optimum moisture content.
3. Increasing the moisture content above the optimum value for sand-marl mixes resulted in complete loss of the soil strength. It was clear that the water sensitivity was attributed to the marl.
4. Portland cement was proven to be a suitable chemical additive in stabilizing the sand-marl mixtures having a ratio of 35% sand and 65% marl. A cement content of 5% resulted in a material meeting the strength and durability requirements, and in excess of these requirements.

5. The Modulus of resilience (M_R) values for samples treated with 5% cement and allowed to cure under sealed conditions was higher than those under exposed conditions. Increasing the curing temperature caused a reduction in M_R values for both sealed and exposed cured samples.
6. The unconfined compressive strength of cement-sand-marl mixtures increased with time in both sealed and exposed curings. Both curing conditions tended to respond similarly for the first 7 days. Thereafter, the exposed curing showed very slight gain in strength while the sealed curing continued gaining strength.
7. The unconfined compressive strength of cement-sand-marl mixtures increased for sealed curing with an increase in the curing temperature whereas the strength attained at different temperatures was somewhat the same for exposed curing.
8. The durability test results using ASTM D 559 and slake durability tests are in a very close agreement and the values are far below the maximum allowable weight loss. Slake durability test is more consistent and reliable than ASTM D 559 (i.e. this was the case with increasing sand content).
9. The emulsified and cutback asphalts was proven to be effective in increasing the stability. However, both were ineffective in reducing the stability loss of sand-marl mix. Therefore, both were rejected as stabilizing agents.

10. The Modulus of resilience (M_R) values for sand-marl mix treated with emulsified asphalt tended to increase after 7 days of curing and became almost constant afterward. The M_R values decreased with the increase in testing temperature.

5.3 Recommendations for Further Study

1. There is a need to perform a mechanical and chemical stabilization on the sand-marl mixes using different binders, such as polymer.
2. A study needs to be performed on sand blended with different gradations of the same marl as well as different marls.
3. A study on the tensile cracks that develop during the durability testing of the cement sand-marl mixes need to be performed on macro and micro levels to investigate the possible causes of the cracks, their various effects, and the possible preventative measures.

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