Effect of gradation and testing procedures on the load carrying capacity of calcareous sediments

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Civil Engineering

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Abstract

Calcareous sediments are commonly used, in the eastern region of Saudi Arabia as base and subase material for roads and runways. The construction materials are produced from crushers in which materials with different qualities and particle sizes are mixed together and the final product is intended to have a certain gradation. The produced materials have acute water sensitivity. In addition to the upnormal behavior of the calcareous base course materials under the prevailing environmental and loading conditions. The adequacy of use of conventional testing procedures for strength determination of these materials is questionable. This is caused by the poor correlation between the laboratory and the field results as result of discarding the oversize particles for the laboratory samples.

In this search program, the effect of gradation and testing procedures on the load carrying capacity of calcareous sediments (marls) was studied. This was achieved by performing a testing program using the CBR. Unconfined Compressive Strength and Clegg Hammer tests. Three different gradations for two different marls were used in the study. In addition, a large size compaction and CBR testing setup was used to study the effect of oversize particles on the CBR and CIV values. Furthermore, the applicability of the common oversize correction methods was investigated.

The results clearly showed that soil gradation has a remarkable effect on the UCS values while its effect on the CBR and CIV results was not that significant. In addition, the maximum particle size, which was included in the specimens was found to have great significance on the CBR values. Furthermore, the maximum dry density and the optimum moisture content values were found to be independent of soil gradation. The mold confinement was found to have a significant effect on the CBR values. About 100% increase occurred on the CBR values as a result of mold confinement. In addition, CBR values greater than 200% were found to have questionable practicality.

After implementing the oversize correction methods for the selected marls, it was observed that all equations approximated the dry density of the entire material and produced density values close to those obtained using the large mold, with less than 2% tolerances. However, the ASSHTO-1 and ASTM correction equations gave underestimated and overestimated values, respectively. While AASHTO-2 equation and scalp and replace methods gave accurate results.

EFFECT OF GRADATION AND TESTING PROCEDURES ON THE LOAD CARRYING CAPACITY OF CALCAREOUS SEDIMENTS

BY

OSMAN EL HUSSIEN MOHAMMED

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ABSTRACT

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Title: **The Effect of Gradation and Testing Procedures on the Load Carrying Capacity of Calcareous Sediments**
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Calcareous sediments are commonly used, in the eastern region of Saudi Arabia, as base and subbase material for roads and runways. The construction materials are produced from crushers in which materials with different qualities and particle sizes are mixed together and the final product is intended to have a certain gradation. The produced materials have acute water sensitivity. In addition to the upnormal behavior of the calcareous base course materials under the prevailing environmental and loading conditions, the adequacy of use of conventional testing procedures for strength determination of these materials is questionable. This is caused by the poor correlation between the laboratory and the field results, as a result of discarding the oversize particles for the laboratory samples.

In this research program, the effect of gradation and testing procedures on the load carrying capacity of calcareous sediments (marls) was studied. This was achieved by performing a testing program using the CBR, Unconfined Compressive Strength and Clegg Hammer tests. Three different gradations for two different marls were used in the study. In addition, a large size compaction and CBR testing setup was used to study the effect of oversize particles on the CBR and CIV values. Furthermore, the applicability of the common oversize correction methods was investigated.

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الخلاصة

الإسم : عثمان الحسين محمد

العنوان : تأثير التدرج الحبيبي وطرق الاختبار على قوة تحمل التربة الجيرية

القسم : الهندسة المدنية

التاريخ : فبراير ٢٠٠١م

تستخدم التربة الجيرية عادة في إنشاء طبقات أساس الطُرق و مدارج المطارات في المنطقة الشرقية للمملكة العربية السعودية. وهذه المواد تنتج بواسطة كسارات حيث تُخلط مواد ذات خواص وأحجام مختلفة للحصول على التدرج الحبيبي المطلوب. وعادة ما تكون هذه المواد ذات حساسية عالية للماء. بالإضافة للخواص غير الطبيعية للمواد الجيرية خصوصا تحت الظروف البيئية و زيادة أحمال الطرق فإن ملائمة إستعمال طُرق الإختبار المتعارف عليها لإيجاد قوة تحمل هذه المواد أصبح مثار تساؤل، وهذا يتج من ضعف العلاقة بين النتائج المستقاة من المعمل والنتائج الحقلية بسبب إستبعاد الأحجار ذات الأحجام الكبيرة من عينة المعمل.

في هذا البحث تمت دراسة تأثير التدرج الحبيبي وطُرق الإختبار على قوة تحمل التربة الجيرية (المارل). و قد تم ذلك بإجراء برنامج إختبارات تقليدية باستعمال إختبار نسبة تحمل كاليفورنيا، قوة الإنضغاط غير المحصور و مطرقة كلج. و استعملت في هذه الدراسة تدرجات حبيبه مختلفة لنوعين من التربة الجيرية. بالإضافة إلى ذلك، تم تحضير عينات إختبار نسبة تحمل كاليفورنيا في قالب اكبر حجماً من القالب القياسي بغرض دراسة تأثير اشتمال العينة على أحجار كبيرة الحجم على قيمة نسبة تحمل كاليفورنيا وقراءة مطرقة كلج. كما تم أيضاً دراسة درجة دقة الطُرق المستعملة لتصحيح تأثير إستبعاد الأحجار كبيرة الحجم من العينات.

وقد أثبتت النتائج أن للتدرج الحبيبي تأثير ملحوظ على قوة التحمل بواسطة إختبار الضغط غير المحصور، بينما ليس له تأثير يذكر على نسبة تحمل كاليفورنيا وقراءة مطرقة كلج. بالإضافة إلى ذلك فإن الحد الأقصى لحجم الحجارة المشمولة في العينة كان له تأثير مهم على نسبة تحمل كاليفورنيا. كما أن الحصر الناتج من جدار قالب الإختبار له تأثير قوي على قيمة نسبة تحمل كاليفورنيا، حيث يُحدث زيادة تقدر بحوالي 100%. علاوة على ذلك، فإن كفاءة الإختبار يكون مشكوك بها عندما تزيد قيم نسبة تحمل كاليفورنيا عن200% .

كما دلت النتائج على أن المعادلات التي استعملت لتصحيح تأثير إستبعاد الأحجار الكبيرة أعطت قيماً للكثافة الجافة قريبة من القيم التي أُوجدت باستعمال التدرج الكامل و التي تم تحضيرها في قالب الدمك كبير الحجم إذ لا يتعدى الفرق 2%. و من الملاحظ أن معادلة AASHTO-1 أعطت قيماً للكثافة أقل تقديراً ومعادلة ASTM أعطت قيماً اكثر تقديراً من القيم الفعلية، بينما أعطت معادلة AASHTO-2 وطريقة الإزالة والتبديل قيماً أكثر دقة للكثافة.

درجة ماجستير

قسم الهندسة المدنية

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Chapter 1

Introduction

1.1 General

Calcareous soils are defined as soils that contain calcium carbonate, which occur in different forms, such as calcite and aragonite [$CaCO_3$], and dolomite [$CaMg(CO_3)_2$], while gypsum [$CaSO_4.2H_2O$] is often present at varying percentages (Aiban et al, 1999). Due to the massive number of engineering projects, which were witnessed by the Gulf countries in the last few decades, large amounts of calcareous sediments (locally known as marl) were utilized in those projects as back fill behind retaining walls, foundation materials and graded base-course for service roads and highways. The utilization of marl soils was necessary due to the scarcity of good engineering materials. The excessive use of marl soils without enough knowledge of their upnormal behavior, under severe environmental and loading conditions, resulted in unsatisfactory performance of some structures and a complete failure in some cases. Failures were observed in some service roads in Dammam area where marl soils were used as a graded base course (Aiban, 1995).

Marl soils are usually used as a base material after being processed and brought to a certain gradation. This may require the addition of crushed stones with different particle sizes. On the other hand, marl soils can be used in their natural gradation as subbase material for roads after the exclusion of large boulders. The poor performance of this modified material is caused by its acute water sensitivity i.e. the fluctuation of moisture content causes great changes in the strength. In addition, a growing confusion was observed among consultants about the suitability of the conventional testing procedures for these materials. This confusion is caused by the poor correlation between the laboratory and the field results, even when the testing conditions and the quality control procedures are the same. Hence, the possibility of developing new testing techniques or, at least, modifying the existing ones, needs to be investigated.

1.2 Calcareous Sediments

Different kinds and formations of calcareous soils exist in many parts of the earth. The two common types, which are used in construction projects, are calcrete (sometimes called conglomerate) and marl. The word "marl" in English is used for a mixture of clays, carbonates of calcium and magnesium, and remnants of shells (American Heritage Dictionary, 1991). Locally, it is used to identify the carbonate sediments, which cover the costal parts of Saudi Arabia. In general, carbonate soils form as a result of physical and chemical weathering of parent carbonate rocks like limestone, dolomites, carbonate sandstone, etc. (Akili, 1980). The simultaneous deposition of carbonate soils and clay had lead to the formation of marls. These types of deposits are usually found in the form of consolidated or cemented layers. Locally, marl layers are present mostly in the eastern region of Saudi Arabia in Rus Formation, Dammam Formation, Hadrukh Formation, Dam Formation and Hafouf Formation (Aiban, 1995; Roger, 1985).

Generally, calcareous sediments show high water sensitivity, which is observed especially in fine-grained sediments. This phenomenon is substantiated by the sharp changes in the load carrying capacity and strength with slight changes of the water content (Aiban, 1999). Qahwash (1989) reported a reduction of about two thirds of the bearing capacity of marl samples prepared at optimum water content when tested after being fully submerged instead of being tested without soaking.

The range of particle sizes in calcareous soils is tremendous. These soils can range from boulders down to ultra fine-grained colloidal materials. Such variation in gradation has its effects, not only on the engineering behavior but also on the laboratory testing procedures, which have some limitations regarding the maximum particle sizes that can be included within the tested specimen.

1.3 Testing of Gravelly Soils

The accuracy of any testing procedure depends very much on the quality of the specimen tested and on whether the test is reliable to assess the required engineering properties of the specimen. Conventional laboratory test procedures for determining soil strength and load carrying capacity usually require the removal of particles larger than a predetermined maximum size. This leads to inaccurate results and may need corrections. For example, a common criterion for triaxial testing is that the maximum soil particle size should be limited to no more than one sixth of the sample diameter (ASTM, 2000). According to Donaghe and Torrey (1994), Holtz and Lowitz recommended in 1957 a ratio between specimen and particle diameters not less than 5 to 6 for moisture- density relationships for soil containing oversize particles. Because of the fixed ratio between the largest particle size and specimen diameter, testing of soils containing oversize particles is believed to be both problematic and questionable.

In the California Bearing Ratio test, the specimen must be compacted in a 6 in. diameter mold where the soil is usually finer (passing through) than the ¾-in. (19-mm) sieve. This implies that the material retained on this sieve "oversize material" should be excluded. The ASTM standard states that if the test specimen contains more than 5% by weight oversize fraction (coarse fraction) and the material will be excluded before the test, corrections must be made to the unit weight and water content of the test specimen (ASTM, 2000).

Calcareous sediments, which are commonly used as construction materials in eastern Saudi Arabia, usually contain relatively high percentages of oversize material. Hence, there is no test procedure that might be suitable for these materials without excluding some particle size. Therefore, the reliability requirements of the results obtained from these tests in the absence of proper correction methods will be violated. Generally, the quality control of construction projects, in which calcareous materials are used, follows the classical techniques adopted by the standards for general geomaterials. Locally, in eastern Saudi Arabia, the consulting agencies follow the American Society for Testing and Materials (ASTM), and the American Association for State Highway and Transportation Officials (AASHTO) standards. Despite the fact that these standards are internationally adopted, a growing confusion regarding their suitability for local use is observed among consultants. This resulted after realizing high discrepancies between field performance and laboratory test results.

1.4 Objective of the Investigation

California Bearing Ratio test is the most common test, which is used locally for measuring the characteristics of base and subbase materials. However inaccurate results may appear when the standard methods of testing are used (for example ASTM D1883) without realizing the water sensitivity of the material. This is mainly because past practice has shown that CBR results for those materials having substantial percentage of particles retained on the No. 4 sieve are more variable than for finer materials (ASTM, 2000). For materials having maximum particle size larger than ¾ in. (19mm) the effect of mold size will cause significant discrepancies in the CBR values. A ratio of specimen diameter (mold diameter) to the largest particle size should not be less than 5 to 6 as suggested by Donaghe and Torrey (1994). In addition, there is an uncertainty associated with the applicability of the CBR test to coarse-grained soils. Furthermore, the meaning of CBR values greater than 100 percent is not yet clear (Terrel et al. 1984).

Currently, there is no available information regarding the effect of the presence of oversize particles in marl soil when tested using traditional CBR testing techniques. Furthermore, CBR values higher than 100 percent, which are always measured when testing marl soil, need a careful investigation. In addition, there is a need to have a close simulation for field conditions. This can be achieved by the use of larger molds, which will minimize the constraints imposed by the standard (small size) molds.

The main objectives of this experimental research program are:

- To study the effects of oversize particles on the load carrying capacity of marl. The California Bearing Ratio and the Clegg Hammer tests will be used to evaluate these effects.
- To study the effect of mold size on the CBR values of calcareous materials
- To study the effect of gradation on the load carrying capacity of marl.
- To study the applicability of the common correction methods which are commonly used for materials having oversize particles.
- To investigate the meaning of high CBR values by studying the correlation between CBR and other reliable tests such as the unconfined compressive strength.

In summary, the investigation will take into account the effects of the following parameters:

1- Mold size (two sizes).

2- Soil gradation (three different gradations).

3- Moisture content of the soil during compaction.

4- Marl type (two different marls).

5- Oversize correction methods (five correction methods).

The outcome of this study will constitute an advanced step towards better understanding of the behavior of local construction materials. This will contribute to the database of local soils and also will be helpful for road construction industry and for foundations construction. This study will help also in assessing the reliability of the currently used field quality control procedures.

1.5 Thesis Organization

To accomplish the abovementioned objectives, a comprehensive literature review was conducted. The detailed literature review is given in the second chapter, which includes the basic features of calcareous materials, the testing techniques for gravelly soils and the common correction methods adopted for soils with oversize particles. The third chapter is devoted for the detailed description of the experimental program. The fourth chapter is concerned with the presentation of the results obtained from the experimental program and discusses the output of the entire work. Summary, conclusions and recommendations for further studies are presented in the fifth chapter.

Chapter 2

Literature Review

2.1 Calcareous Soils

Calcareous soils are characterized by the presence of carbonates in the soil matrix. These carbonates consist usually of calcium carbonates deposited around or within the soil particles and in the voids of the soil matrix. These soils cover great parts of seabeds where oil and gas platforms are built in many parts of the world such as Australia, India, North Atlantic, the Gulf of Mexico, and the Arabian Gulf (Ismail and Ahmed, 1990). Engineers dealing with calcareous soils are facing many problems associated with the variability of constituents, extreme water sensitivity, leachibility, crushing of particles upon compaction and inapplicability of standard tests and specifications (Aiban et al. 1995). Therefore special attention is required when using such materials in engineering projects.

2.1.1 Geological Formation of Calcareous Soils

Calcareous or calcium carbonate soils have their own properties, which are quite different from those of ordinary soil types. In order to understand their peculiar properties, the geologic formation of calcareous soils must be given some attention. Continental carbonate soils are mostly derived from calcrete development in subhumid to semi-arid climates. Calcrete is the product of calcium carbonate precipitation in soil profile, weathered rocks, as well as alluvium and other clastic sediments of the vadose zone. The accumulation of calcium carbonate will enhance the development of calcrete deposits, which prevails when the precipitation rate exceeded the evapotranspiration rate. Such phenomenon is common in subhumid and semi-arid climates (Horta, 1988). Most carbonate sedimentation results from chemical or biochemical processes, which occur usually in special marine environments that are usually found in clear, warm and shallow water. Fig. 2.1 shows a world map with areas of modern deposition of carbonates, including the equatorial belt, and areas of warm ocean currents (Wilson, 1975). Carbonates are a polygenic group of rocks. When carbonate soils have been buried, significant changes in their original characteristics may occur, for example most dolomites, which are magnesium rich carbonates represent a post depositional alteration of the limestone (Fookes and Higginbottom, 1975).

The parent rocks from which the Arabian Gulf calcareous soils had been generated are weak sedimentary calcareous rocks, which are believed to be of Holocene-Pleistocene age and thus, in geological terms are considered to be very young (Cook, 1999). Geologic studies of the Eastern Saudi Arabia region indicate that the surface rocks are Tertiary in age covered by Quaternary deposits (Roger, 1985; Johnson, 1987). Sediments ranging from Paleocene-early Eocene to Miocene-Pliocene cover most of the area. In addition, unconsolidated materials including gravel of Tertiary age and various sediments of Quaternary age, such as beach gravel and sand, gravel and silt in basin deposits, sabkha sediments and various aeolian sand deposits do exist in the area (Roger, 1985).

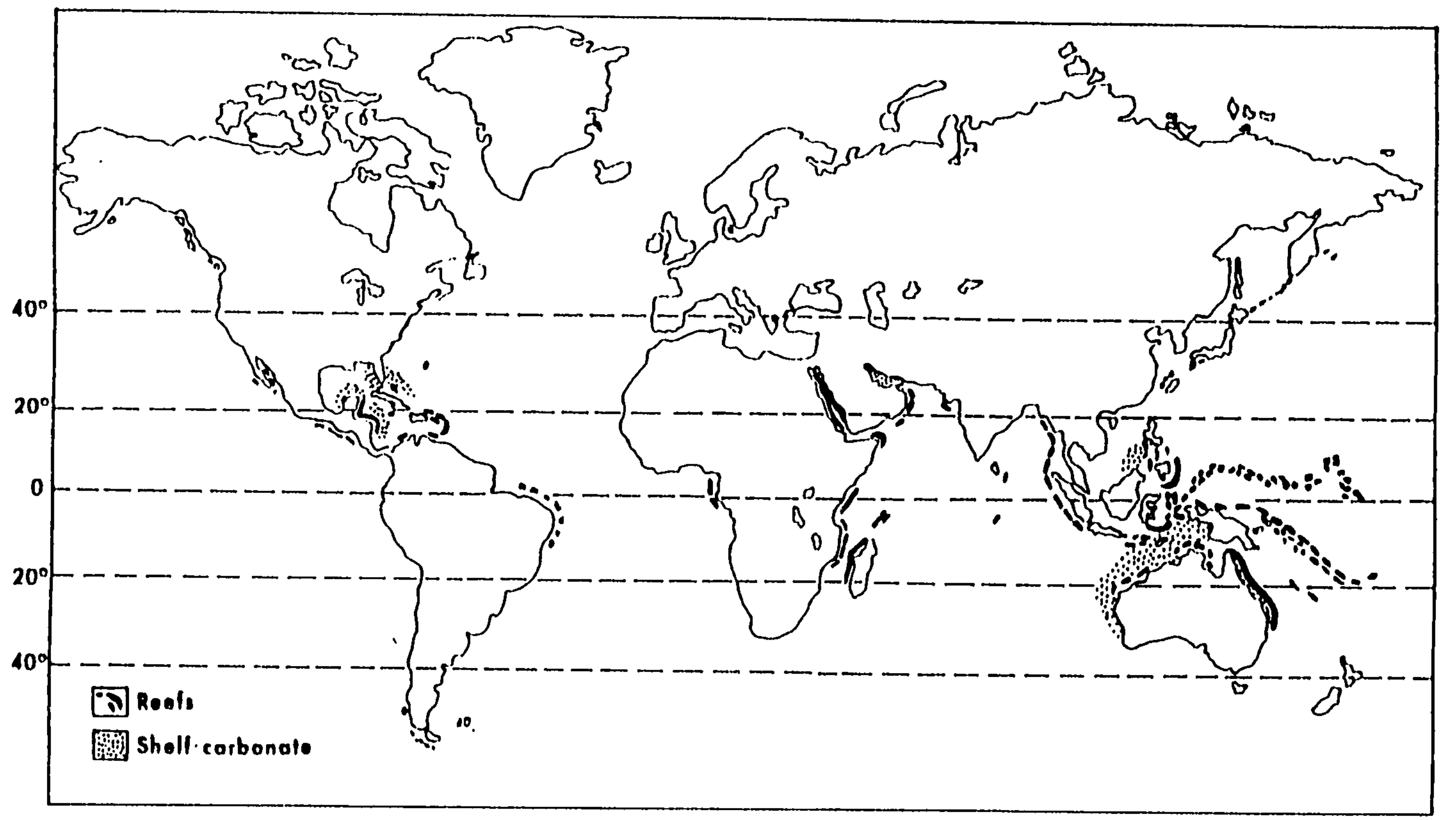


Figure 2.1: Distribution of modern marine carbonate sediments in shallow water (Wilson1975).

2.1.2 Marl Soil

Locally the term marl is used to indicate carbonate sediments, which cover most of eastern Saudi Arabia such as Abqaiq, Dhahran, Dammam, Abu Ali, Hafuf, Berri, Fadhli, Jubail, Abu Hadriyah and Safaniyah areas. The available geomaterials for construction purposes in eastern Saudi Arabia are marl soils, sands, sabkhas and expansive clays. Each of these soils has its problems. Sands are mostly fine to medium in size and they have low strength, especially with the lack of confinement (Aiban et al. 1999). Sabkhas and clays are problematic and have high water sensitivity, which does not qualify them as construction materials. Despite their upnormal behavior, marl soils are considered the best alternative for engineering use. They are commonly used as foundation materials and sometimes as backfill for retaining structures.

Marl soils exhibit wide variation in terms of origin, color, mineral composition, plasticity and other engineering properties and because of these variations, there is no standard definition for marls (Qahwash, 1989). This fact can be substantiated by the different definitions provided by different researchers, as shown in Table 2.1. In addition, the mineralogical composition of such material differs significantly from place to place and even within the same place and depth. The X-ray diffraction analyses of eastern Saudi Arabia marls are summarized in Table 2.2. These values were obtained for marl samples collected from different locations in an area extending from Nariya and Abu Hadriyah in the north to Hafuf in the south and covers places, which have abundant marl soils that are suitable for construction.

Table 2.1: Marl definitions and attributes used by different authors (Aiban et al., 1999)

Author(s)	Year	Definitions and Attributes
Terzaghi and Peck	1967	Stiff to very stiff marine calcareous clays of greenish color.
Pettijohn	1975	Soil or rock like material containing 35-65% carbonate and a complementary content of clay.
Fookes and Higginbottom	1975	A simple binary mixture of calcium carbonate and clay.
Mitchell J.K.	1976 1993	Marl is ranging from relatively pure calcium carbonate to a mixture of calcium carbonate with mud and organic matter formed by biochemical processes.
McCarthy	1977	A soft limestone
Challinor	1978	A mixed rock containing clay minerals and aragonite or calcite, usually together with accessory components, such as silt, in lesser quantity.
Saudi-ARAMCO	1978	Soft limestone contaminated with varying amounts of clay
Sowers and Sowers	1979	Water-deposited sand, silt or clay containing calcium carbonate
Bates and Jackson	1980	It is an old term that is generally loosely applied to a variety of materials most of which consist of an intimate mixture of clay and calcium carbonate.
Mitchell R.S.	1985	Soft calcareous clay-rich mineral
Blyth and de Freitas	1985	Calcareous mudstone
Al-Tayyib et al.	1985	Carbonate soils, the formation of which is attributed to physical and chemical weathering of parent carbonate rocks
McLean and Gribble	1985	Friable carbonate earths deposited in freshwater lakes
Qahwash	1989	Calcareous sediments
Aiban	1994a	Fine-grained calcareous sediments

Table 2.2: XRD mineralogical composition of some eastern Saudi marl soils (Aiban et al., 1999)

Marl	Calcite(%)		Dolomite(%)		Quartz (%)		Others*(%)	
symbol	-#40	-#100	-#40	-#100	-#40	-#100	-#40	-#100
M-ABH1	73	90	-	-	27	10	-	-
M-ABH2	96	80	1	1	3	18	-	100
M-ABH3	31	40	66	55	3	5	-	-
M-ABH4	65	29	31	67	3	4	-	-
M-ABH5	29	15	67	83	4	2	-	-
M-ABH6	54	70	11	13	35	16	-	1
M-ABH7	70	4	13	87	16	3	1	6
M-ABH9	96	80	1	1	3	18	-	1
M-ABQ1	16	12	34	47	48	38	2	3
M-ABQR1	9	-	63	43	28	57	-	-
M-ABQR3	5	13	20	38	72	45	3	4
M-AIND	-	-	81	78	13	13	25	9
M-BAG1	-	-	75	79	16	13	4	9
M-BAG2	6	19	75	35	-	17	19	29
M-DHA1	3	3	97	97	-	5	-	5
M-SHD1	28	71	-	-	70	23	2	6

*include illite, sepolite, montmorillite, playgorskite and gypsum
-#40 implies material passing ASTM Sieve # 40
-#100 implies material passing ASTM Sieve # 100

2.2 Engineering Properties of Calcareous Sediments

2.2.1 General

Calcareous sediments are considered the best candidate for road bases and foundations in eastern Saudi Arabia. These materials are heterogeneous in nature and their engineering properties and performance under the prevailing harsh environmental and loading conditions have not yet been fully researched. In addition, most of these sediments have acute water sensitivity, high grain crushing, non-plastic fines and, by definition, high carbonate contents. Furthermore, the available classification systems and some of the standard testing procedures may not be applicable and may result in erroneous results (Aiban et al., 1999).

Although calcareous sediments may differ significantly from one location to another, carbonate soils may still be characterized by a number of common features. Carter et al. (1999) reported the most common characteristics, after Le Tirant and Nauroy, as follows:

1. The individual grains largely composed of bioclastic material, are extremely angular and weak.
2. The degree of cementation of carbonate soils varies considerably.
3. Carbonate soils are generally highly compressible, and this results from the combined effect of relatively large porosity (both inter-and intra particle), and the irregularity and brittleness of the particles.

4. The particle type, grain size distribution, degree of cementation and mechanical properties such as shear strength, compressibility and permeability can significantly change over short distances.

Calcareous sediments exist usually as consolidated or cemented carbonate deposits. Marl soils usually contain some impurities like, gypsum, anhydrite, aragonite, calcite, expansive clay, sand, chert, quartz geodes and others, which influence not only the marl appearance and structure but also its behavior and engineering properties in general (Aiban, 1995).

Various engineering classification methods have been suggested for these soils. These methods are based on combinations of some parameters such as carbonate content, structure, consistency, cementation, grain size, void ratio, compressibility and strength. From an engineering point of view, engineers must account for the variations in calcareous soils forms, since each form has its own characteristics. For example, when calcareous soils are sufficiently indurated, they may be classified as weak rocks and their strength will qualify them for use in different engineering projects. On the other hand, they may show high compressibility (when they are weakly cemented), causing great problems when used as a base material in different engineering projects (Carter et al. 1999).

According to Fookes and Higginbottom (1975), the main variables, which influence the engineering properties of calcareous soils, are:

1. Mineral composition.

2. Type of carbonate minerals present, whether calcite ($CaCO_3$), dolomite ($CaMg(CO_3)_2$), or siderite ($FeCO_3$).
3. Origin and forming process, whether the formation is due to physical or chemical weathering of parent carbonate rocks.
4. Grain size.
5. Degree of cementation.

2.2.2 Strength Characteristics

California Bearing Ratio test (CBR) is commonly used to assess the bearing capacity of soils. It is used by different agencies because it is not time consuming, easy to perform (in both laboratory and field) and cost effective compared to other sophisticated testing techniques such as the Modulus of Resilient test. The CBR values for calcareous sediments are highly dependent on the molding water content, density and maximum particle size. Both soaked and unsoaked CBR values are higher near the optimum water content and decrease as the molding water content decreases below or increases above the optimum value. According to Aiban (1995) the reduction of the CBR values on the dry side of optimum is caused by the lower density and the porous structure, while the lower density and the softening of the cementing salts by water, which weaken the connectors, cause the reduction on the wet side of optimum. In addition, the presence of excess water may results in a higher lubrication, which will increase particle crushing, and this will lead to finer gradation. Sometimes calcareous sediments show high CBR values, which may exceed 100% depending on the water content. This can be attributed to the common presence of high percentages of stony particles with enough fines in calcareous sediments.

However, this phenomenon still needs careful investigation since CBR values in excess of 100% is considered meaningless (Terrel et al, 1984).

Horta (1988) reported that the bearing capacity of calcareous soils increases with increasing the carbonate content in addition to other factors such as granularity, fines content, sizes of fine particles, shape of coarse particles and hardness of coarse particles. Akili (1980) observed that maximum CBR values occur on the dry side of optimum, this is substantiated by the CBR test results reported by Aiban et al. (1999), which show that the maximum CBR values were attained at moisture contents lower than the optimum moisture content values (i.e. on the dry side of optimum). In addition, in calcareous sediments, the variation of the unconfined compressive strength with molding water content is similar to that of the dry density and CBR variation (Aiban, 1999).

According to Aiban (1995) the angle of internal friction for calcareous sediments passing the ASTM No.4 sieve is not highly dependent on molding water content although ϕ values show a slight decrease as the moisture content increases. On the other hand, cohesion values show high dependency on the molding water content despite the fact that calcareous soils are usually non-plastic.

2.2.3 Effect of Gradation

Soil gradation is one of the engineering properties that affects the engineering behavior of most soils. Calcareous soil gradations, which can contain a broad spectrum of particle sizes, ranging from clay to large boulders, have great effects on their behavior. Ramadan (1999) performed a series of odometer tests on undisturbed samples of calcareous soils, collected form the Egyptian Western Desert, and concluded that the samples containing high fine percent exhibit considerably less collapse potential than samples containing lower fine content. The gradation of the samples shows a correlation with their salt content. It was observed that samples with high fine content contain salt of about twice the amount measured in samples with low fine content. Regarding the compressibility of the samples, it was observed that samples with high amount of fines compress less than samples with coarse gradations. In addition, the fine content affects the bearing capacity of calcareous soils, as indicated by Horta (1988), who stated that high CBR values were obtained only when the fine content lie below 35%. This is because 30 to 35% fines. is generally considered the maximum amount of fines for which the friction between the coarse particles of a soil can be mobilized.

According to pile driving records, carbonate soils with relatively low fine content exhibit higher strength characteristics than those with high fine content. As a result, Johnson et al. (1999) concluded that carbonate soils must have relatively low fine content to be considered for improvement (i.e. upgraded) of its strength parameters. Alsanad and Albader (1990), while investigating the leaching effects on the engineering properties of some calcareous soils in Kwait, noticed that leaching increases with the increase of the fine content of the soil.

The specific gravity of the fines for cemented calcareous soils is higher than that of the coarser particles. In addition, the grain size distribution by weight of calcareous soils, like calcrete, is found to be overestimating the volume occupied by fine particles (Datta et al. 1982).

2.3 Graded Base Course Material Characterization

2.3.1 General

Many tests have been devised for determining the characteristics of the graded base materials for pavements construction. Some of these tests are arbitrary in the sense that their efficiency lies in the correlation of the results with the field performance. By such correlation, many materials testing procedures have been standardized; the procedures must be followed at all times in order to obtain reproducible results. However, it is not uncommon to find some discrepancies in test conditions and results for a particular test procedure.

There are many standard tests to characterize materials for use as a base material in pavement construction. Municipalities and consulting authorities use certain tests in order to assess the quality of the material before its use in construction. The types of tests are usually defined in order to predict the basic strength and durability parameters that are necessary for proper pavement construction with high level of service for the paved road. Locally, in eastern Saudi Arabia, the basic tests specified by the municipalities are, gradation, plasticity, California Bearing Ratio, compaction, Los Angles abrasion, sand equivalent and soundness of aggregates. For each test there is a limit beyond which the material will not be eligible for use. For example, consultants usually specify upper and lower limits for base gradation. The gradation limits used by Saudi Ministry of Communications and Dammam Municipality are shown in Fig. 2.2.

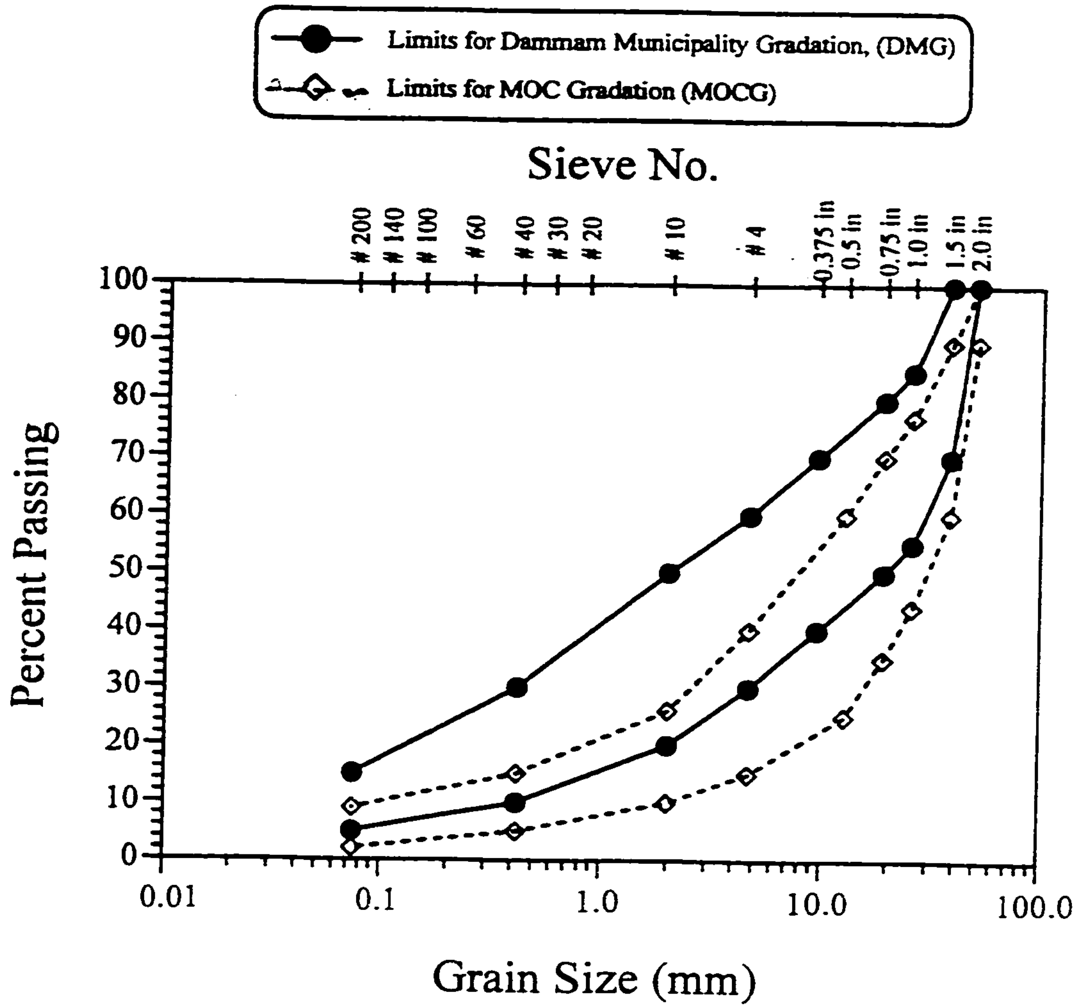


Figure 2.2: Some of the base gradation limits used in Saudi Arabia (Aiban and Al-Abdul Wahab, 1997).

2.3.2 Compactibility

Generally, the density of soils is the most important parameter that affects the strength, compressibility, swelling, permeability and the other engineering properties. The common practice, in characterizing base course materials, includes performing compaction test, in order to determine the maximum dry density and the optimum water content. For the Dammam municipality, and after realizing the acute water sensitivity of marl soils, the pre-qualification procedure requires superimposing the compaction and the CBR curves in one figure. Hence, the range of moisture contents within which the soil must be compacted is given such that it produces a certain CBR value, which is specified for each project.

In order to illustrate the effect of the maximum grain size, the compaction curves for calcareous sediments from Eastern Saudi Arabia (Abqaiq area) were determined using the modified proctor test (ASTM D1557) for samples containing particle sizes up to 19 mm (3/4 inch) and for samples passing ASTM sieve No. 4. The difference between the two compaction curves shown in Fig 2.3 reflects the effect of the maximum grain size. It is clear from the figure that samples with larger particles have higher density and lower optimum moisture content values compared to those with finer particles. Therefore it is expected that the field density will be higher than the laboratory value because of the presence of particles larger than 19 mm sieve in the field (Aiban, 1995). However, in the laboratory, this is usually accounted for using one of the standard oversize correction methods.

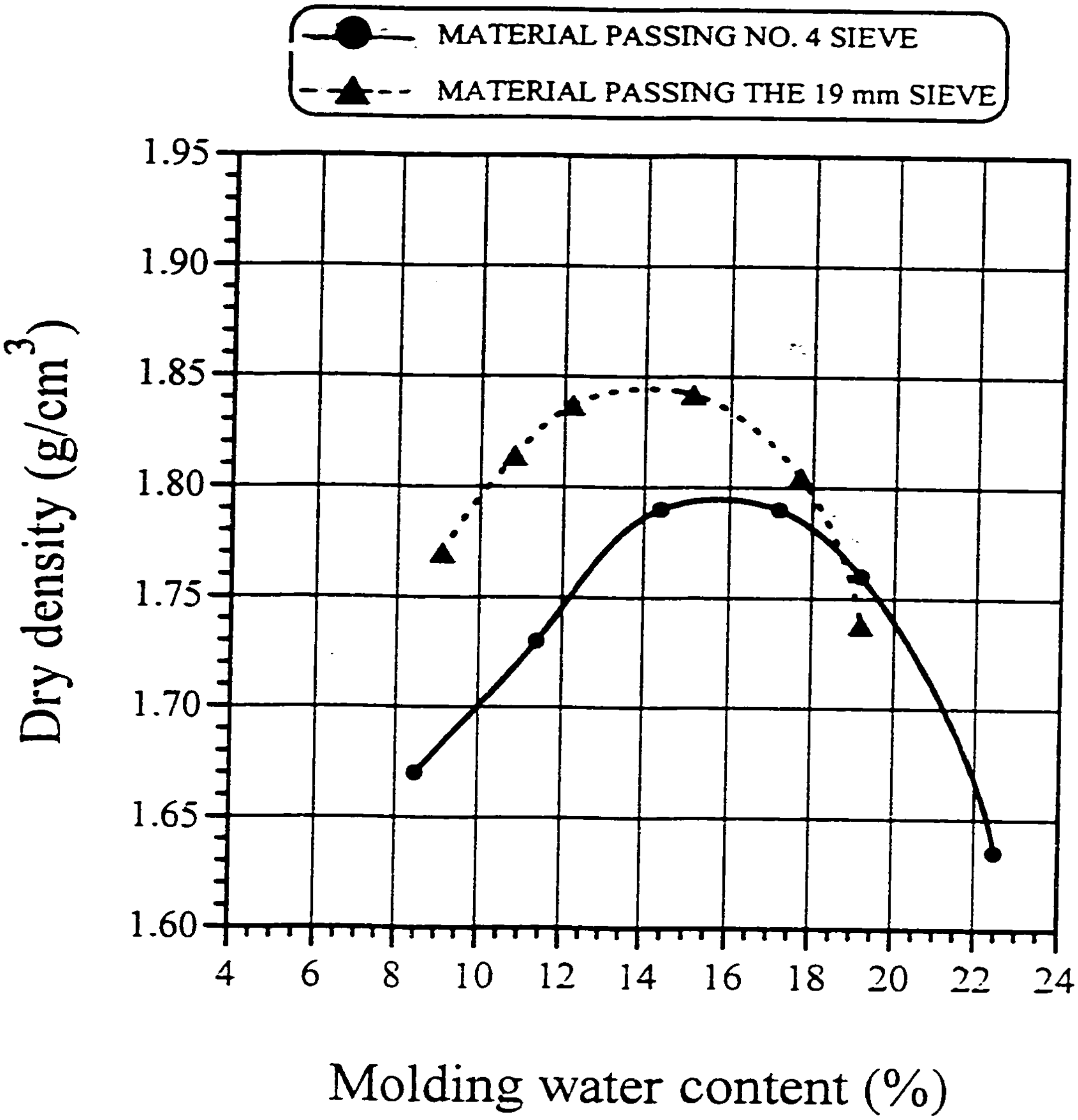


Figure 2.3: Moisture-Density curves for Abqaiq marl (Aiban, 1995).

In addition, the performance of fine-grained calcareous sediments is greatly influenced by the preparation method. A remarkable difference in behavior was observed between samples prepared at optimum water content and those prepared on either the dry or wet sides of optimum. Samples compacted on the dry side of optimum had lower strength (CBR and unconfined compressive strength) in both as molded conditions and upon inundation, and higher compressibility upon wetting compared to samples compacted at optimum. Furthermore, samples compacted on the wet side of optimum had lower strength and higher compressibility compared to those compacted at optimum moisture content (Aiban, 1995).

Some marl samples from eastern Saudi Arabia were tested after being mixed with different sand contents and all mixes gave lower optimum moisture content values as more sand was added. This was caused by the lower water affinity of the sand and to the total decrease in fines as more sand was introduced into the mix. The reduction of fine content of any sample will reduce its specific surface and hence its tendency to absorb water. Moreover, the range of moisture content from the dry side to the wet side of optimum moisture content was reduced when the amount of the added sand increases. This is attributed to the absorption of great amount of water when more marl is present (Qahwash, 1989).

The compactibility of soils in general, and calcareous soils in particular, is very much affected by the degree of grain crushing. The amount of particle breakage of a soil sample is defined by the particle size distribution curves measured before and after loading.

According to Hardin (1985) the amount of particle crushing that occurs in an element of soil under certain stress depends on the following parameters:

1. Particle size distribution.
2. Particle shape.
3. State of effective stress.
4. Effective stress path.
5. Void ratio.
6. Particle hardness (hardness of the cementing material or the weakest constituent of a particle and weakest particles of the element).
7. Moisture content.

The grain crushing of calcareous sediments from eastern Saudi Arabia, was studied for both dynamic and static compaction tests, to identify the effect of compaction method on particle breakage. The results indicate that grain crushing occurred during compaction and it highly depends on the compaction method and molding moisture content. The energy applied in each test causes two actions, compression of soil mass and rearrangement of soil particles. During the compaction process, high contact stresses are generated and considered responsible for the crushing of the aggregates. It is found that the maximum crushing of particles was observed in the case of static compaction. This is mainly because of the difficulty of the rearrangement of particles. The molding moisture content contributes to the grain crushing by lubrication. Grain crushing was less for samples compacted on the wet side of optimum for all compaction methods except for modified proctor test. It was also observed that the extent of crushing at optimum and dry of optimum moisture content was almost the same (Ahmed. 1995).

2.4 Evaluation of the Load Carrying Capacity of Soils Using CBR Test

2.4.1 General

The CBR test is performed by pushing a steel plunger with a standard cross-sectional area (3 in.^{2} (1290 mm^2)) into a specimen at a fixed rate of penetration and measuring the force required to establish a certain penetration (0.1 in. or 0.2 in. (2.5 mm or 5.0 mm)). The California Bearing Ratio (CBR) value is obtained from the load – penetration curve. The test was developed during the 1930's at the laboratory of the Materials Research Department of the California Division of Highways, USA. The classical methods for the assessment of the quality of materials for use in graded base and subbase layers were found to be no longer adequate especially after the increase of the volume and weight of traffic. The CBR test was recommended to the American Society for Testing and Materials (ASTM) as a standard test by Stanton in 1944, and is now designated as ASTM D1883. The procedure was then developed to enable its use for airfields construction by Parter and Davis in 1949 (Head, 1982).

After the development and standardization of California Bearing Ratio method for pavement design and its wider adoption, different test procedures were introduced. The equivalence of these procedures and the conditions under which they should be used has led to an increasing confusion and a need for careful investigation.

In general, the CBR test can be performed on:

(a) Undisturbed samples that are cut from the ground and carefully trimmed to fit the standard mold.

(b) Remolded samples prepared in the standard laboratory mold.

(c) In-situ on the surface of either the natural soil formation or a compacted layer.

Researchers reported many discrepancies between laboratory and field CBR values even when both tests were performed at the same density and moisture content. The discrepancies can be attributed to the differences between test conditions such as the effect of mold confinement, gradation, preparation (compaction) method and the oversize particles. However, CBR values can differ even when the tested samples are prepared under the same testing conditions having the same density, moisture content, gradation.... etc. This is mainly because soil samples compacted to nominally the same conditions may have different pore water pressures, which affects the test results (Black, 1961).

2.4.2 CBR-Density Relationship

The CBR value of a soil is highly dependent on the dry density and moisture content. Because of this, it is always convenient to relate CBR values of remolded samples to moisture-density relationship of the soil, obtained by the compaction method used for preparing CBR samples. For soils in general, and cohesive soils in particular, for a given degree of compaction the CBR values of a soil decrease with the increase of moisture content beyond the optimum. According to Head (1982), Davis (1949) reported that, the decrease of CBR values with the increase of moisture content, beyond the optimum, is sharper for granular soils. A general relationship between CBR and moisture content is shown in Fig. 2.4.

For calcareous sediments, the CBR value is known to be water sensitive. Generally, the CBR values increase with increase of the molding water content till a peak is reached near the optimum water content, after which the CBR values start to decrease on the wet side of optimum as the moisture content increases. Such decrease is very sharp for marls and is usually observed for the CBR samples prepared on the wet side of optimum moisture content. A decrease in the CBR values from 211% to 76% occurred when only 0.9% water, by weight of soil, was added to the sample during compaction as shown in Fig. 2.5. This can be attributed to the softening of calcareous particles and the matrix supporting the coarse particles.

2.4.3 Effect of Preparation Method

Specimens prepared in the laboratory, whether by static compression or dynamic compaction, will not necessarily give the same CBR values as those obtained in the field, or on undisturbed samples taken after being compacted in the field, to the same density and at the same moisture content. Head (1982) listed the following reasons for such discrepancies:

1. The non-homogeneity of the density distribution within the soil layer in the site when compared to the laboratory compacted specimen.
2. Moisture changes can occur quite rapidly in the exposed formations in the site.

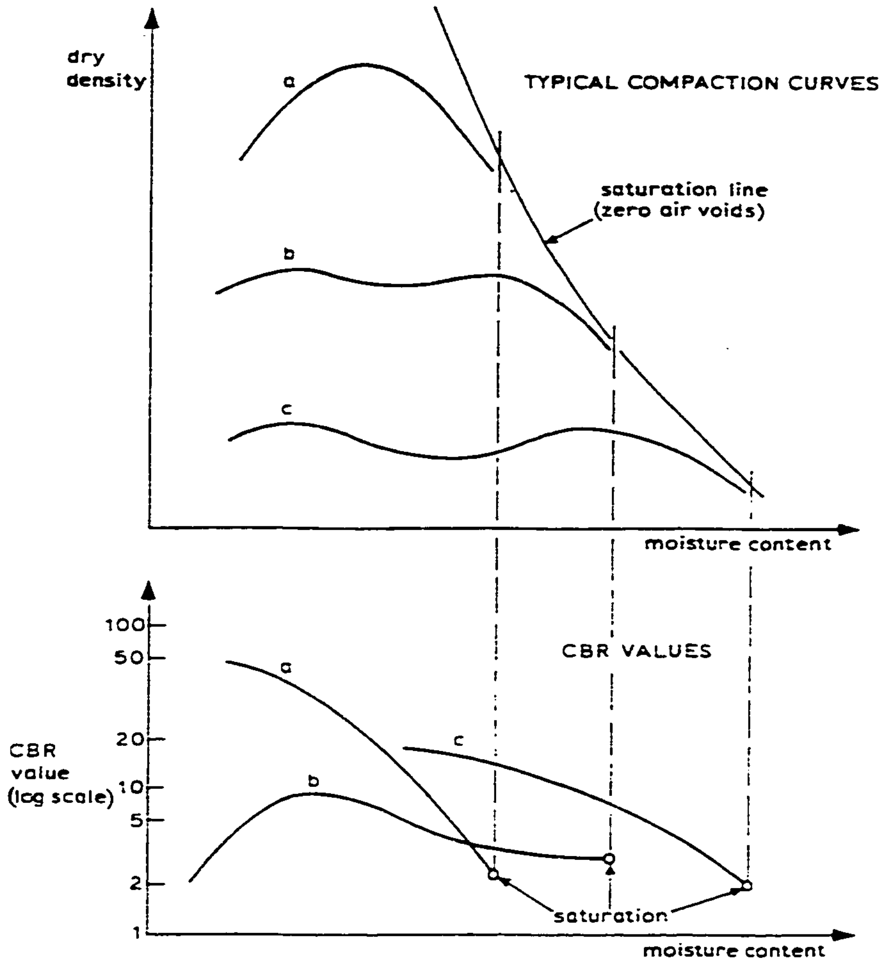


Figure 2.4: Variation of CBR and density with moisture content for typical soils: (a) Well-graded silty sand with clay, (b) uniform fine sand, (c) heavy clay, (Head, 1982).

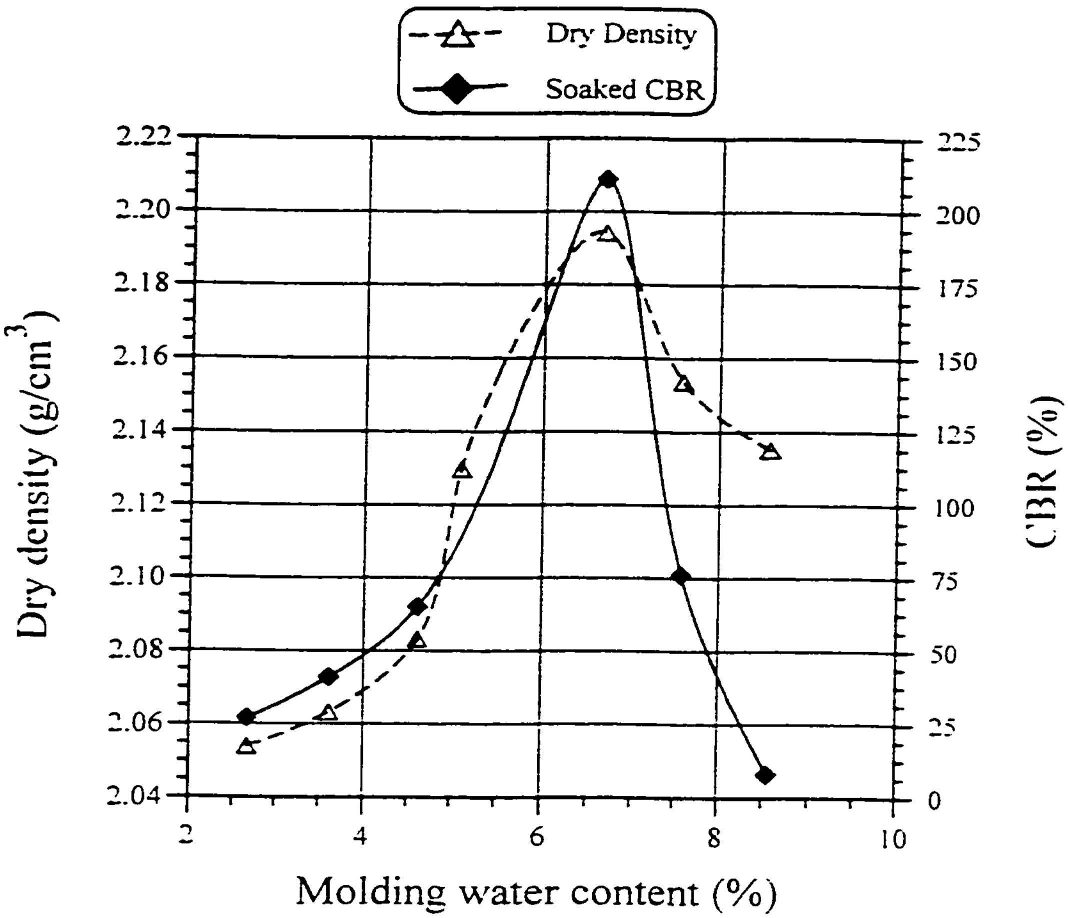


Figure 2.5: Typical variations of the dry density and CBR values with the molding water content for marl soil from eastern Saudi Arabia

3. The boundary conditions, caused by the effect of edge restraint of the compaction mold, are absent in field condition; such effect varies with the soil type.

4. Remolding the natural soil in the laboratory destroys its original fabric, and may lead to different gradation.

5. There are also differences between the static, dynamic and vibration compaction procedures for the preparation of laboratory specimens. Comparisons of CBR results between one method of preparation and another may not be valid.

It was found that, deformation of samples before failure varies depending on the method of compaction, which affects the lateral pressure in the CBR mold. For samples compacted by rammer, the soil was remarkably unstable and shows large strain at failure. When compacting by a large static load, the strain at failure was reduced to one-sixth of the value for the soil compacted by the rammer. The third compaction method is performed by dropping the mold 40 drops, when it is full of soil and surcharged with 30 lb (13.6 kg) weight, with a drop height of 18 in. (457 mm), this method gave an intermediate result between those of the previously mentioned tests (Black, 1961).

2.4.4 Effect of Oversize Particles

The CBR test is useful for evaluating subgrade soils, subbase and base coarse materials containing only a small amount of material retained on the ¾ in. (19 mm) sieve (AASHTO 1982). In the field, soils may contain gravel, fragments of rock, shale, brick or other hard material but for laboratory samples, material coarser than ¾ in. (19 mm) is excluded during compaction and an oversize correction for the density is usually made. The density and CBR values obtained in the field for the total material containing all sizes cannot be compared directly with the results of laboratory prepared samples. For a soil specimen compacted in the laboratory, the presence of large size particles gives the material higher density compared to the material in which large size particles were excluded. This is mainly because the large particles have greater density than the matrix material they replace. Several attempts were made to calculate the resulting insitu density of the whole material.

When using the standard Proctor compaction mold, which is 4 in. (101.6 mm) in diameter and 5 in. (127 mm) deep, only 20% or less by weight of the material retained on No. 4 (4.75 mm) sieve can be included (Proctor, 1933). However, for compaction in CBR mold, which is 6 in. in diameter, 30% or less by weight of the material retained on ¾ in. (19.0 mm) sieve can be included (ASTM, 2000). Therefore for material containing oversize particles, the correction for dry density and moisture content must be implemented in order to obtain equivalent values for the total material. However, there is no correction method available for the CBR test other than the scalp and replace method, which has a questionable reliability due to the commonly observed large differences between laboratory and field CBR values.

In order to investigate the effects of the particle size on the CBR values for gravelly soils, several tests were performed using different particle and mold sizes. Materials containing up to 1 ¾ in. (45 mm) particles were tested in a 9.4 in. (240 mm) diameter mold. The total material tested in the large mold, using the standard plunger size, showed higher CBR values compared to the material, which was tested in the same mold but after replacement of the oversize particles by the same weight of material passing ¾ in. sieve and retained on No. 4. This is attributed to the presence of large size particles, which leads to higher CBR values. In addition, the CBR values obtained using the standard CBR mold were found to be higher than those obtained from the large mold for the same gradation. This is caused by the effect of confinement and the fact that in the large mold the penetration was done without inverting the mold (Osman, 1995).

2.4.5 Effect of Confinement

One of the most important factors that affects the results of the CBR test and causes discrepancies between field and laboratory results is the confinement provided by the mold. The confining effect of the rigid mold in which the laboratory tests are carried out may lead to CBR values many times greater than the insitu tests. After observing that the field CBR results are not matching the laboratory test values, few investigators have studied the effect of the mold size. According to Nataamadja (1988), Morgan in 1972 showed limited data indicating that the larger the mold the smaller the CBR values for the same material.

Nataatmadja (1988) investigated the effect of the mold size on the CBR values where different mold diameters were used ranging from 4 to 10 inches (101.6 to 254 mm). All samples were tested under the standard surcharge pressure resulting from a weight of 10 lbs (4.5 kg). It was found that the CBR values obtained for different penetrations decrease with the increase in the mold diameter as shown in Figs. 2.6 and 2.7. These results

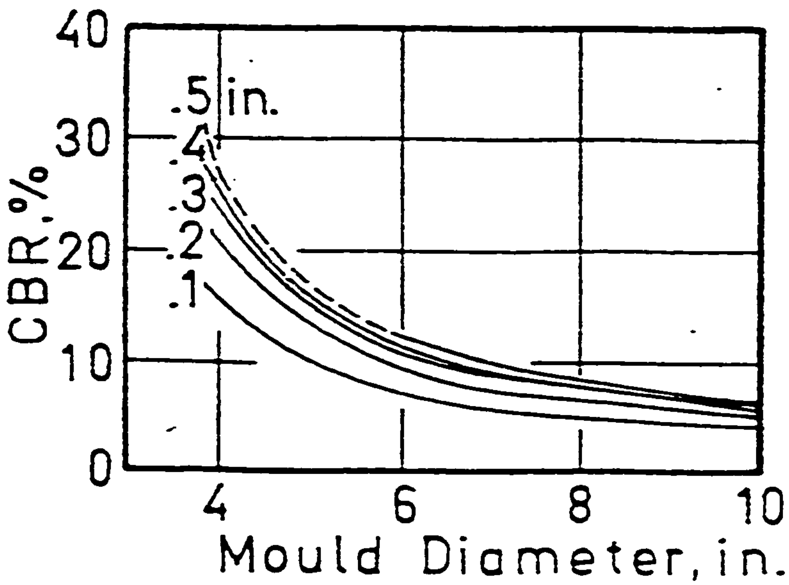


Figure 2.6: CBR versus mold diameter using the standard ASTM density (Nataatmadja, 1988).

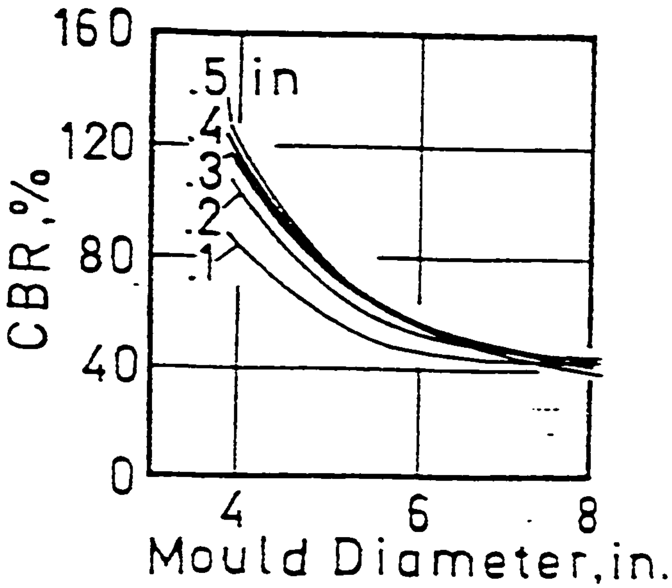


Figure 2.7: CBR versus mold diameter using the modified ASTM density (Nataatmadja, 1988).

substantiate the fact that mold size results in laboratory measured CBR values that are higher than the corresponding field values, especially for granular material.

Black (1961) stated that the mold sides arrest the displacement of the soil along the shear surface produced by the CBR plunger for a material with an internal friction angle greater than 30°. This conclusion was derived assuming that the failure beneath the plunger will take the shape shown in Fig. 2.8. However, Nataatmadja (1988) considered the data produced by Hight and Stevens (1982) and stated that a complete failure below the plunger was not guaranteed. Their finite element analysis suggested that for stiff soil a bearing capacity failure could not occur; but for soils of low stiffness the ultimate bearing capacity was not reached at a penetration of 0.2 in. (5 mm). In addition, the shear strength of the soil was mobilized only in the zone close to the edge of the plunger. Hence, they recommended the use of full load- penetration curve in order to obtain the CBR values. However, they indicated that significantly different load-penetration characteristics are likely to occur at penetrations larger than 0.3 in.

2.4.6 Merits and Limitations

The CBR test is adopted for use by many authorities because of its advantages despite some limitations associated with its use. The main merits of CBR test are:

1. It can be applied to a wide variety of soil types ranging from clay to fine gravel.
2. Test data are applicable to the design of airfield runways and taxiways as well as roads.
3. It can be performed in the field and in the laboratory.
4. It can be performed on undisturbed and compacted materials.

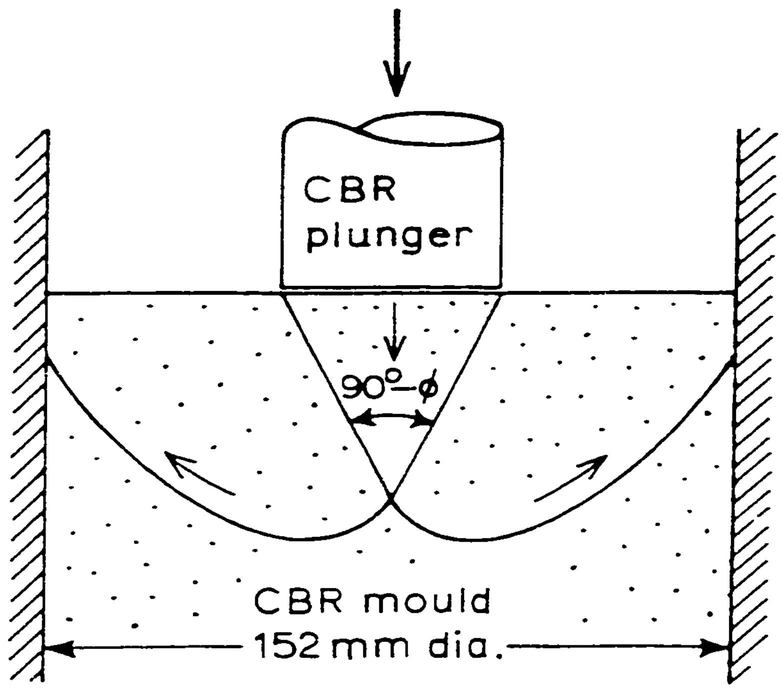


Figure 2.8: The assumed failure mechanism of a soil beneath the CBR plunger (Black, 1961).

5. The test is relatively quick and simple to operate and gives immediate results.

On the other hand, the test have some limitations, which still need further investigation, these limitations include:

1. The test results are applicable only to pavement design for which the procedure was devised.
2. The test is highly dependent on the testing conditions, hence, the results can hardly be compared with those obtained elsewhere.
3. There are no reliable correlations between the CBR values and the fundamental soil properties like the soil strength parameters, compressibility, gradation...etc.
4. The test has less reliability when used for materials containing oversize particles. Furthermore with the exclusion of the oversize particles from the tested sample, there is no reliable correction method available to find the corresponding CBR values for the total material.
5. The laboratory and field test results are still not accurately comparable. This may be caused by the effect of mold confinement, and the differences in pore water pressure, compaction methods and soil gradations, especially with the presence of oversize particles.

2.4.7 Correlation Between the CBR and Soil Strength Parameters

There is no reliable correlation between CBR and the fundamental soil properties governing shear strength or compressibility. One of the widely used correlations is the one developed by ASSHTO Guides for Design of Pavement Structures, in which scales were obtained to calculate the "layer coefficients" that have traditionally been used in the original AASHTO flexible pavement design procedure (AASHTO Guide 1986). Relations between CBR, soil support value (R), Texas Triaxial and Modulus of Resilience can be obtained using the scales shown in Fig. 2.9. However many of these correlations would not yield the correct answer when back-correlated through an intermediate parameter. This is simply because errors are accumulated every time there is an additional correlation. For example the relation between Resilient Modulus and CBR is given by:

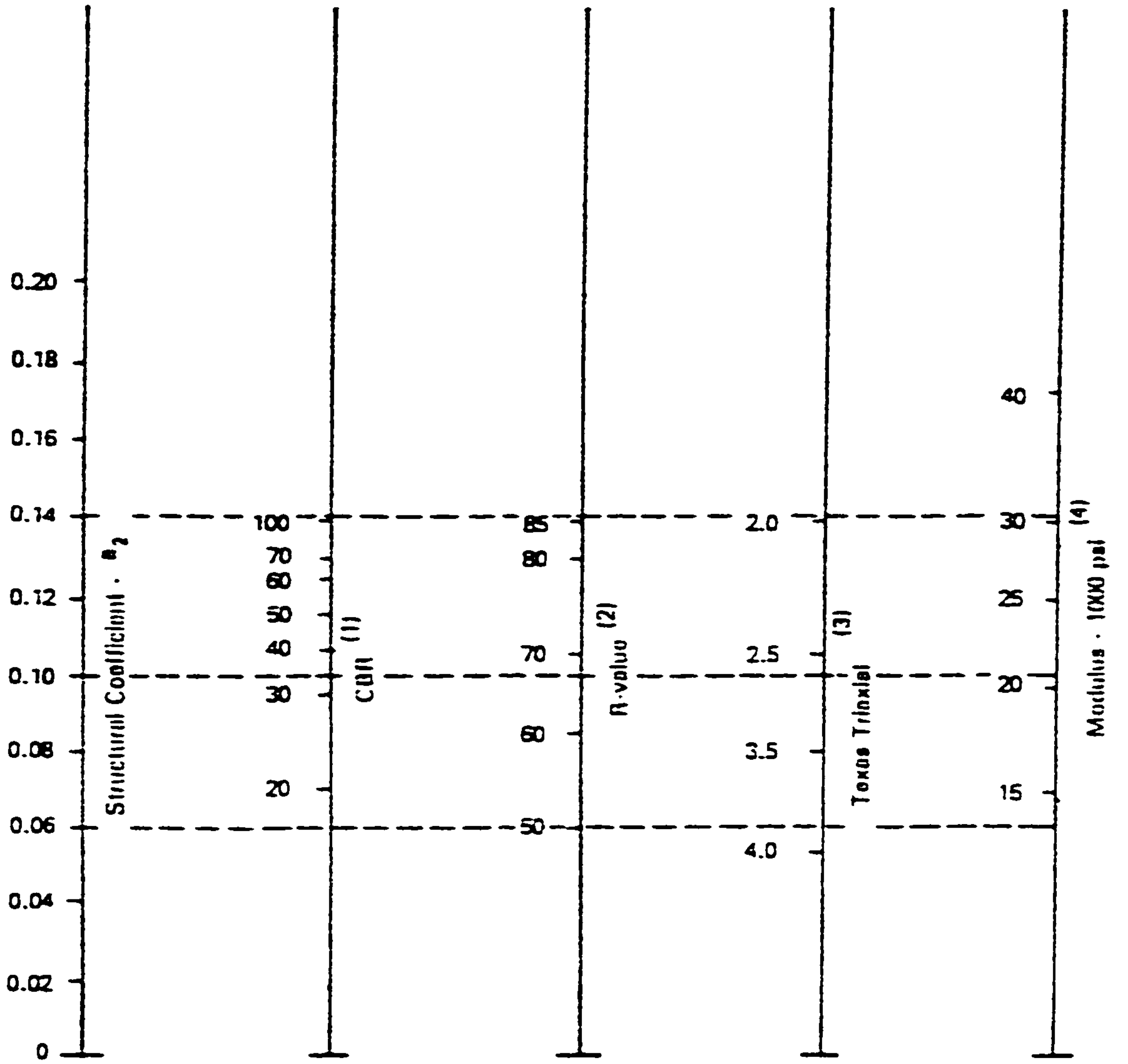
$$MR = 1500 \times CBR \tag{2.1}$$

and between Modulus of Subgrade Reaction and CBR is given by:

$$K = 10^{[1.733958+0.568048\,[\log(CBR)]]} \tag{2.2}$$

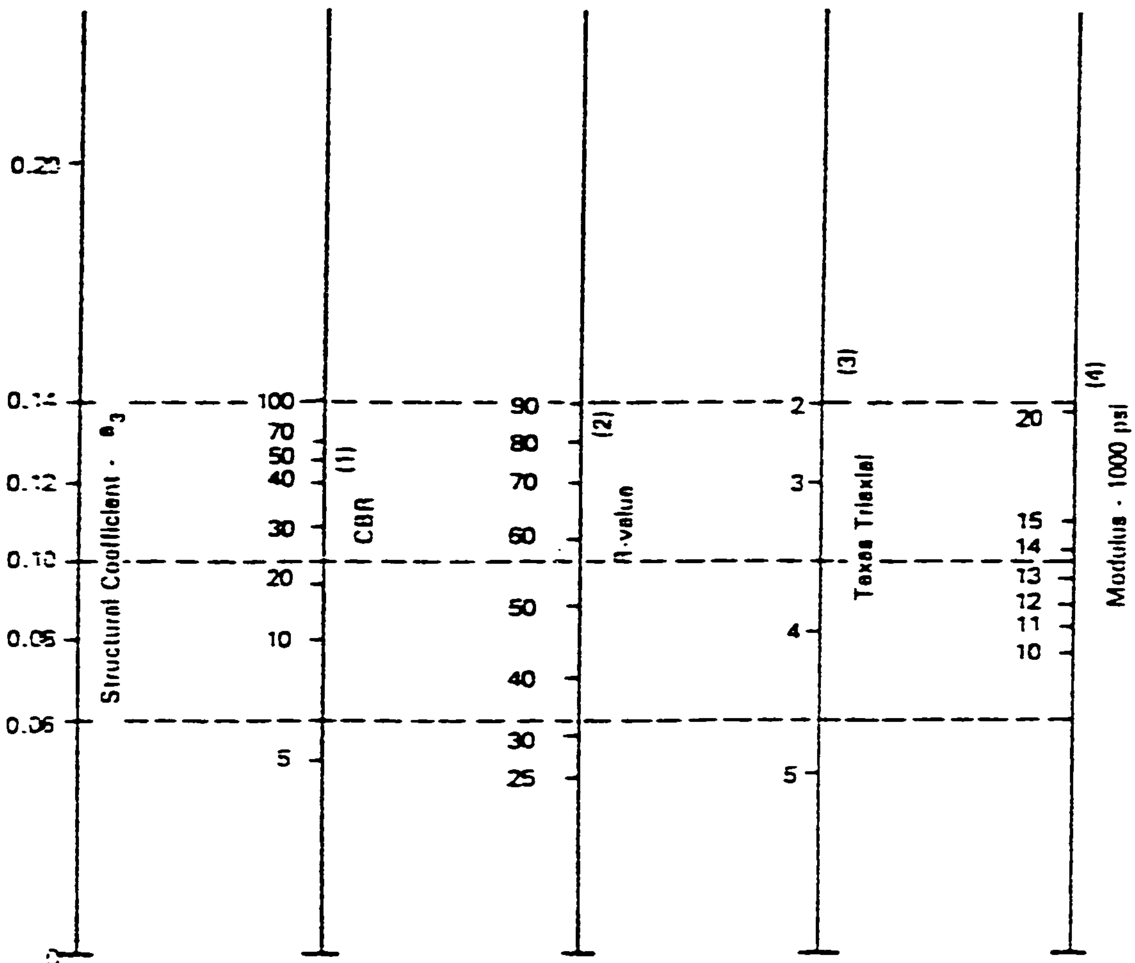
It is not recommended to obtain a direct relation between Resilient Modulus and Modulus of Subgrade Reaction using the abovementioned relations. The errors observed in back correlations are attributed to the fact that these relations cannot be utilized for all types of soil. For example the MR-CBR relation shown in equation (2.1) is not accurate since the coefficient, which is multiplied by the CBR value can range from 750 to 3000 depending on soil type (Southgate and Mahboub, 1994).

A relationship between the CBR and the unconfined compressive strength was obtained for some granular and fine-grained soil and cement mixtures (Terrel et al., 1984), as shown in Fig. 2.10. The difference between the relationships for fine grained and granular-treated soils results is probably attributed to the uncertainty associated with the use of the CBR test for coarse-grained soils.



(1) Scale derived by averaging correlations obtained from Illinois.
(2) Scale derived by averaging correlations obtained from California, New Mexico and Wyoming.
(3) Scale derived by averaging correlations obtained from Texas.
(4) Scale derived on NCHRP project *(3)*.

Figure 2.9-a: Variation in granular coefficient (a_2) with subbase strength parameters AASHTO Guide (1986)



(1) Scale derived from correlations from Illinois.

(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming.

(3) Scale derived from correlations obtained from Texas.

(4) Scale derived on NCHRP project (3).

Figure 2.9-b: Variation in (a_3) for granular subbase with base strength parameters AASHTO Guide (1986)

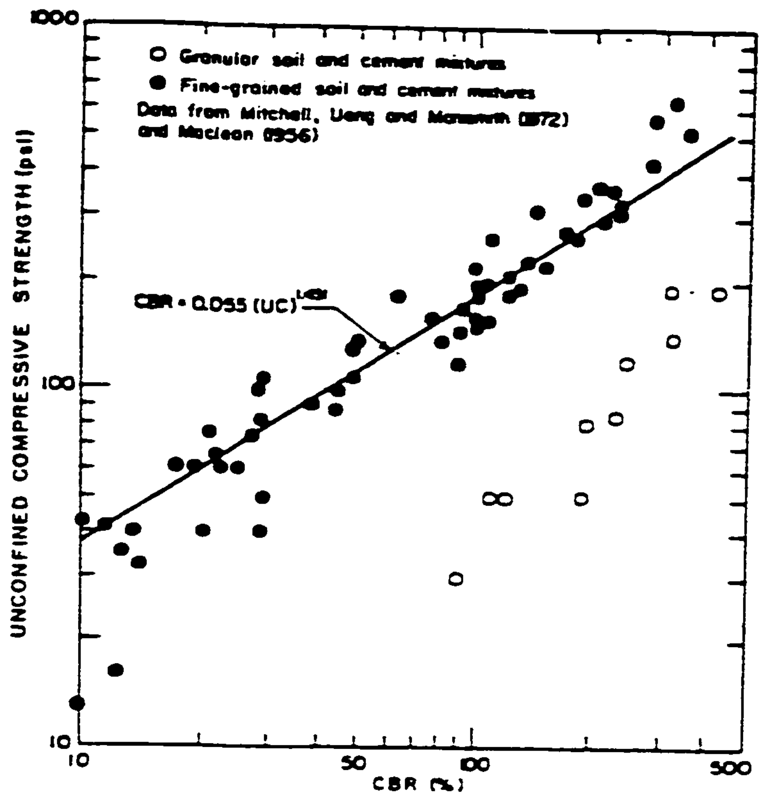


Figure 2.10: The relation between CBR and the unconfined compressive strength of soil and cement mixture (Terrel et al., 1984)

Livenh (1989) reported the following relations between CBR and the dynamic cone penetrometer (*DCP*), the dynamic probing type A (*DPA*), the standard penetration test (*SPT*) and the vane shear test (*S*) as follows:

$$Log\ CBR = 2.2 - 0.71(Log\ DCP)^{1.5} \qquad (2.3)$$

where, *DCP* is the dynamic cone penetration test result value

$$Log\ CBR = -5.13 + 6.55\ (Log\ SPT)^{-0.26} \qquad (2.4)$$

where, *SPT* is the Standard Penetration test result value

$$Log\ CBR = 2.2 - 0.45\ [Log\ (3.47DPA)]^{1.5} \qquad (2.5)$$

where, the *DPA* is the Dynamic Probing (type A) test result value

$$CBR = 4.79\ S^{0.63} \qquad (2.6)$$

where, *S* is the Vane Shear test result value

2.5 Compactibility of High Gravel Content Soils

2.5.1 General

Due to the material availability and some times cost considerations, engineers are increasingly using fill materials containing gravel to boulder size particles. However, the assessment of the quality of the fill needs careful consideration since all conventional testing methods excludes oversize particles before testing. The term oversize refers to particles, which are too large to be included in a particular test apparatus; therefore, it is not fixed dimension. The determination of certain parameters like dry density and optimum moisture content, for the total material, is necessary for any pre-qualifications tests for fieldwork.

The use of laboratory determination of a reference dry density for compaction control in the field is based on the implicit assumption that the material compacted in the laboratory is equivalent to the material compacted in the field. However, when the fill material contains gravel or rock, this assumption is generally not valid, because large aggregate sizes are replaced by finer aggregates. Several methods are available to account for the effect of excluding the coarse fraction on the reference dry density and the associated optimum moisture content. Each technique adopted by the engineer should be clearly stated in the compaction specifications to reduce the possibility of conflict with the contractor. The method for accounting for the rock fraction will have a significant impact on the evaluation of the reference density. Modifications to the compaction standards, such as ASTM D698, could significantly affect compactive effort for soils with high rock content unless the required relative compaction values are adjusted to account for the differences in the methods used to adjust the maximum dry density (Walsh, 1994). For the 6 in. diameter mold, the oversize fraction is considered to be the portion retained on the ¾ in. sieve. The limiting percentage of oversize particles for which the correction is valid may be lower than the limiting percentage when the oversize fraction is considered to be the portion retained on the ASTM No. 4 sieve. Therefore, contractors, construction inspectors and designers should be consistent in the method adopted for rock correction, and should follow the procedure outlined in a well-written specification.

Winter (1998) conducted an extensive laboratory test program to determine the proportion of particles larger than 20 mm at which the transition from matrix to stone behavior (the behavior which is governed by the significant numbers of stone-to-stone contacts) occurs. In general, it was found that for an increase in stone content up to 45% to 50%, there was an increase in maximum dry density and a decrease in the optimum moisture content. This increase in density does not necessarily represent improvement in the stability of the soil but reflect the replacement of high moisture content, low density matrix material with low moisture content, high density stones. For stone contents above 45% to 50% a decrease in the maximum dry density was observed in addition to the decrease in the optimum moisture content. If the proportion of particles larger than 20 mm is greater than 45% to 50% then the stones will determine the behavior.

However, it was found that, for soils with less than 45% to 50% of particles larger than 20 mm, the matrix would determine the compaction behavior.

Garga and Madureira (1985), investigated the effect of different factors on the compactibility of a gravelly soil from Sao Simao Hydro Power Dam site in Brazil and they listed the following observations:

1. After studying the effect of gravel content, the percentage of gravel fraction was found be an important factor influencing the maximum density of gravelly soils. At high gravel content, there is an increasing gravel-to-gravel contact, which interferes with transmission of compaction energy to the finer material in the voids. In addition, at high gravel content, there may also be insufficient fines to completely fill the voids. For the tested soils, it was found that at gravel content of approximately 20-25%, particle interference begins to affect the compaction of fines. For material passing the ASTM No. 4 sieve, the dry density values decreased with increasing gravel content. However for material passing the ¾ in.

(19 mm) sieve, densities achieved by this fraction were found to increase with the increase of gravel content.

2. The influence of the energy of compaction was investigated and results showed that for materials with similar grain size distribution, regardless of the maximum particle size, the difference between the maximum dry densities for standard and intermediate energies is independent of the percentage of gravel fraction. The maximum density values, under both the modified and intermediate energies reach the same value at gravel content between 65 and 70%. This may be attributed to the particle interference. At low gravel content. approximately less than 30%. there is insignificant particle interference and the density values are proportionally dependent on compactive effort. While at gravel contents greater than approximately 60-70%, it is not possible to obtain denser arrangement as a result of particle interference and insufficient amount of fines to fill all void spaces.

3. Different mold sizes were used to study the effect of mold size on the compactibility of gravely soils. Mold diameters of 4 in. (100 mm), 6 in. (152 mm). 12 in. (305 mm) and 20 in (508 mm) were used. The maximum density values were obtained from a mold diameter eight times the maximum particle size. Despite the use of the standard compactive effort for all mold sizes. some differences in dry densities were observed. These differences may be due to the different impact pressures produced by the rammers.

4. Two different samples were used to investigate the effect of the water absorbed by gravel fraction on the maximum dry density. The results indicated that the variation in the absorbed water content of the gravel fraction does not influence the maximum dry density values. It was concluded that low values of absorbed water content, e.g., up to 3%, can be neglected in calculating the water content of the total sample.

5. The effect of maximum particle size was studied using the maximum dry density attained by the 3 in. maximum size materials as a basis for comparison. The densities were found to be identical up to 40% gravel content, regardless of the particle size. With a subsequent increase in gravel content, the density of the ¾ in. maximum size fraction decreased by 0.9% for 50% gravel content, and by 1.4% for 60% gravel content. These differences in densities are attributed to an increase in uniformity of the coarse fraction with higher gravel content.

6. Two pairs of samples with gravel contents of 50% and 60% were prepared with two different grain size distribution curves in order to study the effect of gradation of gravel fraction. The results of the maximum dry densities showed that the compacted density was essentially independent of the shape of the grain size distribution curve. The maximum difference between the obtained dry densities was less than 0.5%.

7. For the tested soil, the gravel content was found to have an inversely proportional relation with the optimum water content values. The optimum water content of the total material for the tested soil was found to decrease by approximately 1% for each 10% increase in gravel content. This decrease may be attributed to the absorption of the compaction energy by the coarse fraction.

2.5.2 Methods of Estimating the Dry Density for Oversized Material

Several methods are available to account for the effect of excluding the coarse fraction on the dry density. The available methods may be categorized as rock correction equations and laboratory testing modifications. Each of these methods is used to obtain the dry density of the oversized material. However, not all methods produce the same dry density for use as a reference value. Some of the well-known techniques include:

a) Small-scale tests without replacement (Elimination Method)

The test consists of performing small-scale compaction tests for which the material passing the ¾ in. (19 mm) sieve is compacted in a 6 in. (152.4 mm) mold, while the material coarser than the ¾ in. (19 mm) sieve is discarded [e.g., ASTM D698, method C]. This method is used only when the material coarser than the ¾ in. (19 mm) sieve is less than 10% by weight.

b) Small-scale tests with replacement (scalp and replace method)

The test consists of performing small-scale compaction tests for which the material coarser than ¾ in. (19 mm) is removed before compaction and replaced with an equal weight of material retained on No. 4 but finer than ¾ in. (19 mm). This is done in accordance with (ASTM D698-92) or (AASHTO T99). This method is the scalp and replacement method, sometimes referred to as the procedure for replacement of oversize aggregate. However, according to Fragaszy et al. (1990), Donaghe and Townsend have shown in 1976 that scalp and replace method can give lower maximum dry unit weights than those obtained when using the total sample. This method is no longer available in the latest ASTM standards edition (ASTM, 2000), however it is still used by local agencies.

c) Large-scale tests

The test consists of performing large-scale tests using the entire material, as it is intended for field use, to get the maximum dry density and optimum water content. The test methods shall be modified in order to accommodate the oversize particles. According to Dongahe and Terrey (1994), Holtz and Lowitz reported in 1957 that the ratio of specimen diameter to the largest particle size should not be less than 5 or 6. Garga and Madureira (1985) developed by recommended a ratio of 6 to 8.

Donaghe and Torrey (1994) developed a method for determining the compaction characteristics of soil-rock mixtures having maximum particle sizes up to 2 or 3 in. (51 or 75 mm) while maintaining the standard compaction effort. This method was developed according to a testing program using 12 and 18 in. (305 and 457 mm) molds in which mold size effects are minimized. This was achieved by reproducing the results, which were obtained using a conventional 6 in. (152 mm) diameter mold.

The 12 in (305 mm) diameter mold test program was performed by duplicating, as much as possible, the parameters used in the conventional method. These parameters include the ratio of the hammer foot diameter to mold diameter, hammer drop height and the number of layers. The authors recommended the use of this procedure to determine the moisture-density relationships of soils with gravel fractions containing particles larger than the ¾ in. (19 mm) sieve and finer than the 3 in. (762 mm) sieve. This will be achieved by compacting the soil in 12 in. or 18 in. (305 and 457 mm) molds using standard compactive effort with a 131.4 lbf (584.5 N) rammer dropped from a height of 12 in. (305 mm).

Wagner (1970) proposed another method of test for moisture-density relations of gravelly soils. In his method, a cylindrical metal mold 20 in. (508 mm) diameter with a height of 15 in. (380 mm) was used. The compaction was done by lifting a metal rammer weighing 186 lb (83 kg) 18 in. (457 mm) and letting it to fall free at the rate of 12 blows per minute. This method is applicable for soil passing the 3 in. (75 mm) sieve.

d) Rock correction equations

The method consists of performing compaction tests on the material passing the ASTM No. 4 or ¾ in. (19 mm) sieve to get the maximum dry density and optimum water content for the fine material. Knowing the percentage of rock, the values obtained for the finer material are corrected to obtain an estimate of the maximum dry density and optimum water content of the entire (total) soil. This is done in accordance with ASTM procedure D4718, AASHTO procedure T224, or U.S. Bureau of Reclamation (USBR) procedure 5515-89 (Houston and Walsh, 1993).

The well-known rock correction equations are given in Table 2.3. The difference from one method to another varies depending on the material characteristics, such as the plasticity index of the material passing the No. 4 sieve (Walsh, 1994).

There exist other methods, to account for the effect of oversize particles such as the method introduced by Hsu and Saxena (1991), who proposed the following general formula for determining densities including oversize particles:

$$\gamma_t = \frac{1}{(1+e)\,[P/G_g \cdot \gamma_w) + (1-P)/G_s \cdot \gamma_w)]} \tag{2.7}$$

where,

e = The void ratio of the total material

P = The ratio between the weight of gravel to the weight of total material

G_g= The specific gravity of gravel

G_s= The specific gravity of soil binder

γ_w = The density of water.

The void ratio can be determined as follows:

$$e = e_o + A\,P + B\,P^2 + C\,P^3 + D\,P^4 \tag{2.8}$$

Where, e_o is the initial void ratio, A, B, C and D are constants which can be obtained from large scale compaction tests of a typical soil compacted with a specific energy.

Table 2.3: Some of the Rock Correction Equations (Houston, 1993)

Equation Designation	Reference	Equation*	Comments
AASHTO-1	AASHTO T224	$D = (1-P_c)D_f + 0.9\,P_c\,(62.4)G_m$	D_f is determined using Method A or B, AASHTO T99 or T180.
AASHTO-2	AASHTO T224	$D = \dfrac{62.4}{\dfrac{P_c}{G_m} + \dfrac{62.4(1-P_c)}{r_a\,D_f}}$	D_f is determined using Method A or B, AASHTO T99 or T180. r_a depends on rock content.
ASTM	ASTM D4718	$D = \dfrac{62.4}{\dfrac{P_c}{G_m} + \dfrac{62.4(1-P_c)}{D_f}}$	D_f is determined using ASTM D698 or D1557.
USBR	USBR 5515	$D = \dfrac{62.4}{\dfrac{P_c}{G_m} + \dfrac{62.4(1-P_c)}{r_u\,D_f}}$	D_f is determined using USBR method 5500. r_u depends on rock content and plasticity of fines.

* Definitions:

D_f = Maximum dry density of the fine material (pcf)

D = Maximum dry density of the total soil (pcf)

P_c = Percent rock (material retained on the No. 4 or the ¾ in. (19 mm) sieve by weight (decimal)

G_m = Bulk specific gravity of rock

r_a = Correction factor in AASHTO equation to account for interference of large aggregates

r_u = Correction factor in USBR equation to account for interference of large aggregates

Torrey and Donaghe (1994) proposed another method for compaction control of earth-rock mixtures. The proposed method was developed to calculate the maximum dry unit weight and optimum water content of the total material from the corresponding values obtained on either the material finer than ¾ in. (19 mm) or the No. 4 (4.76 mm) sieve. This method was shown to be applicable to a wide range of gradations of earth-rock mixtures. According to The maximum dry density γ_{tmax} and the optimum water content W_{topt} of the total material are calculated as follows:

$$\gamma_{t\,max} = \frac{100\ I_c\ P_G\ \gamma_{f\,max}\ \gamma_w G_M}{\gamma_w\ P_F + I_c P_G^2\ \gamma_{t\,max}} \qquad (2.9)$$

$$W_{topt} = \frac{100\ W_{fopt}}{P_G\ F_{opt}} \qquad (2.10)$$

where,

I_c = Density interference coefficient

P_G = Percent gravel

P_F = Percent finer fraction by weight

γ_{fmax} = Maximum dry unit weight of the finer fraction

γ_w = Unit weight of water

G_m = Bulk specific gravity of the gravel

W_{fopt} = Optimum water content for finer material

F_{opt} = Optimum water content factor

McLeod (1970) proposed another method for correcting the maximum dry density of compacted soils containing oversize particles. The method was intended to provide a calculated value for the maximum dry density (γ_{max}) for the total soil, when the moisture density relation tests are made on the portion of the soil passing either No. 4 or ¾ in. (19 mm) sieves. The calculated γ_{max} value for the total soil is required for comparison with the measured field density value for a layer of soil undergoing compaction in the field. The maximum dry density of the total soil is calculated as follows:

$$W_t = \frac{W_o W_c}{OW_c + CW_o} \qquad (2.11)$$

where,

W_t = calculated maximum dry density.

W_o = density of the oversize material as given by its ASTM bulk specific gravity multiplied by the density of water.

W_c = measured maximum dry density of the portion of the total soil passing either No. 4 or ¾ in. (19 mm) sieve.

O = fraction by weight (dry basis) of the portion of the oversize particles in the total soil expressed as decimal.

C = fraction by weight (dry basis) of the portion of the total soil passing either No. 4 or ¾ in. (19 mm) sieves, expressed as decimal.

This method is applicable only when the volume of the compacted portion passing either No. 4 or ¾ in. (19 mm) sieve is at least sufficient to fill the void spaces between the oversize particles.

Day (1989) investigated the relative compaction of fill having oversize particles using three different methods; the methods used in the research were the elimination method and the replacement method using the following equation:

$$\gamma_{adj} = \frac{1-0.05(F)}{\frac{F}{162}+\frac{1-F}{\gamma_1}} \tag{2.12}$$

which obtained from the Navy Engineers Command Manual. In this equation:

γ_{adj} = The adjusted total maximum dry density (*pcf*)

γ_1 = The laboratory maximum dry density of the soil matrix (*pcf*)

F = The fraction of oversize particles by weight

This method is considered suitable when the oversize percentage is less than 60% by weight, for well-graded materials.

The three methods used did not give the same result for the tested soil. Assuming that the fill must be compacted to 90% relative compaction, the elimination method required the highest field dry density values, while both the replacement and the equation methods required the same values of field dry density, although the replacement method cannot be used for F values greater than 30%.

Hence, after this literature survey it is observed that different researchers studied the effect of the particle size on the compactibility of gravelly soils and recommended different correction methods to account for the removal of the oversize particles from test specimen. However, there is no attempt made to study the applicability of the conventional tests for local soils. In addition, the reliability of the common correction methods for local use is still not investigated. Furthermore, the effect of calcareous sediments gradation on their load carrying capacity is not fully researched.

CBR test is the most common test, which is used locally to assess the base and subbase geomaterials strength characteristics. However, its reliability is highly questionable, due to the high discrepancies observed between the field and the laboratory values. The researches made to study the deficiencies associated with the CBR test are not enough. The local concentration on the CBR test is attributed to its relative easiness and practicality in both the laboratory and the field. In addition, the Clegg Impact Hammer provides a quick and very easy tool to predict the CBR value of soils utilizing the correlations obtained between the two tests. Hence compared to the Modulus of resilience test, which is complex, time consuming and may give inaccurate results when back calculated (Mikhail, et al, 1999), the CBR test still can be valuable after improving its reliability. This can be achieved after clarifying the meaning of the CBR values greater than 100%, and by correlating the CBR value with another soil strength parameters. In addition, a practical laboratory CBR values must be obtained using modified testing procedure that gives better simulation for the filed conditions or by proposing a reliable correction methods to correct for the effect of the boundary conditions and preparation methods for the laboratory CBR results.

Chapter 3

Experimental Program

3.1 General

The nature and behavior of calcareous soils still need more investigation in order to understand their engineering performance and characteristics. Regardless of the efforts made recently to study these soils, more research work is urgently needed in order to build a database, which will provide a better understanding of these soils and will help also to avoid problems associated with their use. Despite the fact that calcareous soils are commonly used, in many parts of the world, as a construction material which expose them to various types of loading conditions, their load carrying capacity and the factors associated with it were not properly investigated.

This experimental work was devised to study the effect of gradation and testing procedures on the load carrying capacity of calcareous sediments. Marl soils from eastern Saudi Arabia were used for the testing program. The first part of the program comprised of characterizing the selected marls (two different samples) following the standard testing procedures stated in the ASTM and the AASHTO standards. Characterization tests were performed for both the aggregate fraction and the fine material. The second part of the experimental program was devised to study the effect of gradation and testing procedures on the moisture-density relationships and on the load carrying capacity of the collected marls. The tests used for the second part include the, Modified Proctor Test. California Bearing Ratio Test, Unconfined Compressive Strength Test and Clegg Hammer Test. The conventional CBR mold and a large (modified) mold were used for the comparison. The flowchart of the experimental program is shown in Fig. 3.1, and the experimental design is shown in tables 3.1 and 3.2, respectively.

3.2 Collection of Marl Samples

It is well known that the engineering properties of soils, such as strength, volume change and permeability are highly dependent on gradation as well as other parameters. During the process of compaction some of soil particles may crush. Calcareous soils, which contain soft carbonate aggregates, are more susceptible to crushing during compaction. This crushing of particles will alter the natural gradation; hence some engineering properties of the soil will change.

Two eastern Saudi marl samples were collected from different sources, which are still utilized for construction purposes within the region. The two marls were selected from different borrow areas in order to have some varieties in the parameters. Although the two marls are found to be eligible for use as construction materials, according to specifications stated by Dammam Municipality, there exist some quality differences between them. The first marl, which was collected from Abdullah A. Al-Dossary Company borrow area was found to have an abrasion value, according to the Los Angeles Abrasion Test (ASTM C131), in excess of 40%. However, the material obtained from

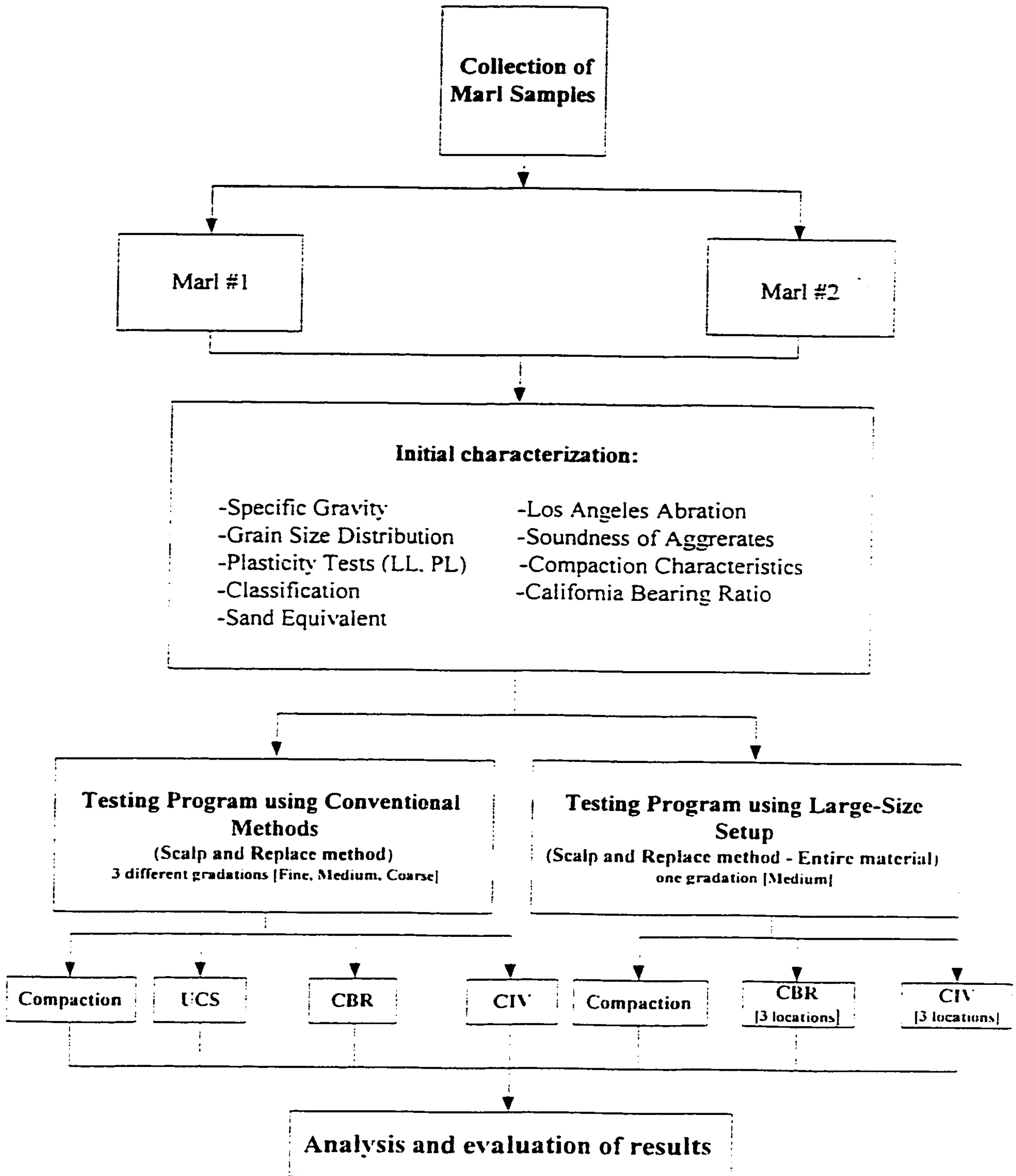


Figure 3.1: The flow diagram of the experimental program

Table 3.1: The Experimental design for the small-size mold

	Marl #1																	
	Fine Limit Gradation						Medium Gradation						Coarse Limit Gradation					
m.c%	2	3	4	5	6	7	2	3	4	5	6	7	2	3	4	5	6	7
γ_{dry}	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
CBR%	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
UCS gm/cm^3	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
CIV	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x

	Marl #2																	
	Fine Limit Gradation						Medium Gradation						Coarse Limit Gradation					
m.c%	2	3	4	5	6	7	2	3	4	5	6	7	2	3	4	5	6	7
γ_{dry}	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
CBR%	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
UCS gm/cm^3	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
CIV	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x

(x) represents results of the tests

Table 3.2: The experimental design for the small and large-size molds

		Marl #1										Marl #2									
		Scalp and Replace Method (medium gradation)					Entire Gradation (medium gradation)					Scalp and Replace Method (medium gradation)					Entire Gradation (medium gradation)				
	m.c%	3	4	5	6	7	3	4	5	6	7	3	4	5	6	7	3	4	5	6	7
Small-size mold	γ_{dry}	x	x	x	x	x						x	x	x	x	x					
	CBR%	x	x	x	x	x						x	x	x	x	x					
	CIV	x	x	x	x	x						x	x	x	x	x					
Large-size mold	γ_{dry}	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
	CBR%	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
	CIV	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x

(x) represents results of the tests

Al-Derbas Company borrow area was not exceeding 33%. Other parameters such as the natural gradation and the plasticity of fines were also different for the two marls. Thus, in the process of selecting marl soils for this research program, the aggregates susceptibility to crushing was intended to be one of the major variables. However, the existence of some differences in other parameters, like the plasticity and the grain size distribution contributed positively to the study.

3.3 Characterization of the Collected Samples

Generally, laboratory testing permits a greater degree of accuracy of measurements than field testing. This is attributed to the proper control of test conditions in the laboratory, which may be difficult to control in the field. However, the eligibility of a soil for use as a construction material is usually predicted using laboratory tests assuming satisfactory quality control procedures for the fieldwork.

Preliminary characterization tests were performed to assess the basic engineering properties of the two collected marl samples. These preliminary tests included plasticity tests and the grain size analysis. In addition, the characterization included also specific gravity, compaction, soaked and unsoaked California Bearing Ratio, Los Angeles abrasion, sand equivalent and soundness of aggregates tests.

3.3.1 Specific Gravity Test

The specific gravity of soil solids is often needed for various calculation purposes in soil mechanics. It is used in almost every equation that expresses the phase relationship of air, water and solids in a given volume of soil. It is used as a parameter in the determination of some important properties of soil such as void ratio, unit weight of soil and soil particle size analysis. Since both marl soils are composed of particles larger than and smaller than the No. 4 sieve, the samples were first separated on the No. 4 sieve, and then the appropriate test method was used for each portion. For the large particle sizes the specific gravity value is obtained following ASTM C127 method, and for material passing the No. 4 sieve, the ASTM D854 method was followed. The weighted average of the two values was reported as the specific gravity of the soil.

3.3.2 Grain Size Distribution

The grain size analysis was conducted for the two marls using both dry and washed sieving techniques (ASTM D422). First, in order to separate the gravel particles, the air-dried samples were passed downward through a stack of sieves with opening sizes 1½ in., 1 in., ¾ in., ½ in., 3/8 in. and 4.75 mm (ASTM No. 4 sieve). The amount of soil retained on each sieve was collected in separate bags and their masses were recorded. In order to assure a complete separation of fines, the material passing the No. 4 sieve was subjected to washed sieving. This was done by taking a representative soil sample then immersing it in a distilled water before being dispersed by a mechanically operated stirring device. The sample was then washed through a set of sieves including ASTM No. 10, 20, 30, 40, 60, 100, 140 and 200 sieves until the water passing through each sieve was clear. The material retained on each sieve was collected and oven dried before recording its mass.

3.3.3 Plasticity Tests

The liquid limit and plastic limit tests (ASTM D4318) can provide a quick means of determining the moisture contents limits from which the liquid and plastic phases will start for soils containing appreciable amounts of fine-grained particles. Both tests were conducted on material passing No. 40 sieve. For the first marl sample, which was collected from Al-Dossary borrow area, the liquid limit test was performed with some difficulty, this is attributed to the relatively low amount of clayey particles. The second marl, which was collected from Al-Derbas borrow area, showed no plasticity and its liquid limit was reported as "not defined".

3.3.4 Compaction Test

Laboratory compaction tests provide the basis for control procedures used in the field. They provide a relationship between dry density and moisture content for a given compaction method. Different compaction testing procedures are used depending on the grain size distribution of the soil sample. In this investigation the modified Proctor compaction test (ASTM D1557) was used. The soil retained on the ¾ in. (19 mm) sieve was excluded from the total sample and replaced by the same mass of soil passing the ¾ (19 mm) in. and retained on the No. 4 sieve. Each soil sample was reconstituted to its natural gradation to eliminate variations in the grain size and grain size distribution.

3.3.5 Soaked and Unsoaked CBR Tests

California Bearing Ratio (CBR) test is intended for determining the relative bearing value of soils and soil-aggregates when they are compacted to varying density values and moisture contents. It has been used in the structural design and evaluation of pavements. Locally in eastern Saudi Arabia, the CBR test is the major test in assessing the strength of base course construction materials before their use. Despite the fact that CBR test is an empirical test and still not well correlated with other soil parameters, it is recognized world wide because of its simplicity and applicability.

In the preliminary characterization, both soaked and unsoaked CBR tests (ASTM D1883) were performed. The soil was reconstituted to its natural gradation for each sample. The soil was compacted following the ASTM D1557 compaction method in the CBR mold. For the unsoaked sets, the samples were tested right after their compaction to avoid moisture losses. While for the soaked sets, the samples were immersed in water after being surcharged with 10 lb (4.54 kg) weights for 96 hours before testing.

3.3.6 Los Angeles Abrasion Test

This test is usually performed in order to assess the resistance of aggregate sizes smaller than 1½ in. to abrasion, using the Los Angeles testing machine. For marl samples, the test was conducted using the standard procedure stated in ASTM C131. The test samples were prepared according to gradation A and using 12 standard steel balls as the abrasive charge. The weight of the sample was recorded to the nearest 1 gm before testing, after being washed and oven dried at 110 C° to a substantially constant weight.

The sample was placed with the steel balls inside the hollow steel cylinder of the Los Angeles machine. After the prescribed number of revolutions (500 revolutions with a speed of about 31 rpm), the sample was discharged and then sieved using ASTM No. 12, sieve. The weight of the material retained on the No. 12 sieve was recorded. The difference between the original and the final dry weights of the material retained on the No. 12 sieve was expressed as a percentage of the original weight of the tested sample, and the value was reported as the percentage of wear.

3.3.7 Soundness of Aggregates Test

This test is intended to determine the resistance of soil aggregates to disintegration by saturated solution of sodium or magnesium sulphate. It helps to judge the soundness of aggregates subjected to difficult weathering conditions, particularly when there is no adequate information available from service records of the material exposed to actual weathering conditions.

The test was performed according to ASTM C88 standard procedure. The aggregates were prepared in three different sets according to their sizes. The first set included aggregates passing the 1½ in. sieve and retained on the ¾ in. sieve. The second set included aggregates passing the ¾ in. sieve and retained on the 3/8 in. sieve. The last set was prepared using aggregates passing the 3/8 in. sieve and retained on the No. 4 sieve. The masses of the three sets are approximately 1500, 1000 and 300 grams, respectively. Each of the three sets was immersed in a prepared saturated solution of sodium sulphate for a period of 16 hours. After which the samples were collected from the solutions and washed by distilled water. The samples were then oven dried to a condition of constant mass (12 hours was found to be appropriate to reach this condition). This procedure was repeated five times, and then the three sets of aggregates were passed through 5/8 in., 5/16 in. and No. 5 sieves respectively. The mass of the materials retained on each sieve was recorded. The difference between each of these amounts and the initial weight of the fraction of the sample tested is the loss in the test and was expressed as a percentage of the initial weight. The summation of the percentages of weight loss was reported as the percent loss by disintegration for the total sample.

3.3.8 Sand Equivalent Test

The sand equivalent test is considered to serve as a rapid field-correlation test (ASTM, 2000). The theme of this test is to indicate the relative proportions of the clayey or plastic fines in granular soils and fine aggregates that pass the No. 4 (4.75 mm) sieve.

This test (ASTM D2419) was performed, for the marl samples, by pouring a measured volume of the fine material into a graduated plastic cylinder containing a working calcium chloride solution. The specimen was then irrigated using additional amount of solution and then shook to loosen the clay like coatings from the sand particles. After a sedimentation period, the height of flocculated clay was read and the height of sand in the cylinder was determined. The sand equivalent is the ratio of the height of sand to the height of clay times 100. The average of two calculated sand equivalent values for two specimens was reported as the sand equivalent of the soil.

3.4 Testing Program using Conventional Methods

Soil gradation is believed to be one of the most important parameters, which govern the geotechnical properties of soils, especially their strength characteristics and volume change. The ASTM standards suggest different gradations for geomaterials for their use as bases and subbases for pavements. To study the effect of gradation on the load carrying capacity of marl soils, a series of tests were performed using different gradations. The tests used in this investigation are the CBR, Unconfined Compressive strength and Clegg Hammer tests.

3.4.1 Samples Preparation

In this investigation, three different gradations were selected for the experimental program. The first gradation is similar to the Dammam Municipality fine limit gradation and the second is its coarse limit. The third gradation is a median gradation, which lies between the fine and the coarse limits. The three gradations used are shown in Fig. 3.2.

Large quantity of each marl sample was sieved to separate different sizes. Each grain size was kept in bags, and stored inside the laboratory in order to achieve stable initial moisture content values. All samples were then stored in plastic bags to prevent moisture changes.

3.4.2 Compaction Test

Compaction tests were performed for each marl on four different sets. The first three sets were prepared with gradations following the fine, medium and coarse gradation mentioned previously. The compaction process for the three sets was conducted following the scalp and replace method (ASTM D1557, 1991) to correct for the elimination of oversize particles (particles retained on the ¾ in. (19 mm) sieve). The forth set was prepared with the medium gradation, using the elimination method. In such method, the material retained on the ¾ in. (19 mm) sieve was excluded and the test was performed on material passing the ¾ in. (19 mm) sieve without replacement. All compaction tests were performed by securing approximately 1% increase of moisture between each successive test specimens. These slight increases of moisture were made because of the high sensitivity of marl to molding moisture content. A prescribed amount of each grain size was weighed and the different sizes were mixed to reconstitute the samples to the selected

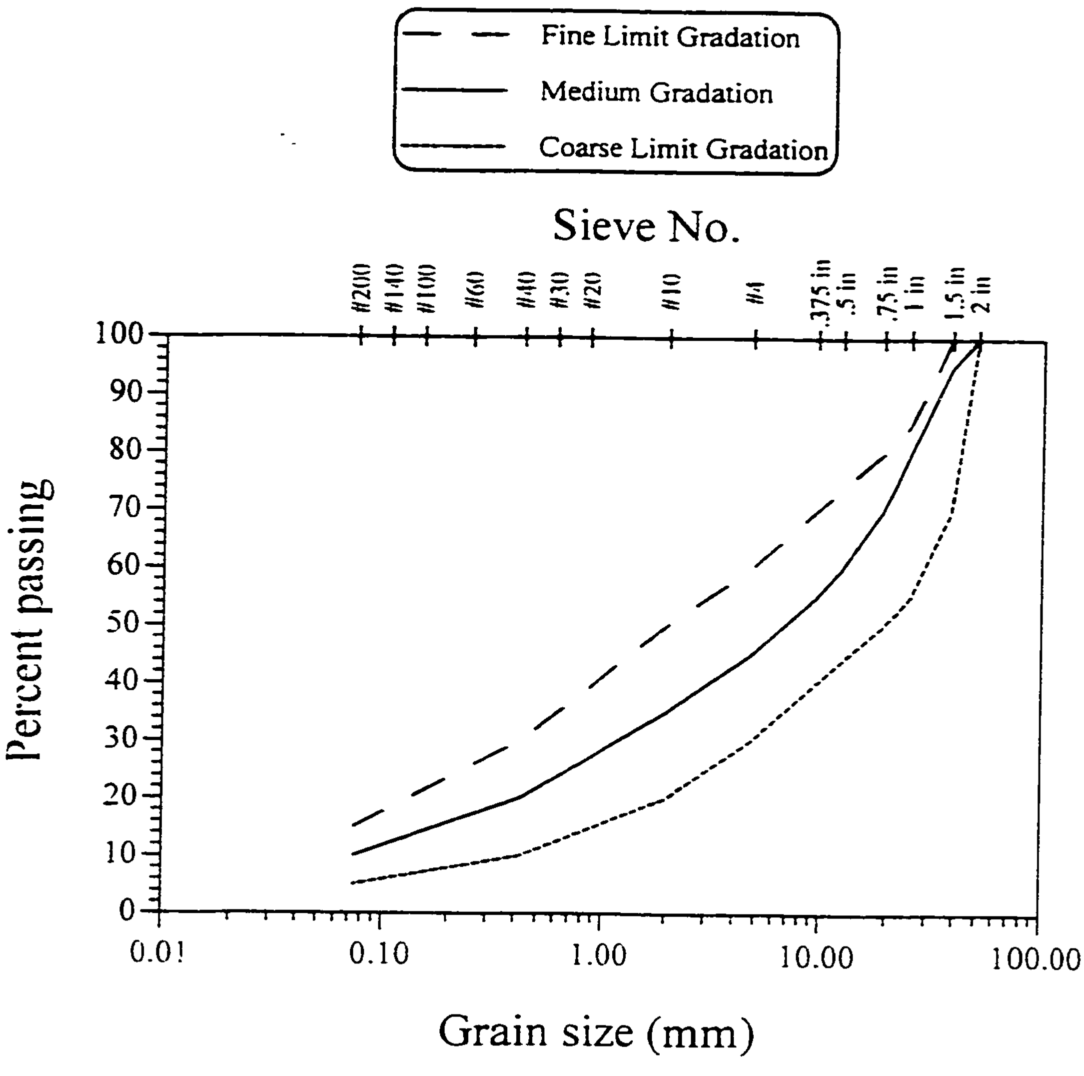


Figure 3.2: The selected gradations for the experimental program

gradations. For each marl, the compaction test for the medium gradation set was repeated in order to study the repeatability of the results. A representative amount of the prepared sample was taken in order to obtain the moisture content value for each specimen. It was observed that, while drying marl samples in the oven, the state of constant mass (full dryness) was usually achieved within a period of one to two days. Therefore, all tested samples were left to dry in the oven for at least 48 hours to assure full dryness.

3.4.3 Unsoaked CBR Test

Unsoaked CBR tests were performed on samples reconstituted to the three gradations for each marl. It was decided to perform the test without soaking the samples, in order to obtain similar testing conditions with the other tests, such as the unconfined compression test, for further correlations. In addition, the use of the large mold was intended to simulate the field conditions, regarding the size of the sample and the fact that the field tests are usually performed without soaking. All tests were conducted in accordance with ASTM D1883 for specimens compacted following the ASTM D1557 procedure. In addition, for each marl, a set of specimens was prepared using the medium gradation and the elimination method. For all sets, samples were tested immediately after the compaction process to avoid moisture losses. The consistency of the testing conditions and its effect on obtaining reproducible results was investigated by repeating the CBR test on samples prepared according to the medium gradation for each marl.

3.4.4 Unconfined Compressive Strength Test

The unconfined compression test is used to determine the undrained strength and stress-strain characteristics of undisturbed, remolded and/or compacted soil samples. This test is applicable for soils possessing some cohesion, where the sample is not allowed to expel water during loading. The soil sample must retain intrinsic strength after removing the confinement, which is provided by the mold walls for compacted samples or by the confinement of the surrounding soil for undisturbed samples. In the structural design of pavements, the unconfined compressive strength (UCS) is used as a strength criterion for base and subbase layers. In common practice, the pavement design standards specify a certain minimum UCS value for different layers.

In this research program, the unconfined compressive strength test was performed after compacting the marl samples in a steel cylindrical mold with a height of 8 in. (203 mm) and a diameter of 4 in. (102 mm), which gives a height to diameter (h/d) ratio of 2 for the three gradations. The specimen size was preferred for better correlation with the compaction test in the CBR mold and it permits inclusion of large particle sizes. The mold used is a split type. The use of a split mold was intended to avoid any change in density while extruding the sample. According to ASTM D2166, the largest particle size to be included should be smaller than one-sixth of the specimen diameter. Particles up to ¾ in. (19 mm) can be included in test specimens of 4 in. (102 mm) diameter (Alkhafaji and Andersland, 1992).

To achieve an appropriate similarity between the modified Proctor compaction method, and the compaction in the USC mold, the samples were compacted in the UCS mold in five layers, and the compactive effort was applied by dropping a 10 lb (4.54 kg) hammer from a height of 18 in. (457 mm). It was observed that, in the trial specimens, compacting a sample, with a certain gradation, in the UCS mold usually gives higher maximum dry density than compacting it in the standard CBR mold. This difference in densities was observed even when the modified Proctor hammer weight, drop height and number of blows were used for both types. The difference is attributed to the increase of the compactive effort in the case of the UCS mold because of the lower volume of the sample compared to that in the CBR mold. Hence, in order to achieve approximately equal dry densities, a series of samples were compacted in the UCS mold with different number of blows, keeping the other parameters unchanged. It was found that, for the UCS mold, 32 blows on each layer was adequate to achieve an acceptable similarity in the dry densities with those obtained using the CBR mold. The methodology used to obtain the appropriate number of blows is explained in details in the results and discussion chapter.

Samples were trimmed after compaction and their surfaces were cut carefully to prevent excessive irregularities and to secure their perpendicularity to the longitudinal axis of the specimens. Each sample was tested immediately after its extrusion from the mold and handled carefully in order to prevent disturbance, changes of cross section or loss of moisture. Two specimens were tested at each moisture content.

3.4.5 Clegg Hammer Test

Despite the fact that the CBR test is internationally adopted, it is both tedious and time consuming specially in the field where it requires a lot of preparations and need different types of equipment. Clegg Impact Hammer shown in Fig. 3.3 was utilized to provide a simple technique that can correlate well with the CBR test. It is easy to operate and provides a cost effective quality control method for the quality of field compaction (Khan

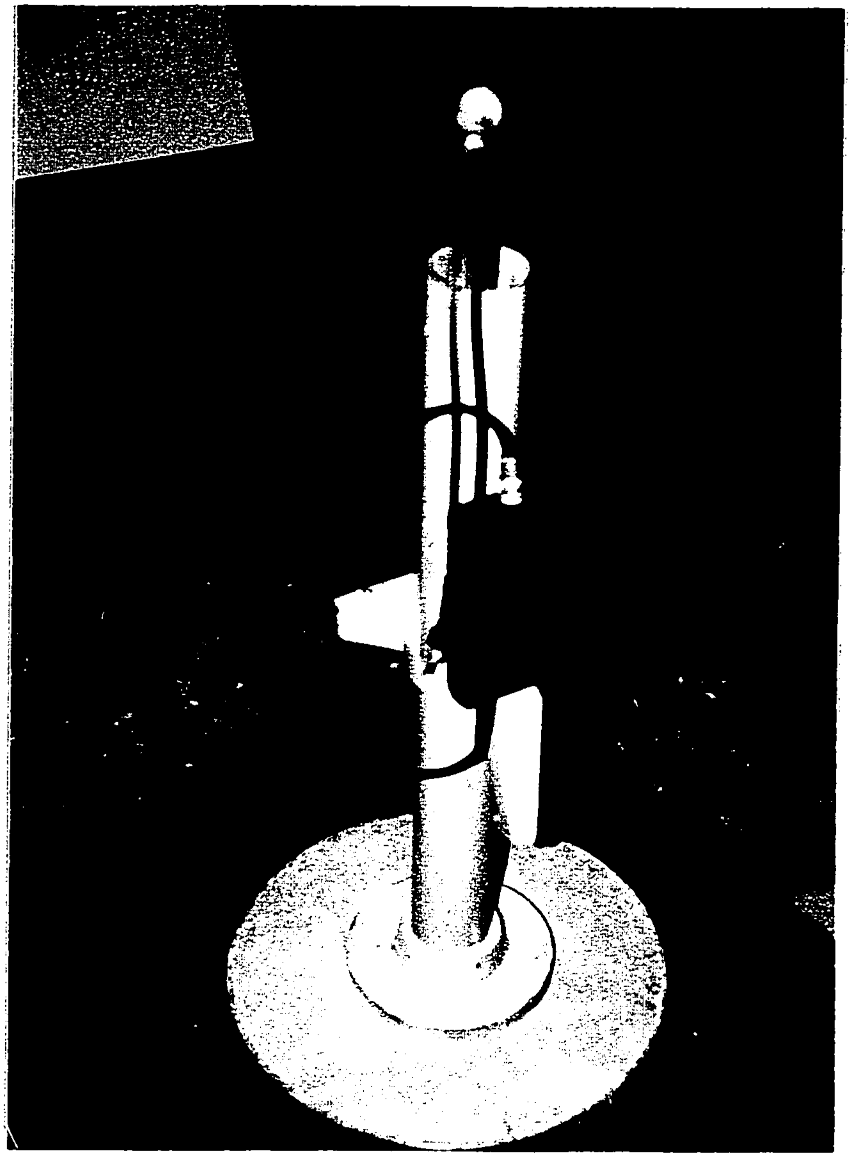


Figure 3.3: The Clegg Impact Hammer.

et al, 1995). This device measures the "Dynamic Impact Value", which is a measure of the strength of the soil layer.

The apparatus consists of a falling part with the same shape and size of the modified Proctor rammer. It is equipped with a piezoelectric accelerometer, which connected to a digital measuring device. The dynamic rebound of the soil against the falling weight of the hammer is recorded as the Clegg Impact Value (CIV) of the specimen (Asi et al. 1992).

The test was performed on marl samples right after the CBR test was finished. The specimen was inverted so that the surface of the specimen tested by the Clegg Impact Hammer was not the one used for the CBR test. The specimen was supported by steel spacer disk, which was placed beneath it. The Clegg hammer was then placed on top of the specimen and the hammer was raised to 45 cm height and released to fall free on the sample. This procedure was repeated four times plus one additional blow to insure CIV stabilized reading. This stabilized reading was recorded as the CIV value of the sample. The Clegg hammer used in this research was manufactured by "Controls Milano Italy (model T 168-005)".

3.5 Testing Program using Large-Size Setup

Marl soils, when they are mechanically modified for base course construction are usually classified as gravelly soils. Such soils are believed to have questionable laboratory test results as a result of discarding large size particles, which always have influential percentages. Hence, the use of large size testing specimens may provide a suitable remedy for such testing problems.

In order to carry out this experimental research program, a new compaction and CBR testing procedure was used. This was achieved by modifying the traditional compaction machine and CBR mold in order to perform the tests using large scale CBR mold such that particles up to 2 in. (51 mm) can be accommodated. The setup consists of 12.5 in. (317.5 mm) diameter mold with a height of 8.1 in. (206 mm) and a collar 2.25 in. (57.2 mm) high. While developing the new method many features of the conventional procedure were maintained. The modified compaction methodology was used throughout the testing program and a compactive effort approximately equivalent to the modified Proctor was maintained. This was achieved using a sector shaped hammer and a drop height of 18 in. (457 mm). The mold volume is 0.57 ft^3 (16258 cm^3). The system was calibrated many times in order to achieve the same results produced by the conventional method for the same soil gradation and water content. The details of the proposed mold and accessories are shown in Fig. 3.4

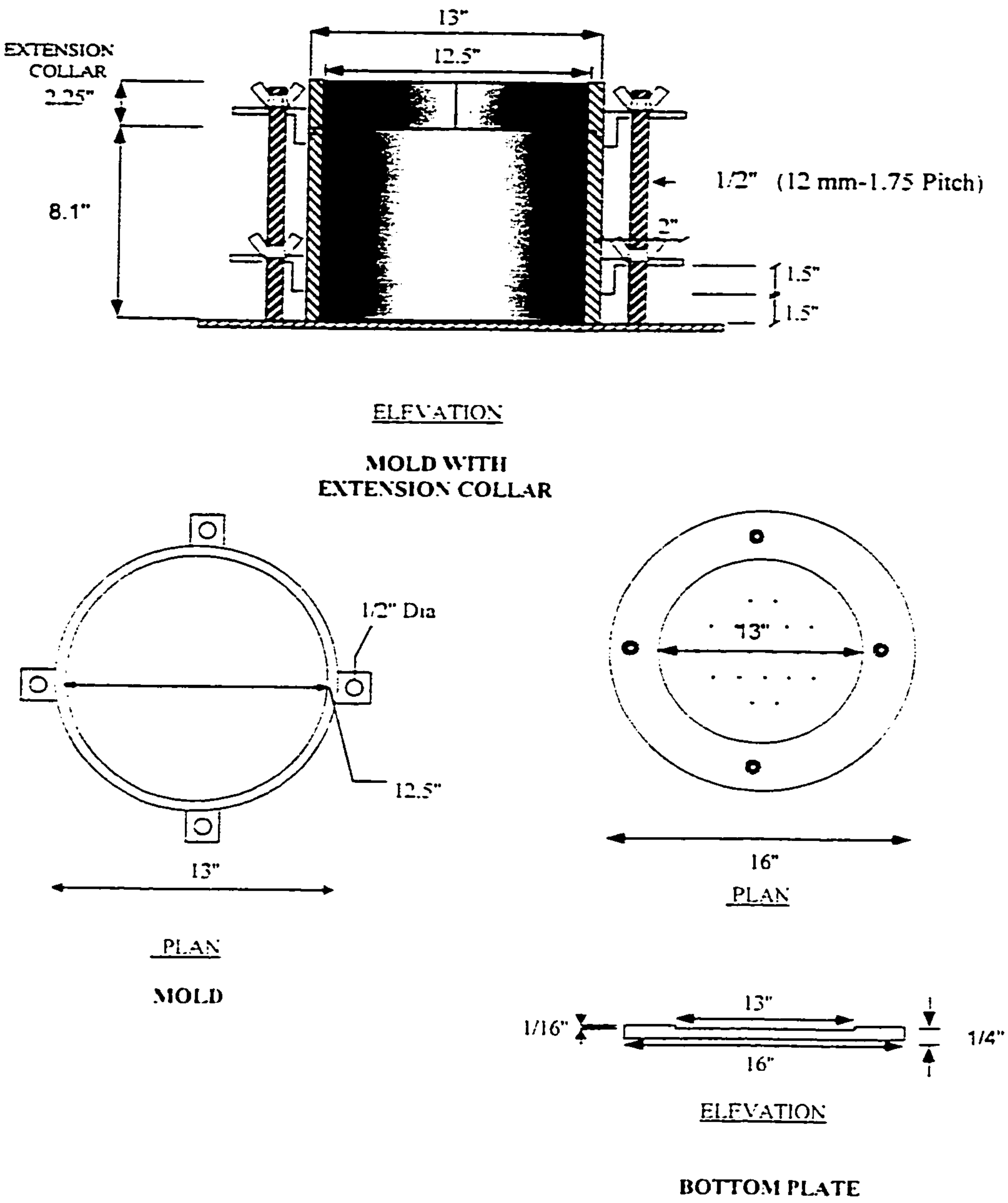


Figure 3.4: The modified compaction and bearing ratio test mold

3.5.1 Experimental Setup

Due to the differences between the two molds, it was necessary to optimize the compaction procedure for the large mold to simulate the standard small mold. This was done by trial and error procedure whereby the hammer weight, drop height and number of blows were the main variables. In the first trial, a 19 lb (8.5 kg) mass hammer was used with a drop height of 18 in. (457 mm) and 60 blows per layer, in order to calibrate the large size setup. The target density was 2.26 gm/cm^3, which is the maximum dry density, obtained using the standard CBR mold for the same gradation for Marl #1. The soil was prepared at the optimum moisture content and then compacted using the abovementioned setup. Initially a dry density of 2.18 gm/cm^3 was produced. Hence, the setup was modified in order to increase the compactive effort. The hammer mass was increased to 31.4 lb (14 kg) and the number of blows was increased to 80 blows maintaining the same drop height and hammer shape. The modified setup produced a dry density of 2.21 gm/cm^3, which was still less than the target density. The third trial was performed using 47.3 lb (21 kg) mass hammer and 100 blows. This setup produced a dry density of 2.26 gm/cm^3, which is basically the target value. Hence, the large size compaction setup adopted throughout this experimental program utilized the standard compaction machine with a 47.3 lb (21 kg) hammer; 18 in. (457 mm) drop height and 100 blows per layer. The sample was rotated manually, during compaction in order to distribute the compactive effort over the whole surface.

3.5.2 Compaction Test

Two sets of samples were prepared using the medium gradation and two preparation methods. The first set was prepared using the scalp and replace procedure; by excluding the particles retained on the ¾ in. (19 mm) sieve and replace them with the same mass of material passing the ¾ in. sieve and retained on the No. 4 sieve. The second set was prepared by reconstituting the soil to the entire gradation curve for medium gradation, without discarding the oversize particles. Particles up to about 2 in. in size were used while preparing the second set. All samples were prepared in five batches to assure the uniformity of gradation through the test specimens. The molding water was added in order to attain about 1% increase of moisture between successive specimens. Samples were handled carefully and tested immediately after compaction. The modified compaction and bearing ratio mold and accessories compared to the standard ones are shown in Fig. 3.5.

3.5.3 Unsoaked CBR Test

The CBR test was performed on the large size compacted specimens, where samples were compacted using both the entire soil gradation and using replacement method. After removing the collar, the sample surface was trimmed and the sample was then placed on the testing machine. The sample was inverted and a surcharge of 0.36 psi (2.48 kN/m^2) was cast in order to simulate the standard surcharge of the conventional small mold. The penetration was performed using the standard plunger, which is the typical routine used in the standard CBR test. However, the test was performed on three different locations. The first location lies on the center of the specimen surface and the other two were selected to lie half distance between the center and the wall of the mold. The off center points were

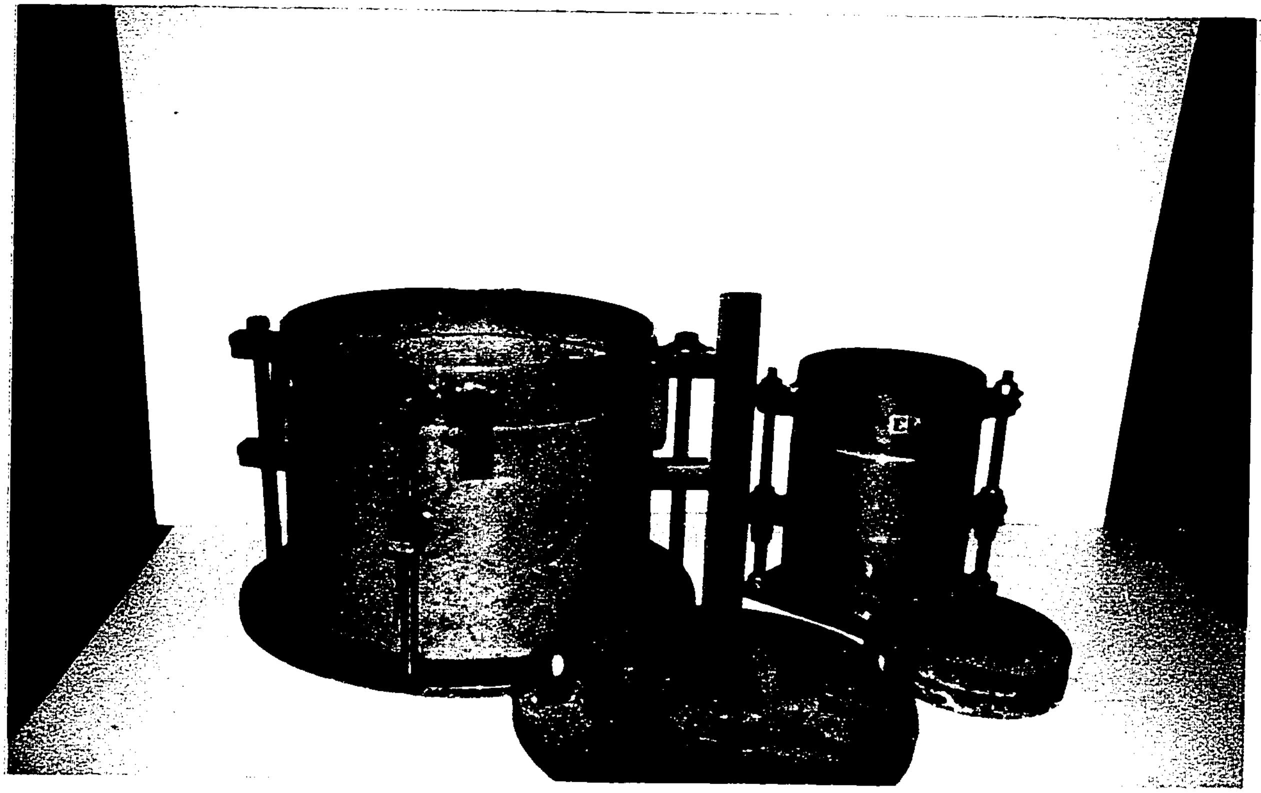


Figure 3.5: The modified and standard compaction and bearing ratio molds and accessories

tested in order to study the effect of confinement on the CBR values. Representative amount of each specimen was taken after the test to obtain the moisture content and to determine the moisture losses during testing. The CBR testing setup for the large-size mold is shown in Fig. 3.6.

3.5.4 Clegg Hammer Test

The Clegg impact value (CIV) was obtained for all specimens, which were prepared for the CBR test. The CIV test was performed on the undamaged surface. The test was performed on three points, the first point lie on the center and the other two points lie midway between the center and the mold wall, on two opposing sides. This was done to study the effect of the wall on the CIV values. The Clegg hammer test was performed on samples prepared using both the entire soil gradation and using the scalp and replace method. The Clegg hammer was dropped 5 times where the fifth drop was made to insure the stability of the reading. The final stabilized reading was taken as the CIV value of the test specimen.

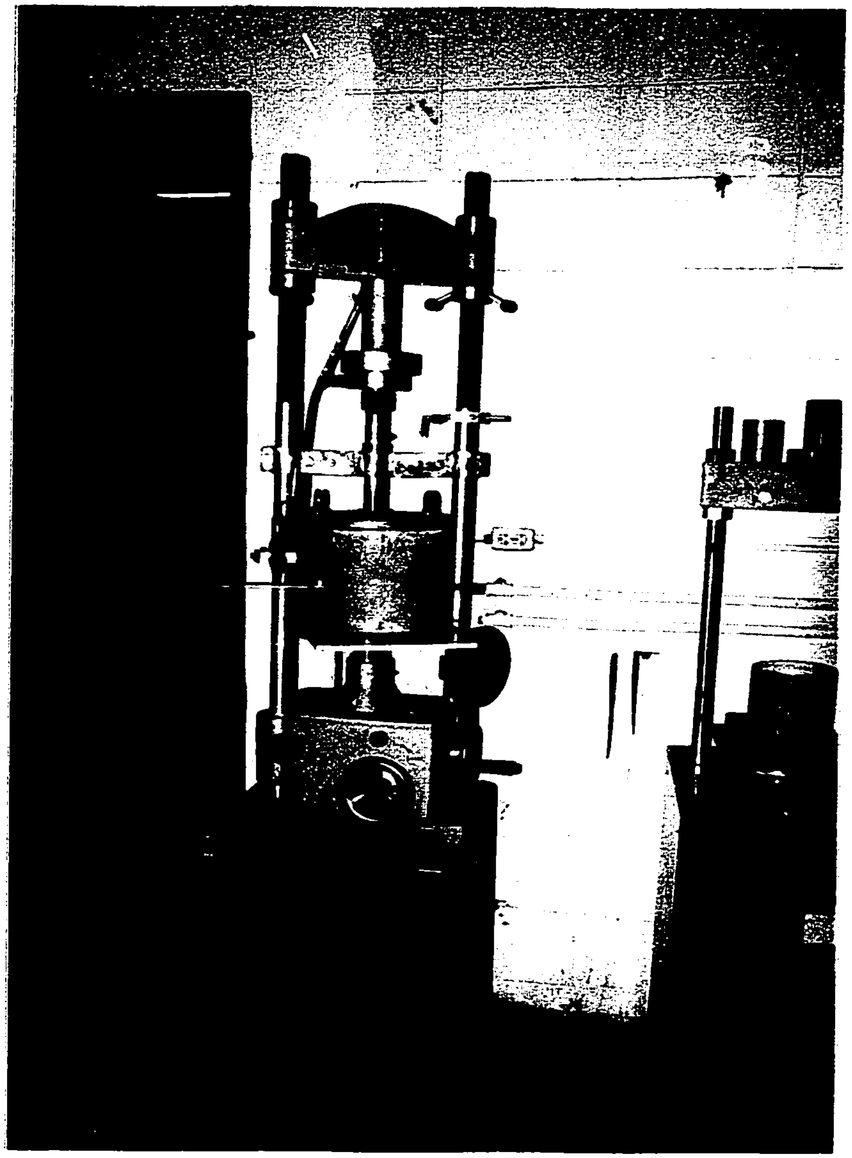


Figure 3.6: The CBR testing setup for the large-size mold.

Chapter 4

Results and Discussion

4.1 Characterization of the Collected Marl Samples

The two marl samples used in this research program were collected from local borrow areas in eastern Saudi Arabia, which are still used as sources of construction geomaterials. Both samples were found to be eligible for use as graded base course material according to the Dammam Municipality specifications. The main differences between the two collected samples were the crushing strength of aggregate and plasticity of fines.

Collected marl samples were subjected to a preliminary testing program to explore their general properties. The characterization-testing program was intended to characterize both coarse and fine fractions. The properties investigated are the specific gravity (for both fine and coarse fractions), grain size distribution, plasticity, moisture-density relation, California Bearing Ratio (soaked and unsoaked), Los Angeles Abrasion, Sand equivalent and soundness of aggregates.

The samples were classified using the standard classification methods stated by the American Association for State Highway and Transportation Officials (AASHTO) and the Unified Soil Classification System (USCS). Marl #1 was classified as GM according to USCS system and as A-1-a according to AASHTO system. However, Marl #2 was classified as SM according to USCS system and A-1-b according to AASHTO system.

Comparing the two marls, Marl #1 shows higher average specific gravity, plasticity index, maximum dry density, unsoaked CBR value and percent wear (using Los Angeles Abrasion Machine). However, Marl #2 shows higher maximum soaked CBR value, sand equivalent and percent loss (by disintegration using sodium sulphate solution). The classification of the two samples and some of their characteristics parameters are shown in Table 4.1.

The natural gradation curves for both samples are shown in Fig. 4.1. Generally Marl #2 shows finer gradation when compared with Marl #1. The AASHTO classification system classified Marl #1 as gravel and Marl #2 as sand. Although the percent passing the ASTM No 200 sieve were almost similar for both marls, Marl #1 showed a plastic behavior, which was absent in Marl #2. It is observed that the percent loss by weight was higher in Marl #1, compared to Marl #2, when using a mechanical abrasive agent (steel balls in the Los Angeles Abrasion Test) but it was lower when using a chemical disintegrating agent (sodium sulphate in the Soundness Test). This indicates that although the aggregate of Marl #2 are harder than the aggregate of Marl #1, they have more solubility in the chemical environment.

Table 4.1: Classification and characteristic properties of the collected marls

Property	Designation	Marl #1	Marl #2
Classification	USCS	GM	SM
	AASHTO	A-1-a	A-1-b
Specific Gravity (for fine fraction)	ASTM D 854	2.71	2.71
Specific Gravity (for coarse fraction)	ASTM D 127	2.46	2.47
Weighted Average Specific Gravity		2.61	2.54
Liquid Limit	ASTM D 4318	18.1	NP*
Plastic Limit	ASTM D 4318	14.2	Non-Plastic
Plasticity Index		3.9	Non-Plastic
Maximum Dry Density	ASTM D 1557	2.26	2.23
Optimum Moisture Content	ASTM D 1557	5.5	5.8
Maximum Soaked CBR	ASTM D 1883	138	243
Maximum Unsoaked CBR	ASTM D 1883	278	258
Percentage Wear	ASTM C 131	41	33
Percentage Weight Loss	ASTM C 88	4	8
Sand Equivalent	ASTM D 2419	15	27

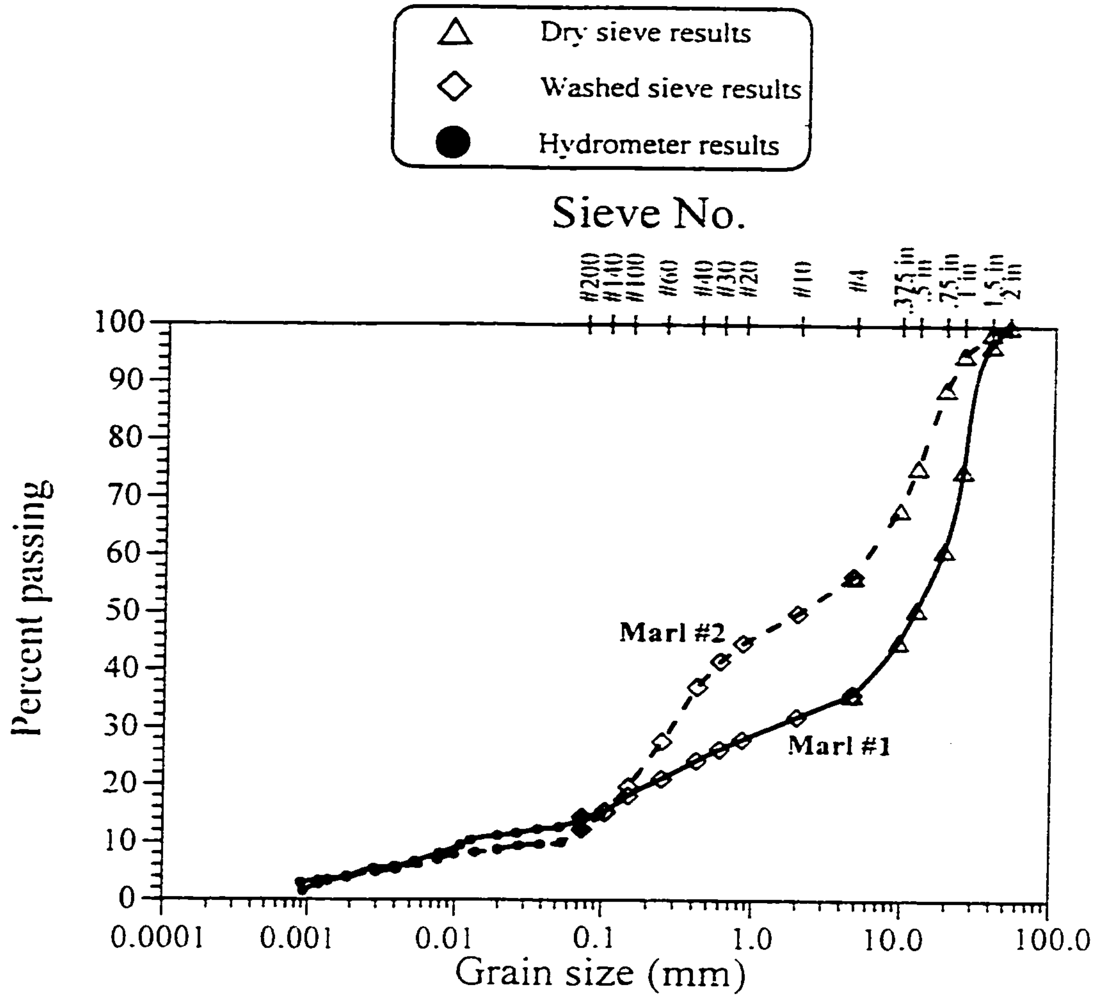


Figure 4.1: Grain-size distribution of the collected marl samples

4.1.1 Moisture-Density Relationships

The moisture-density relationships for the marl samples were determined using the modified Proctor method. The samples were reconstituted to their natural gradation and compacted in the standard CBR mold. The compaction curves for both marls are shown in Fig. 4.2. Marl #1 was found to have a maximum dry density of 2.26 g/cm^3 and optimum moisture content of 5.5% while Marl #2 has a maximum dry density of 2.23 g/cm^3 and optimum moisture content of 5.8%. It is observed that Marl #1 attained its maximum dry density at lower optimum moisture content, which was caused by the coarser gradation of Marl #1, and hence its lower specific surface and lower affinity for water compared to Marl #2. As shown in the figure, Marl #1 shows higher dry density values compared to Marl #2. This is attributed to the fact that Marl #1 is coarser than Marl #2 and to the higher susceptibility to grain crushing of Marl #1 during compaction, (higher percent of wear as shown in Table 4.1) compared to Marl #2. This will modify the original gradation and helps the soil to have denser matrix. It is also noticed that the density values on the wet side of optimum, for both marls started to approach each other

4.1.2 Soaked and Unsoaked CBR Tests

California Bearing Ratio tests (both soaked and unsoaked) were performed in order to characterize the bearing capacity of the collected marls. The CBR-moisture-density relationships of the two samples are shown in Figs. 4.3 and 4.4. The maximum CBR values for Marl #1 were achieved on the dry side of optimum water content, as shown in Fig. 4.3. It is also shown that relatively low CBR values were associated with the

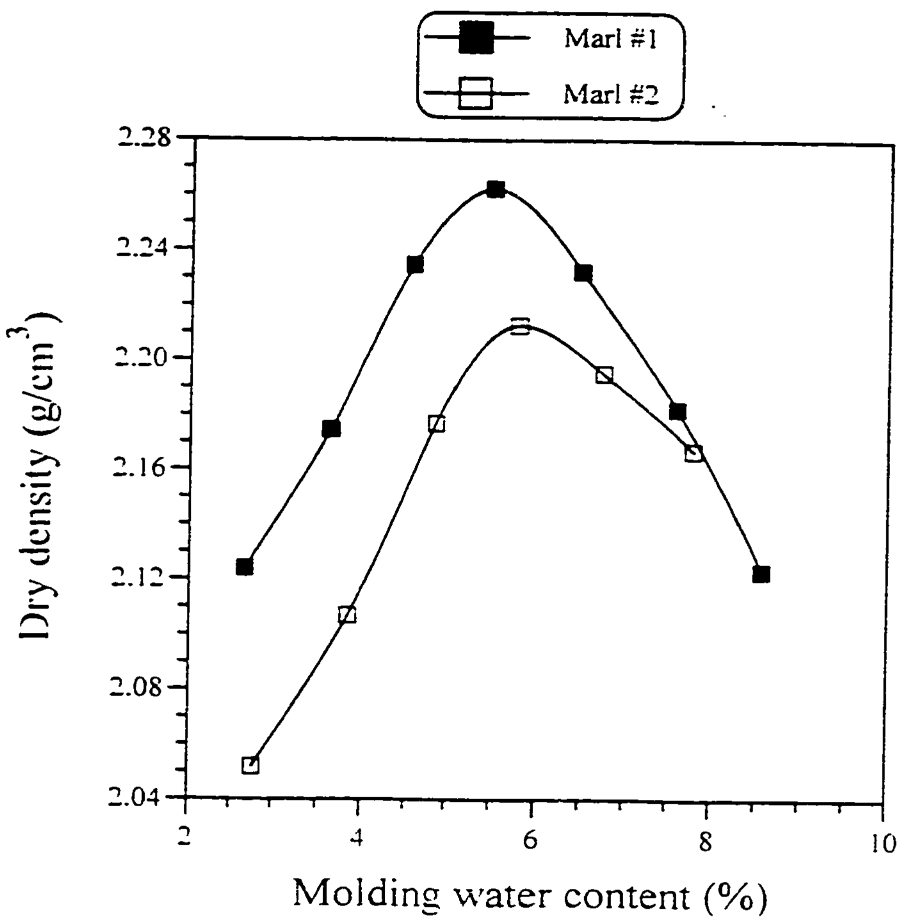


Figure 4.2: The moisture-density relationships for the collected marl samples

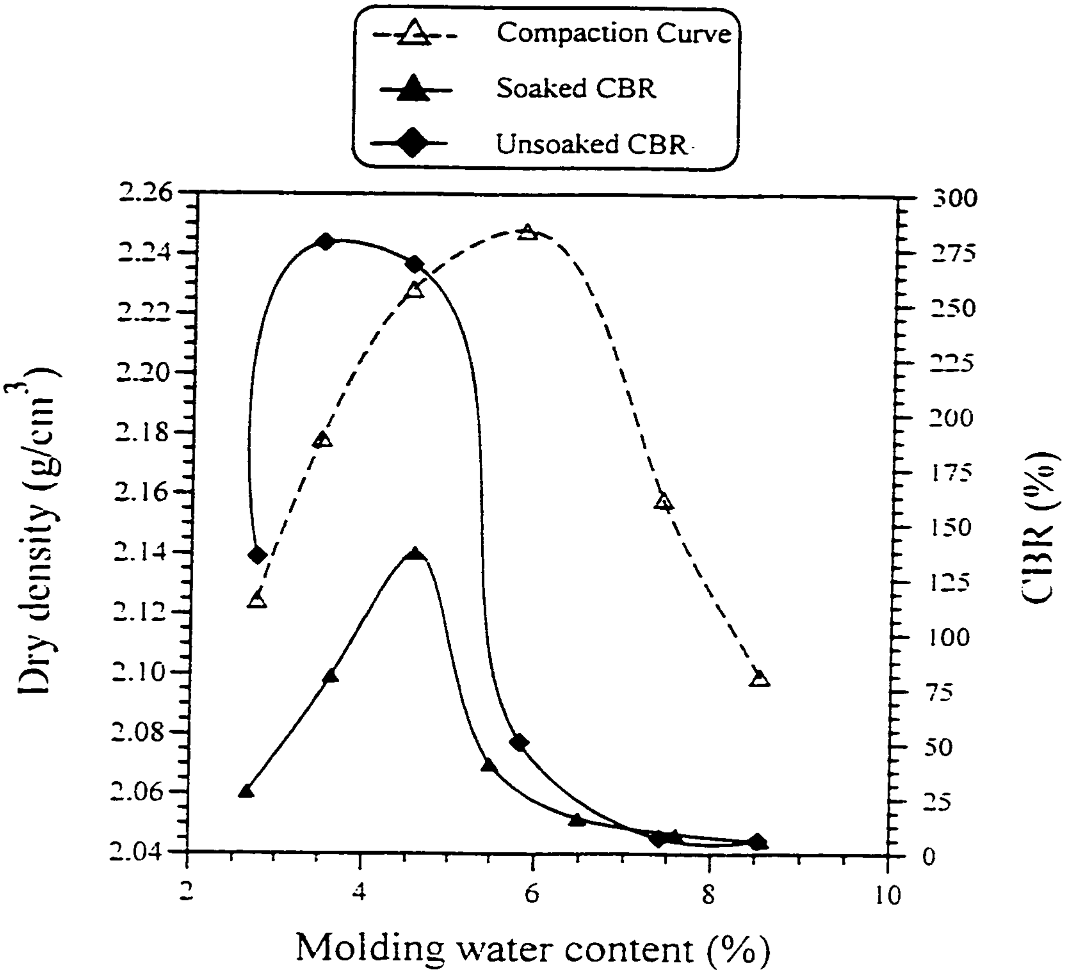


Figure 4.3: Soaked and Unsoaked CBR-moisture-density relationships for Marl #1

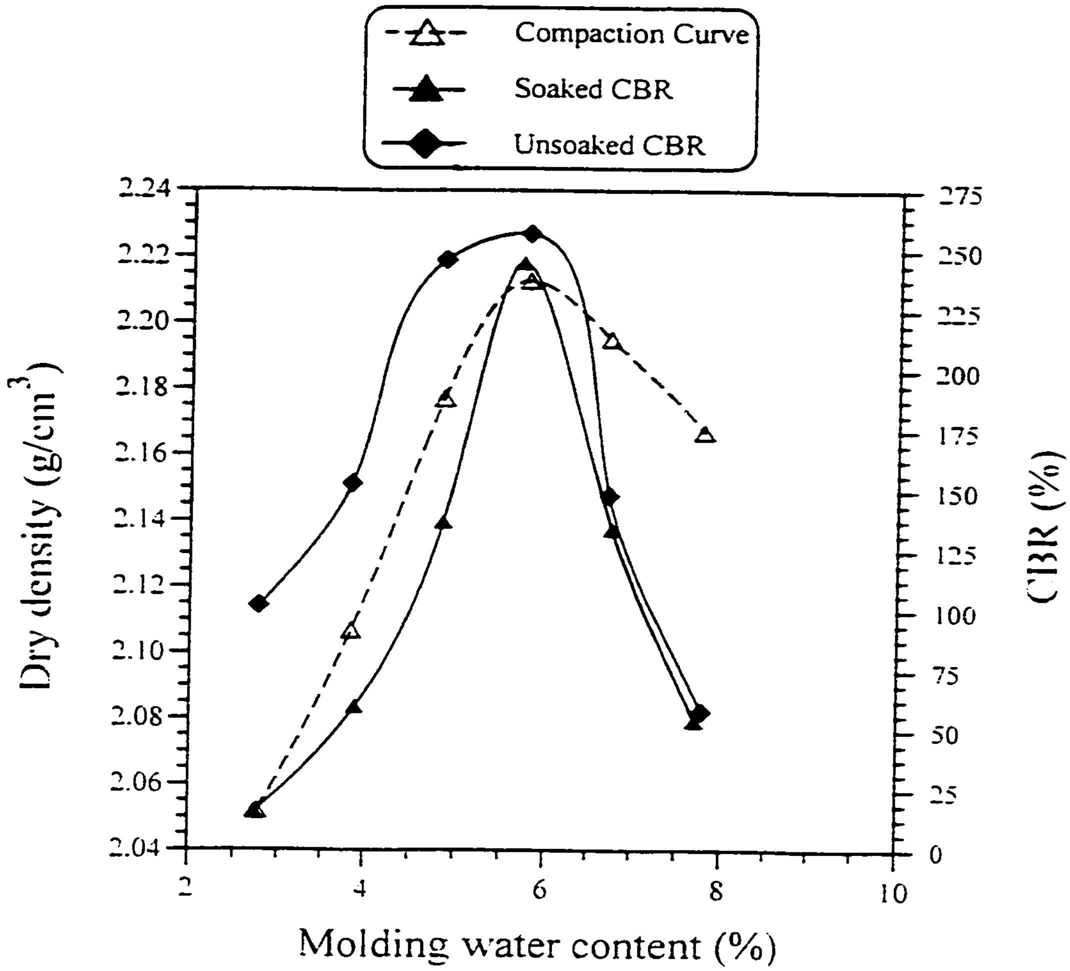


Figure 4.4: Soaked and Unsoaked CBR-moisture-density relationships for Marl #2

optimum water content for both soaked and unsoaked samples. This indicates that, compacting the soil at its maximum dry density is not a sufficient condition to achieve the maximum (or at least high) CBR value. However, for Marl #2, the maximum soaked and unsoaked CBR values were obtained at almost the optimum water content.

Comparing the two marl samples, the maximum CBR values in marl #1 were achieved at lower water contents than that of Marl #2, which substantiate the lower affinity of Marl #1 for water. The maximum CBR values for the two samples exceeded 100% for soaked and unsoaked sets. Despite the fact that there is no technical meaning for these high values, marl samples usually present such values in laboratory tests, especially on the dry side of the optimum water contents. This is attributed to the high percentages of stone fraction in marl soils. For both marls, it is clear that sharp peaks are seen in the CBR curves for the soaked specimens, while a relatively flat peaks are observed for the unsoaked samples. Thus, soaking of marl samples increases their water sensitivity, which may be caused by the weakness of some of the coarse particles due to softening and reduction of the cementing effects between the particles as a result of immersing the specimens in water.

Generally, at the same molding water content, the unsoaked CBR values are higher than the soaked CBR values as shown in Figs. 4.3 and 4.4. The sharper decrease in the CBR values as a result of soaking is observed in marl #1, compared to Marl #2, which indicates higher effects of soaking due to the plasticity of the fine fraction, which led to the loosening of the connectors between soil particles. It is also observed that, for the two marls, the soaked and unsoaked CBR curves tend to merge on the wet side of optimum and they ended up to be very close to each other at higher moisture content values. On the wet side, most of the pores are filled with water and this water will contribute in taking some of the applied stress. Hence, the bearing values of both soaked and unsoaked sets will be approximately similar, at higher moisture contents, due to the excess pore water pressure, which reduces the effective strength due to the reduction in the effective stress ($\sigma' = \sigma - u$). In addition, the water is weakening the connectors, especially when they are plastic, as well as the coarse particles.

Comparison of the soaked CBR values for the two marls is shown in Fig. 4.5. The results clearly show that Marl #2 has a higher maximum soaked CBR value compared to Marl #1. However, for the unsoaked sets Marl #1 has higher maximum CBR value as shown in Fig. 4.6. This again substantiates the acute sensitivity of marl #1 upon soaking. This sensitivity is attributed to the plastic nature of fines and the higher grain crushability of Marl #1. Generally, Marl #1 shows sharp reduction in the bearing values with the presence of excess water i.e. on the wet side of optimum. This phenomenon is clear while observing that for both soaked and unsoaked samples Marl #1 shows higher CBR values, compared to Marl #2, on the dry sides of optimum, but much lower values on the wet sides of optimum moisture content. The reduction of the bearing capacity of Marl #1 after soaking is attributed to its plastic behavior. According to Aiban (1995) cohesive marl samples show a remarkable reduction in cohesion values with the increase in the molding water contents.

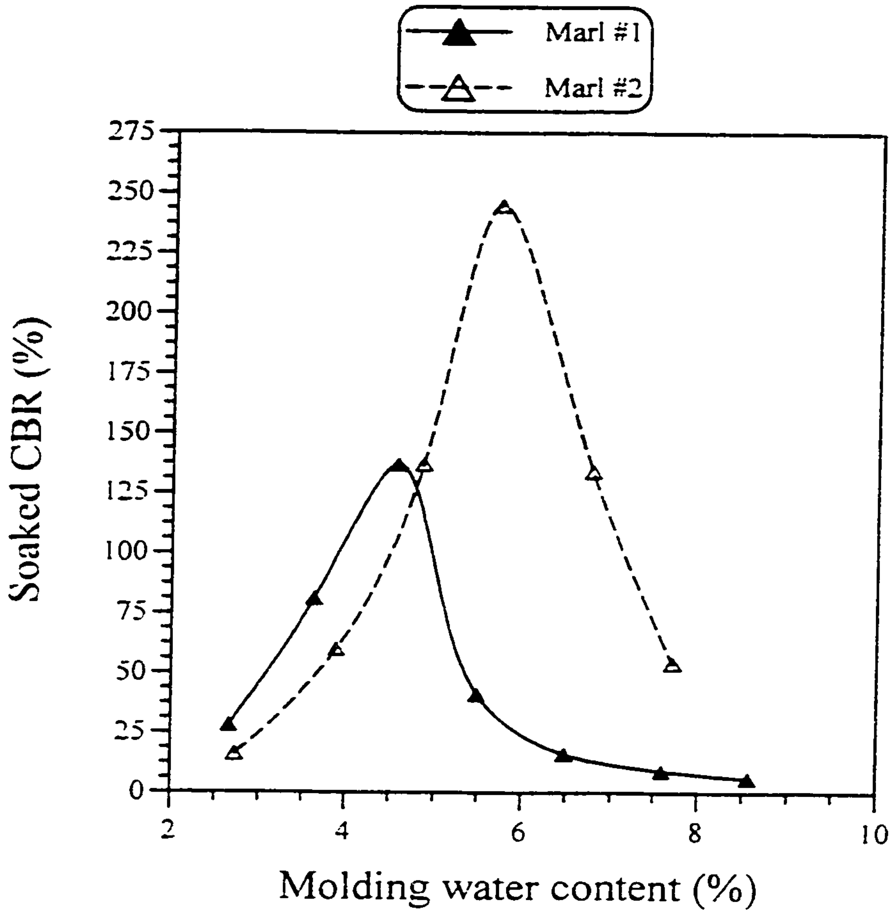


Figure 4.5: Soaked CBR-moisture relationships for the collected marl samples

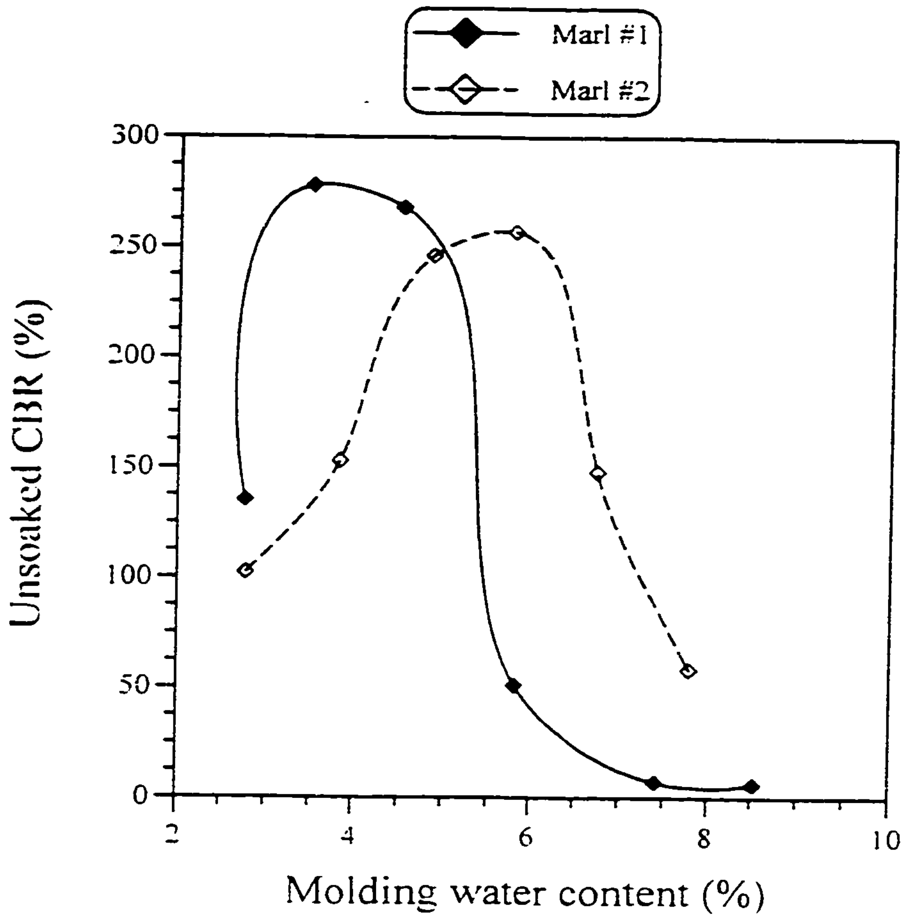


Figure 4.6: Unsoaked CBR-moisture relationships for the collected marl samples

4.2 Testing Program using Conventional Methods

In order to study the effect of gradation and testing procedures on the load carrying capacity of marl soils, conventional tests were performed using the standard methods. These tests include the California Bearing Ratio, Unconfined Compressive Strength and Clegg Impact Hammer. The conditions were intended to be similar for all tests in order to eliminate the preparation and material variables and enable comparison or correlation for other parameters. Although the density may not be a sufficient condition for similarity of testing conditions, the three types of tests were performed on specimens under approximately similar densities and molding moisture content. Other parameters were also maintained the same and only one parameter was considered at a time. The parameters included soil gradation (three different soil gradations were used throughout this testing program), compactive effort and molding water content.

To study the effect of repeating the CBR and compaction tests on the obtained results, testing of samples reconstituted to the medium gradation was repeated for the two marls. The results are shown in Figs.4.7 and 4.8 for Marl #1 and Marl #2, respectively. For the first marl, the maximum difference between the dry density values was found to be 0.013 gm/cm^3, which represents 0.6% of the average value. While the maximum difference between the CBR values was 19%, which represents 7.4% of the average value, for samples prepared at approximately equal moisture contents. For the second marl, the maximum difference between the dry densities, for samples prepared at approximately equal moisture contents, was found to be 0.013 gm/cm^3, which represents 0.6% of the

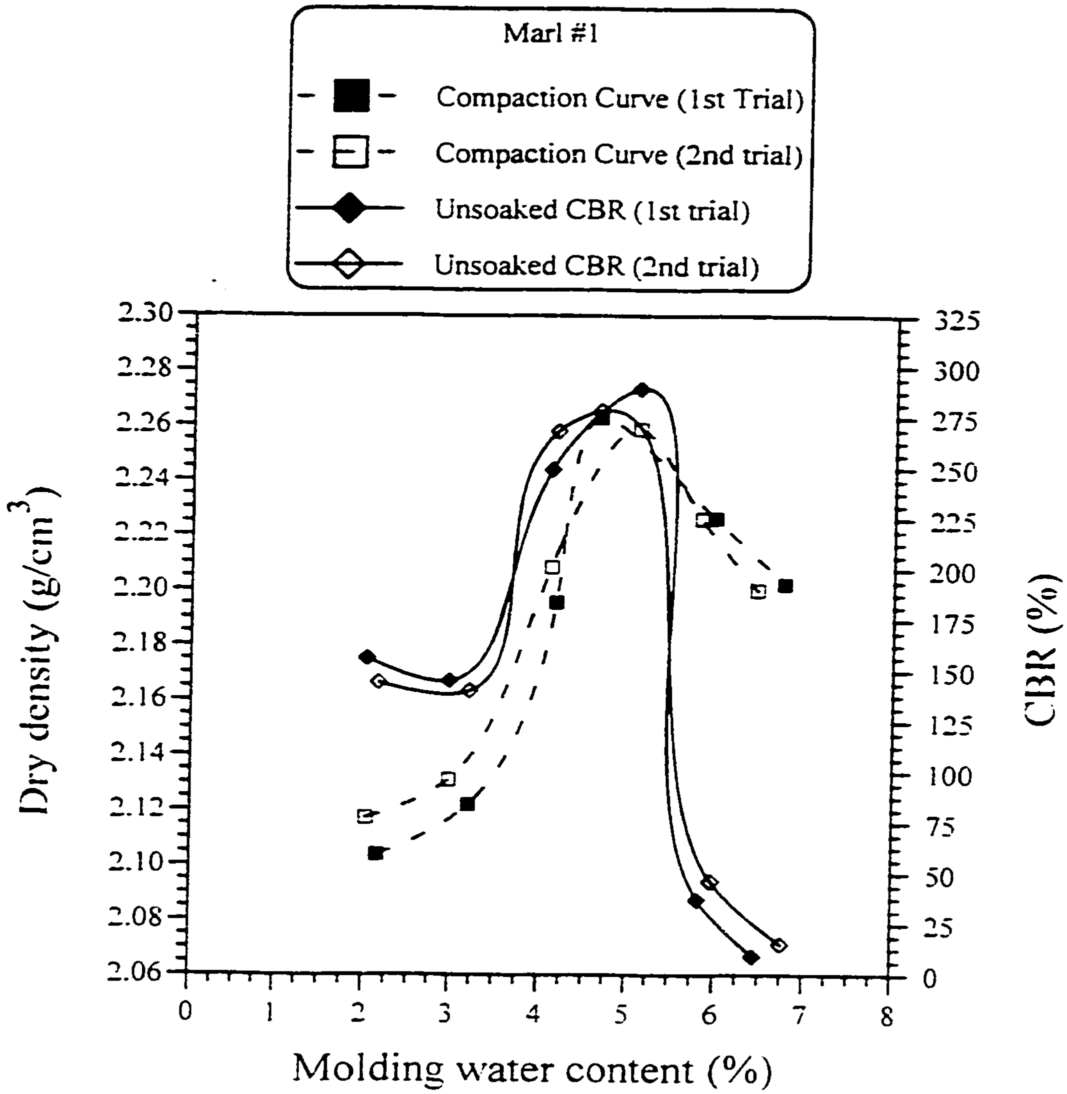


Figure 4.7: The CBR-moisture-density relationships obtained using two trials for marl #1

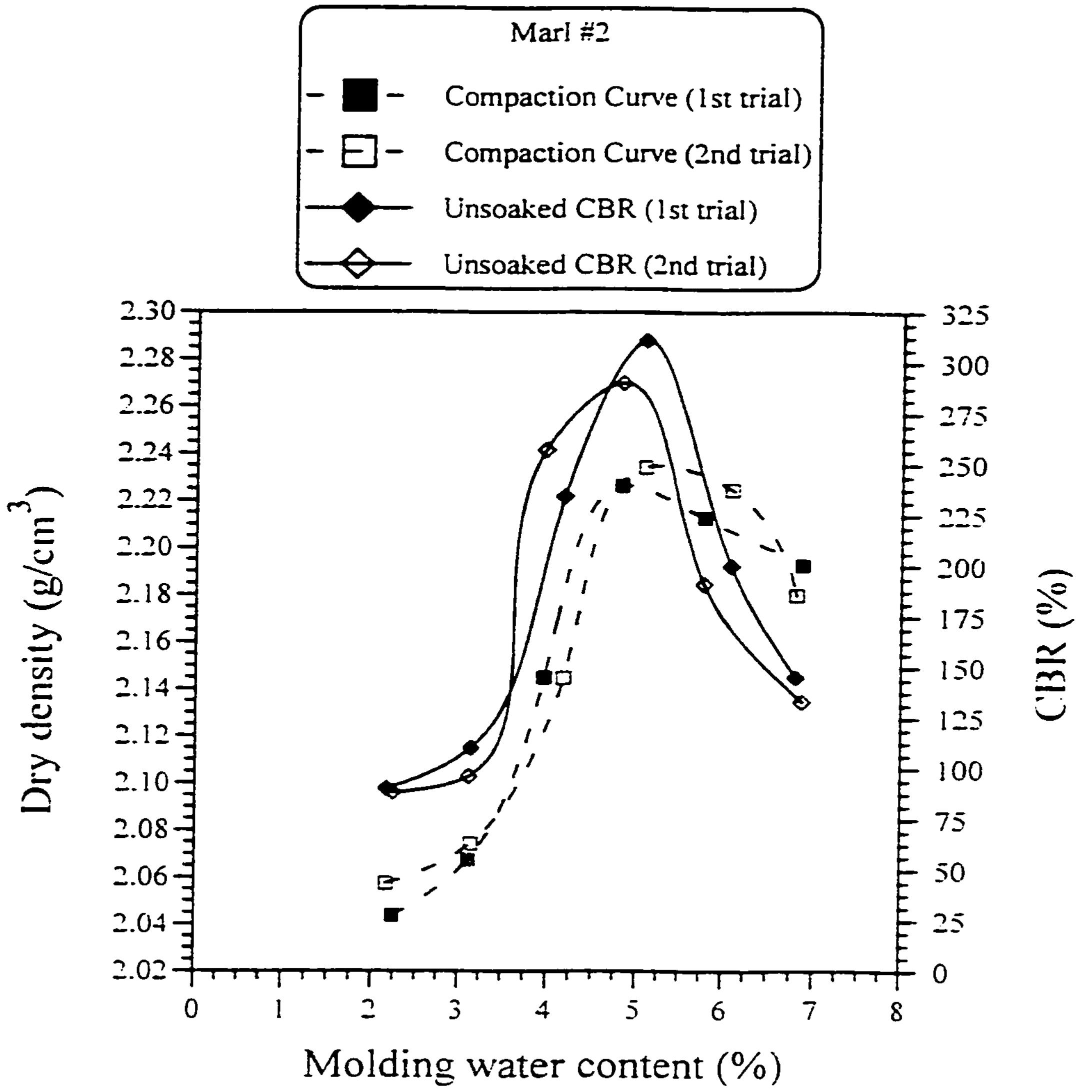


Figure 4.8: The CBR-moisture-density relationships obtained using two trials for marl #2

average value. The CBR values for the second marl gave a maximum difference between the two sets of 23%, which represents 9.4% of the average value. It is observed that the dry density is more repeatable than the CBR values. This could be attributed to the sensitivity of the CBR test to the presence of stony particles. The presence of stony particles underneath the plunger will provide more resistance for penetration and hence gives high variations in the CBR value. Despite this sensitivity the two repeated sets show maximum differences between the CBR values within 10%, which lead to approximately similar curves, and hence an acceptable repeatability. The repeatability of the results was achieved by careful samples preparation and similar testing conditions.

4.2.1 Compaction Tests

The moisture-density relations for the specimens, which are reconstituted to the three selected gradations, were obtained using the modified Proctor method. The three different gradations were chosen in order to study the effect of gradation on the maximum dry density, the optimum moisture content and the load carrying capacity of the material.

The moisture density relations for the three gradations, namely the fine, medium and coarse limits, are shown in Figs 4.9 and 4.10 for Marl #1 and Marl #2, respectively. For Marl #1, the maximum dry densities for the fine, medium and coarse gradations are 2.26 g/cm^3, 2.26 g/cm^3 and 2.25 g/cm^3, respectively, while the corresponding optimum moisture contents are 4.9%, 5.1%, and 4.9% respectively. The maximum dry densities for the three gradations are almost the same. However, the optimum water contents, of the three sets, lie within a range of 0.2%, which represents about 4.1% of the lowest moisture

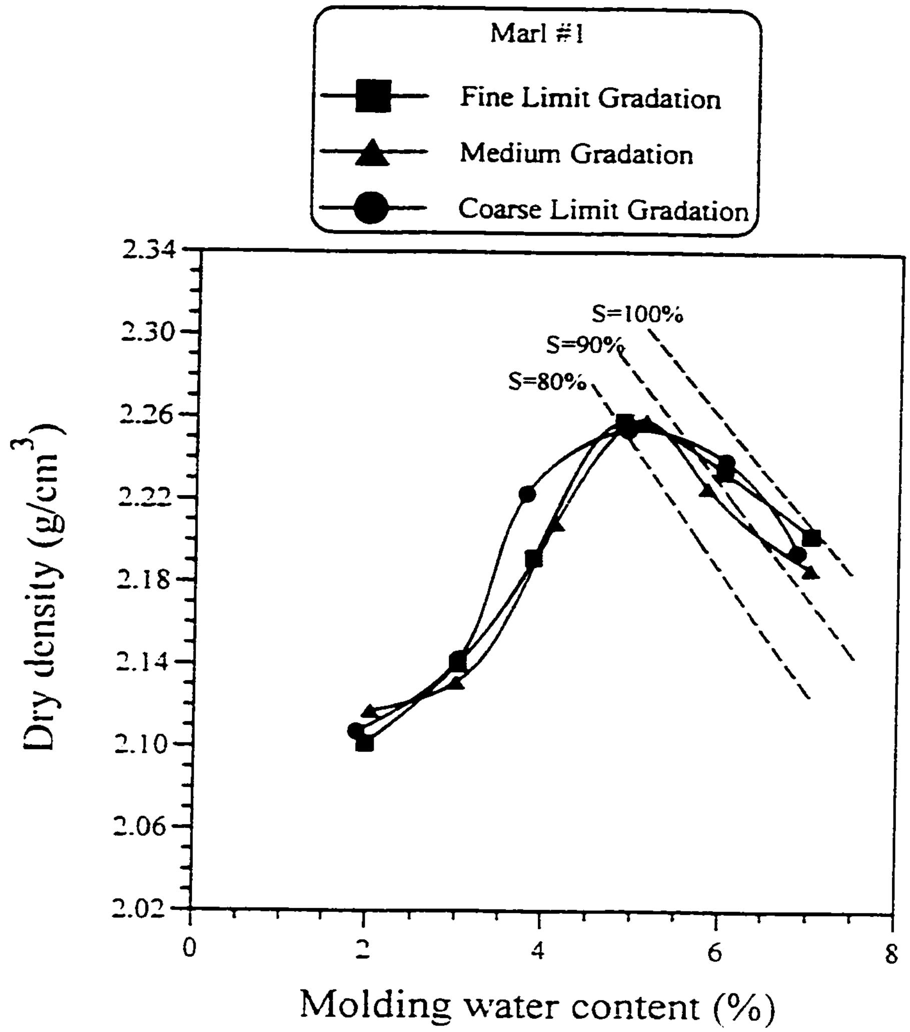


Figure 4.9: The moisture-density relationships of the three gradations for marl #1

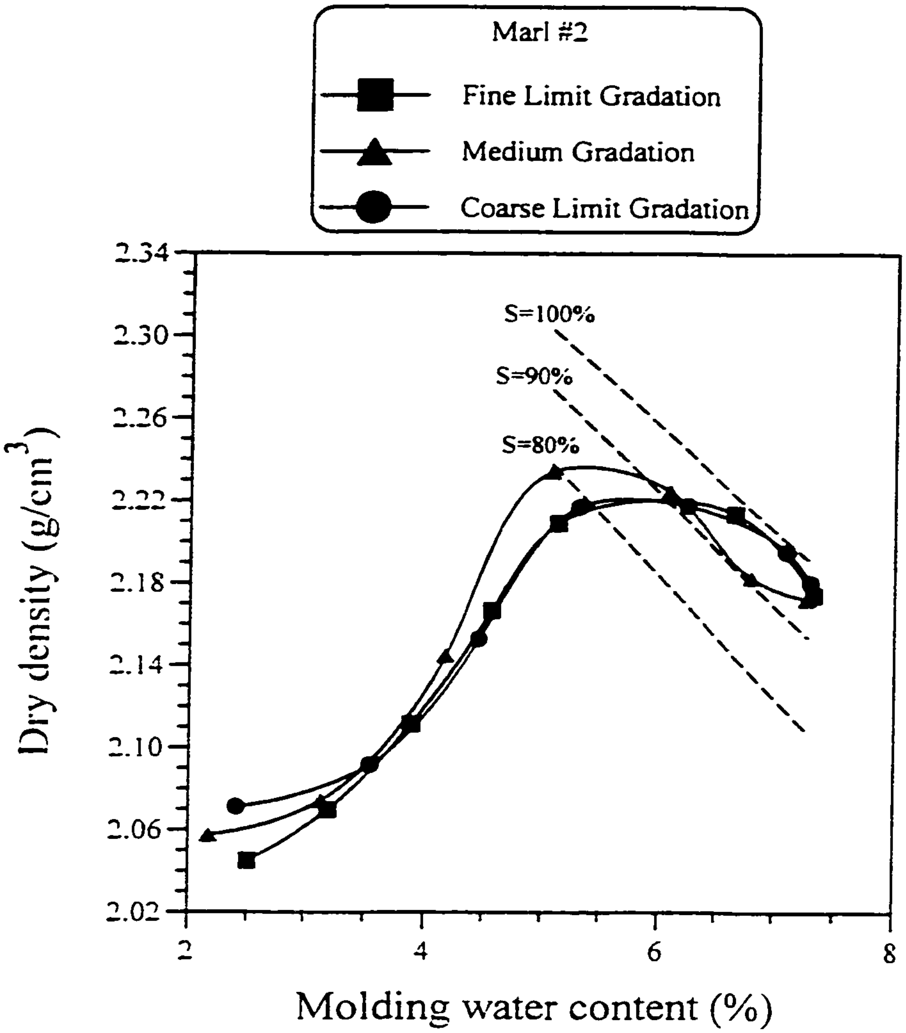


Figure 4.10: The moisture-density relationships of the three gradations for marl #2

content value. The dry density curves at different degrees of saturation are also shown in Fig. 4.9. As shown in the figure, the wet sides of optimum for the three gradations have water contents corresponding to a degree of saturation ranging between 82 and 98%.

For Marl #2, the maximum dry densities for the fine, medium and coarse gradations are 2.22 g/cm^3, 2.24 g/cm^3 and 2.22 g/cm^3, respectively. The corresponding optimum moisture contents are 6.1%, 5.3% and 5.5%, respectively. It is noticed that the maximum dry density values are almost the same. The optimum water contents, of the three sets, lie within a range of 0.8%, which represents 15.1% of the lowest moisture content value. As shown in the saturation curves in Fig. 4.10, the wet sides of optimum for the three gradations have water contents corresponding to a degree of saturation ranging from 85 to 98%.

For both marls, the variation between the maximum dry density values for the three gradations did not exceed 1% of the lowest value, while for optimum moisture contents the maximum variation was 15.1% of the lowest value. Hence, for both marls, the maximum dry density value is somewhat independent of the selected gradations. This is consistent with the findings of Garga and Madureira (1985) who reported the same conclusion for gravelly soils from Brazil. In addition, it is observed that the gradation has slight effect on the optimum moisture content value.

4.2.2 Unsoaked CBR Tests

Both marls were subjected to a testing program to obtain the CBR curves for the three different gradations. The produced test results give an idea about the effect of soil gradation on the CBR- water content relations. As shown in Figs. 4.9 and 4.10, the densities produced by different gradations are almost similar. However, the similarity in densities cannot assure a similar trend of the CBR values, because the CBR values depend also on other factors like the maximum particle size and the pore water pressures of the samples.

The CBR-moisture-density relationships for the three tested gradations are shown in Figs. 4.11 to 4.13. For Marl #1 the maximum CBR values were attained at the optimum moisture content for samples reconstituted to the fine limit gradation and to the medium gradation. However for samples reconstituted to the coarse limit gradation, the maximum CBR value was achieved on the dry side of optimum. The maximum dry density, optimum moisture content and maximum CBR values, for the three gradations, are shown in Table 4.2. The coarse limit gradation shows the lowest maximum CBR value, although it contains higher percentages of stony particles. This can be attributed to the porous structure of the soil reconstituted to the coarse limit gradation and to the effect of grain crushing which is enhanced because of the high friction between the coarse particles. However, the medium gradation set gave the highest CBR values, in all tested sets. This is mainly due to the fact that the sample contains proportional quantities of the different particle sizes and thus it has both coarse material needed for bearing skeleton and fine material needed for voids filling. This is expected to give a matrix that has lower void ratio when compared to the coarse gradation and more bearing skeleton when compared to the fine gradation.

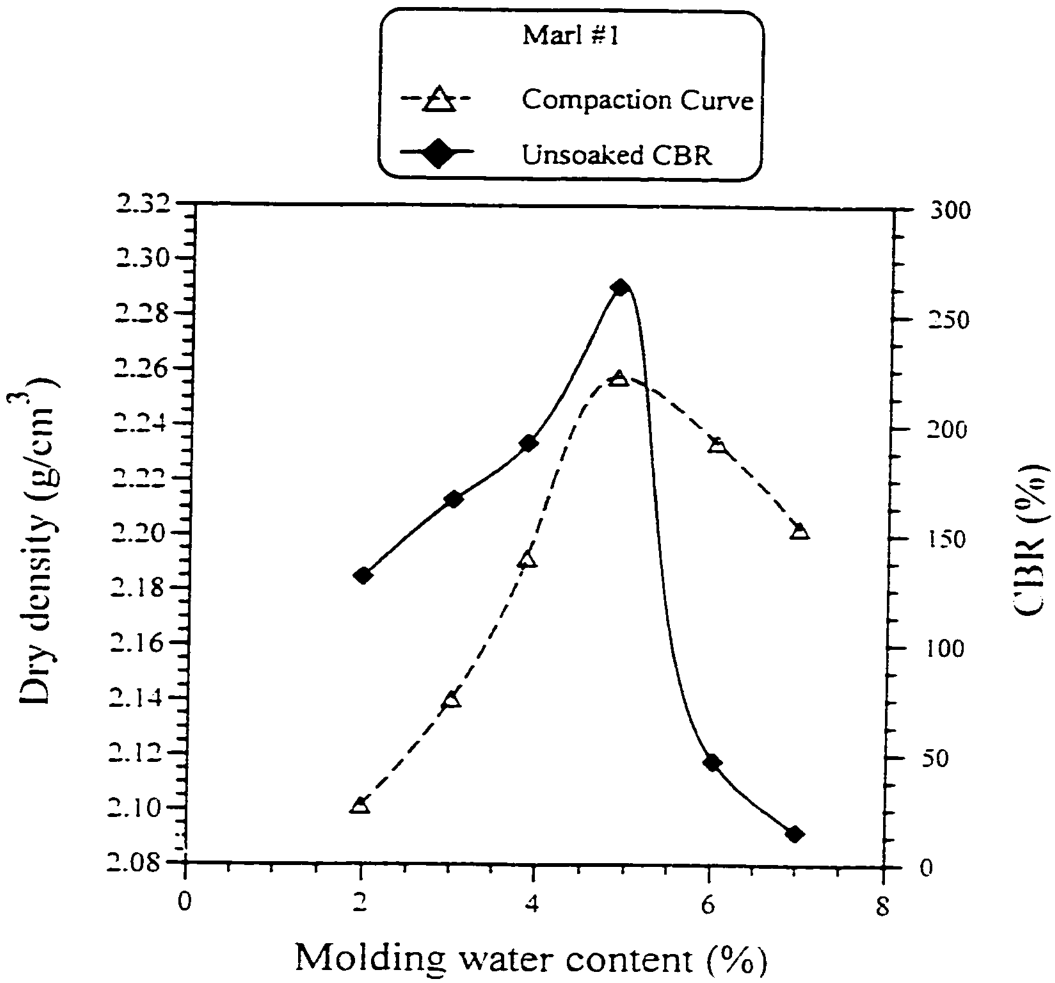


Figure 4.11: Unsoaked CBR-moisture-density relationships of the fine limit gradation for marl #1

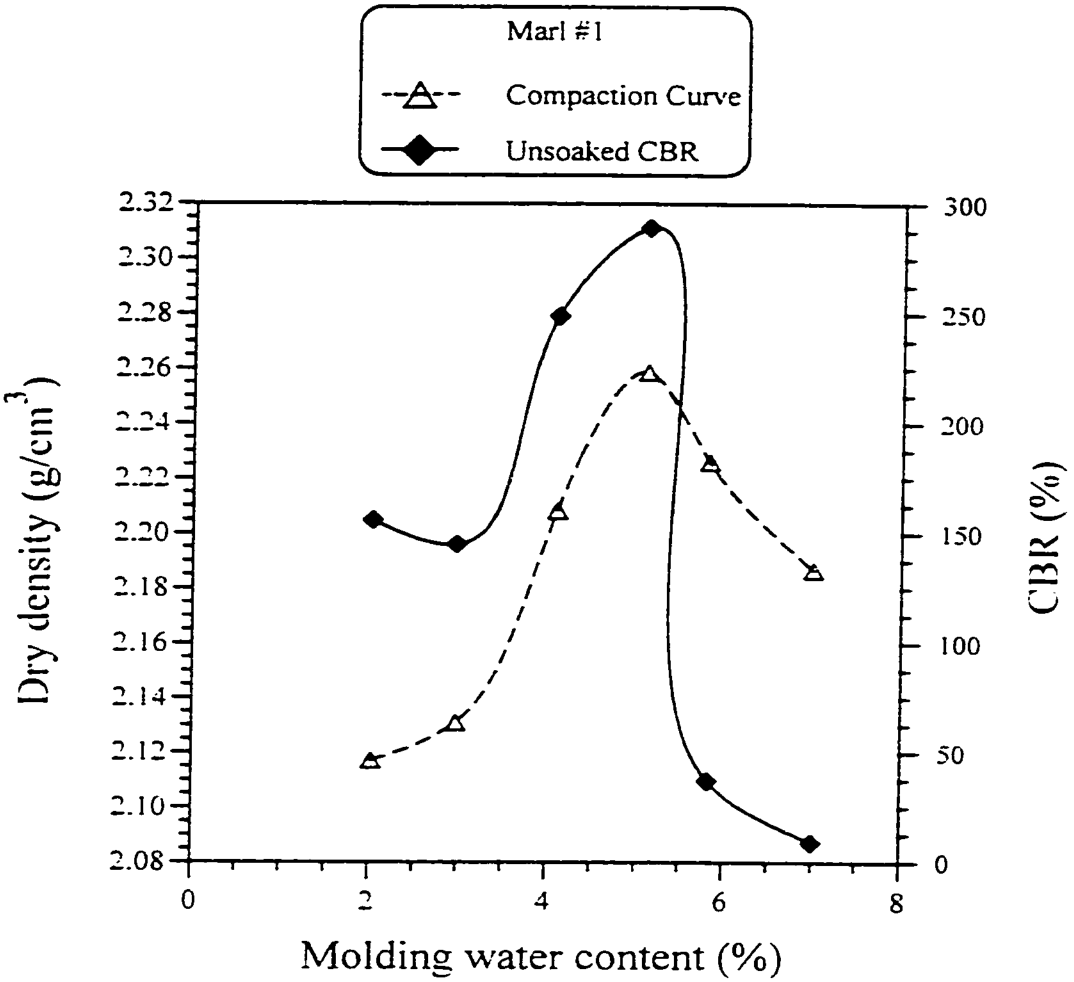


Figure 4.12: Unsoaked CBR-moisture-density relationships of the medium gradation for marl #1

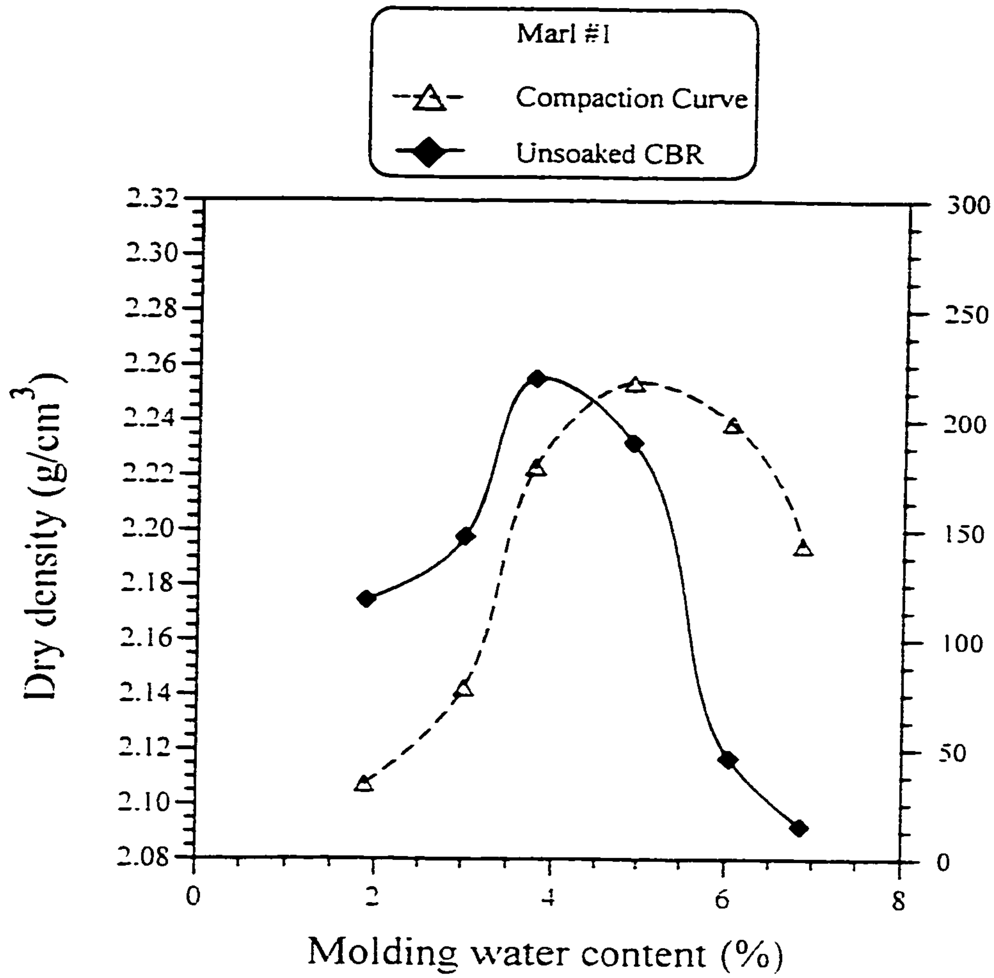


Figure 4.13: Unsoaked CBR-moisture-density relationships of the coarse limit gradation for marl #1

Table 4.2: The γ_{dmax}, w_{opt} and CBR_{max} values of the two marls for different gradations

Gradation	Marl #1			Marl #2		
	γ_{dmax} (gm/cm^3)	w_{opt}	CBR_{max}	γ_{dmax} (gm/cm^3)	w_{opt}	CBR_{max}
Fine Limit Gradation	2.26	4.9	263	2.22	6.1	241
Medium Gradation	2.26	5.1	289	2.24	5.3	311
Coarse Limit Gradation	2.25	4.9	219	2.22	5.5	360

For samples reconstituted using the medium and the fine gradations, a sharp drop after the maximum CBR values was observed when only about 1% moisture was added. However, for samples reconstituted to the coarse limit gradation, a relatively gradual drop after the peak CBR value was observed. The density at which the maximum CBR value was attained may contribute to this phenomenon. For samples reconstituted to the medium and to the fine limit gradations, the densities of the specimens tested beyond the peak CBR points were less than the densities at the peak values. The peak CBR values for the two gradations were achieved at the optimum moisture content. However, for samples reconstituted to the coarse limit gradation, the density of the specimen tested right after the peak CBR point is higher than the density at which the peak CBR was obtained. The peak CBR value was achieved on the dry side of optimum.

It is observed that for the three gradations, the CBR values obtained for all points tested on the dry side of optimum as well as at the optimum water content exceeded 100%. However, the CBR values dropped below 50% on the wet side of optimum. This phenomenon was found to be common although the specimens tested on the wet side have higher densities than those tested at the dry side of optimum. This observation substantiate the fact that compacting marl soil at high density will never be a sufficient condition for a production of high CBR value, especially on the wet side of optimum.

For Marl #2, the maximum CBR values for the three gradations were attained on the dry side of optimum. The maximum dry density, optimum moisture content and maximum CBR values, for the three gradations, are shown in Table 4.2. The CBR-moisture-density relationships for the three gradations are shown in Figs. 4.14 to 4.16. The maximum CBR value was produced by the coarse limit gradation set. This is caused by the high stone content of this gradation and the good quality of the aggregates, which are expected to undergo less degree of crushing during compaction. High stone content soil is usually expected to produce high CBR values because of their high resistance to the penetrating CBR plunger, as long as there is enough filler. These high CBR values are generated by friction between the stony particles as a result of the applied static load. This friction is enhanced when the stony particles are stiff and have high abrasion resistance. The fine limit gradation specimens show the lowest maximum CBR value and this is attributed to their fine gradation and relatively low stone content. It is observed also that the majority of the tested specimens resulted in CBR values exceeding 100% for all tested gradations at all compaction moisture content values.

The unsoaked CBR curves for the three tested gradations for Marl #1 and Marl #2 are plotted in Figs. 4.17 and 4.18, respectively. It is shown that at low and high water contents the three curves tend to approach each other for both marls. On the very dry side, the soil skeleton will have porous structure because of the difficulty of particles rearrangement in the absence of water lubrication during compaction. However, applying the static load through the CBR plunger, the soil beneath the plunger will tend to compress and thus reducing the void spaces. Hence, the penetration will take place till it reaches the standard ½ in. (12.8 mm), while the applied load is transferred partially to the soil aggregates as a result of the presence of the porous media of the sample. It is

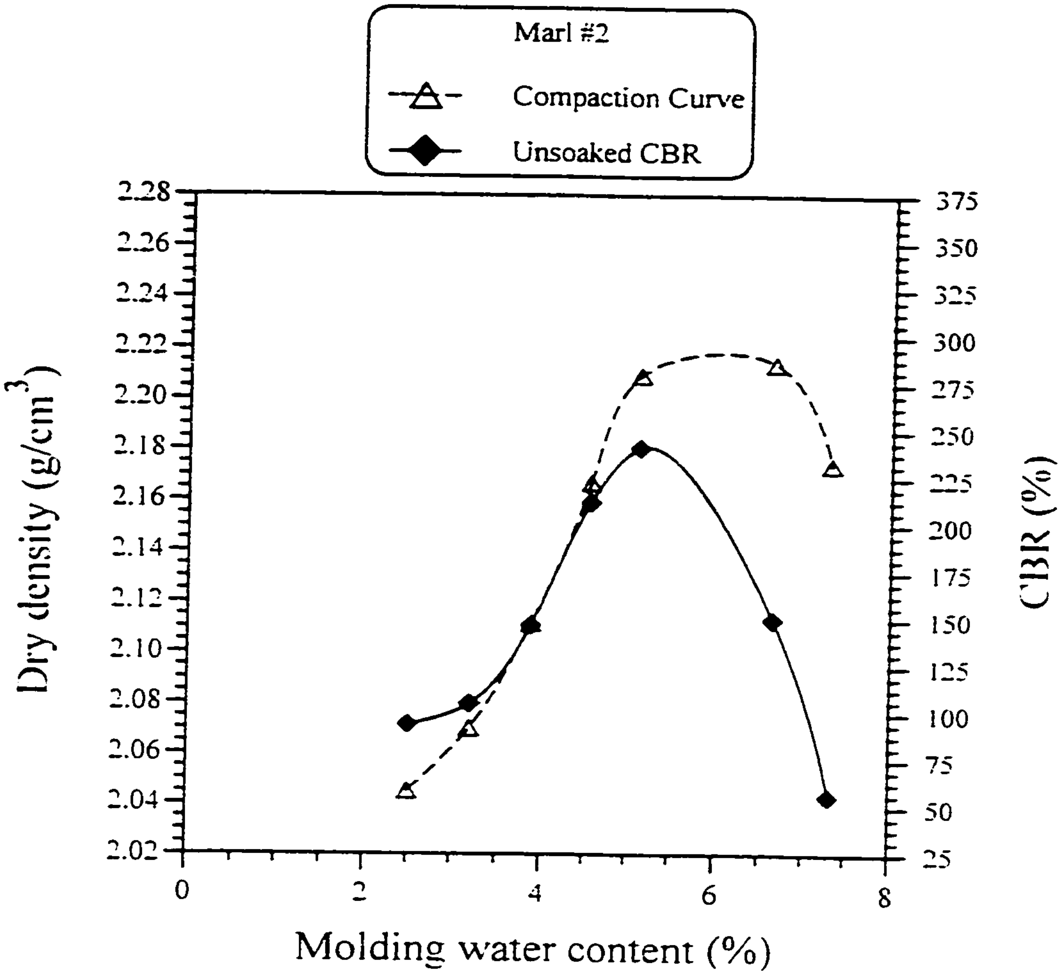


Figure 4.14: Unsoaked CBR-moisture-density relationships of the fine limit gradation for marl #2

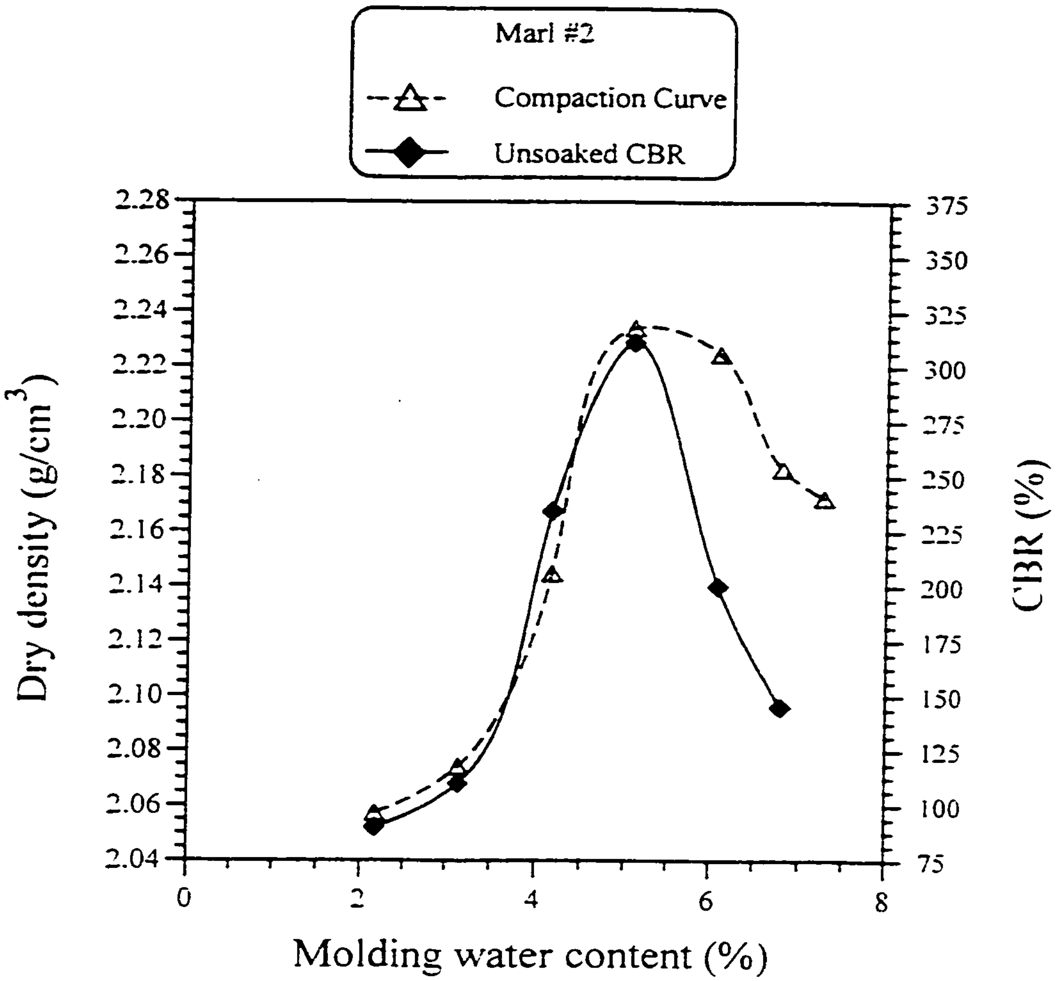


Figure 4.15: Unsoaked CBR-moisture-density relationships of the medium gradation for marl #2

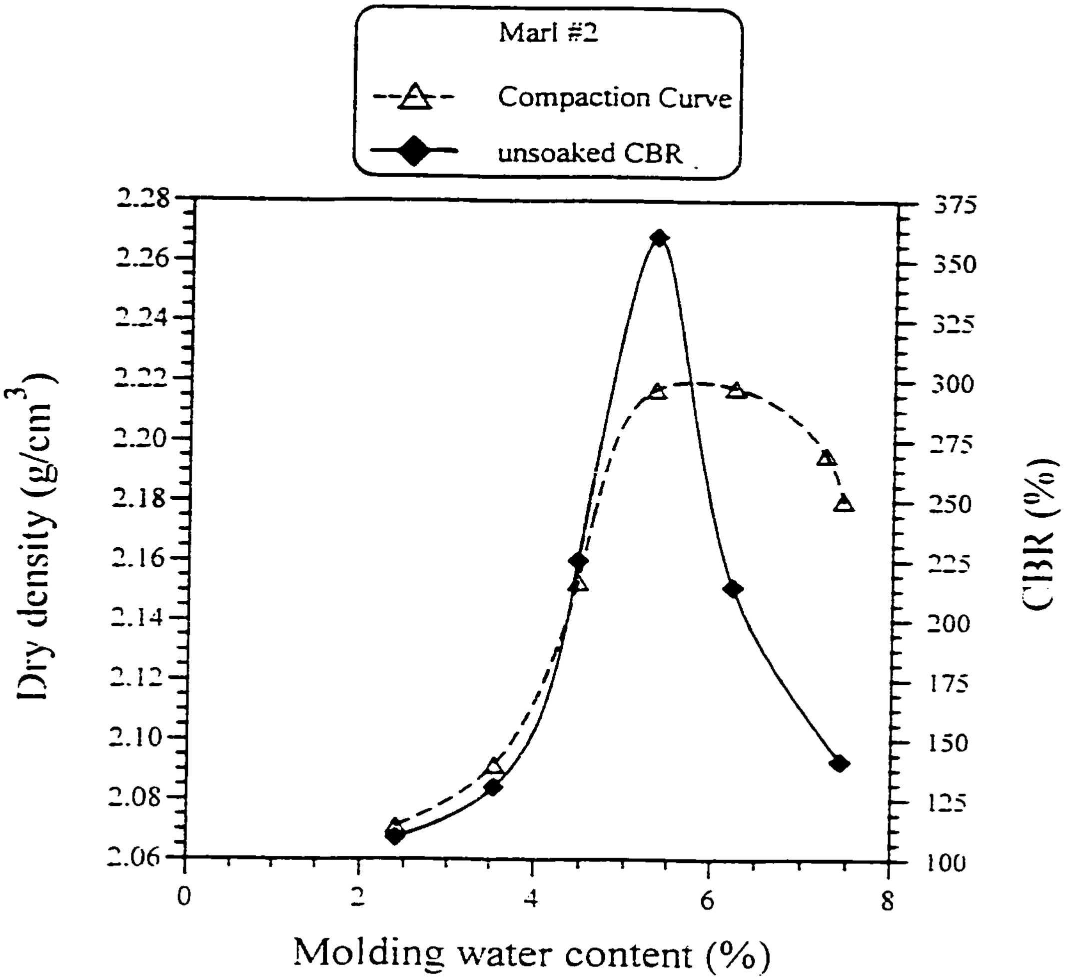


Figure 4.16: Unsoaked CBR-moisture-density relationships of the coarse limit gradation for marl #2

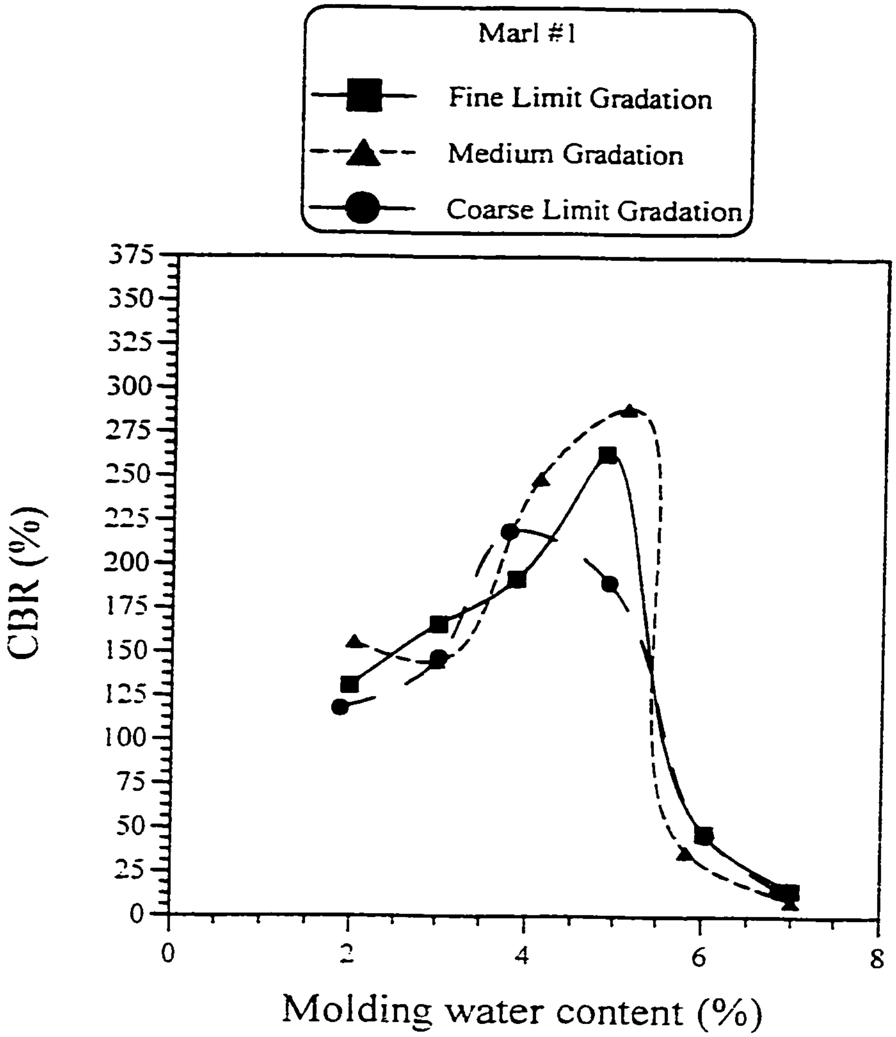


Figure 4.17: Unsoaked CBR-moisture relationships of the three gradations for marl #1

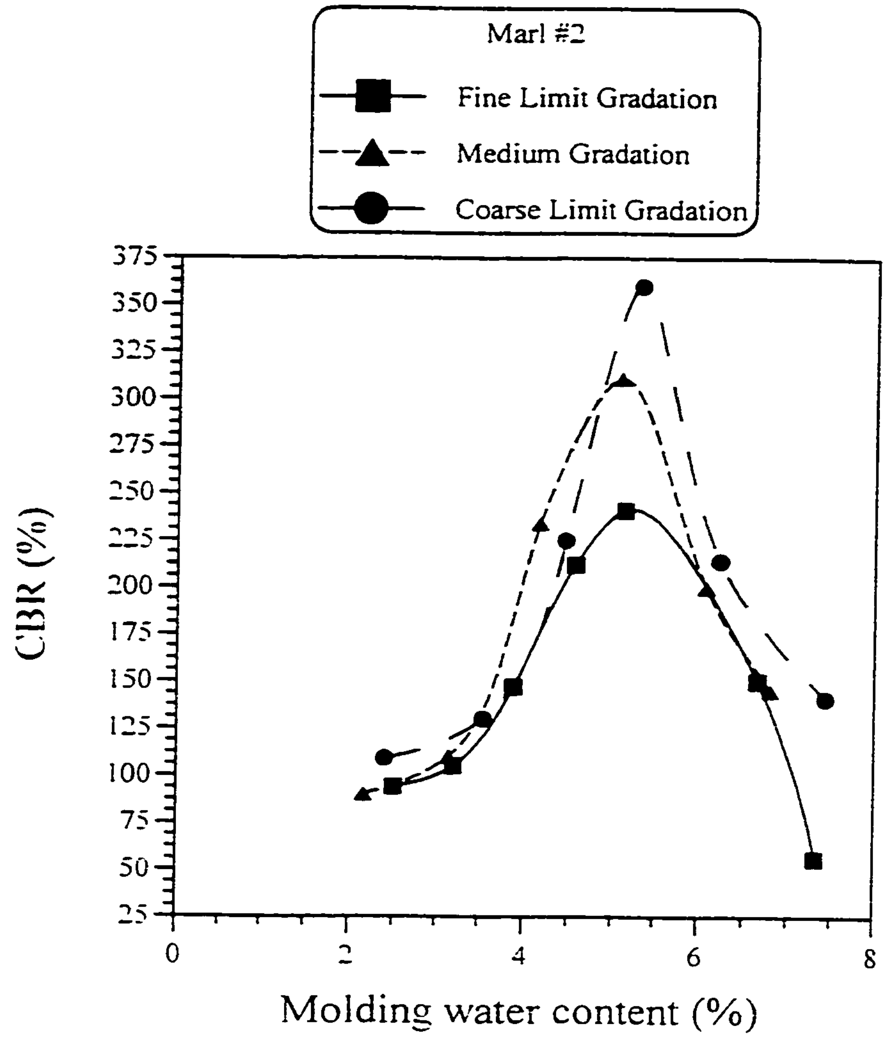


Figure 4.18: Unsoaked CBR-moisture relationships of the three gradations for marl #2

therefore expected that the soil gradation will play a minor role in the CBR values of the soil at low moisture content values.

At high water content values the collapse of the CBR curves for the three gradations towards each other is attributed to the effect of excess pore water pressure and softening of the material. Therefore, at the very wet side of optimum, soil gradation has slight effect on the CBR values. This phenomenon was substantiated by the squeezing of water out of the samples at the very wet side of optimum in the loading process. This clear when considering the saturation curves shown in Figs. 4.7 and 4.8, where the wet side of optimum for the two marls have a degree of saturation ranging from 82 to 98%. Comparing the two tested marls, it is found that generally Marl #1 has higher CBR values on the dry side of optimum, while Marl #2 has higher CBR values at both the optimum and on the wet side of optimum as shown in Fig. 4.19.

4.2.3 Unconfined Compressive Strength Tests

In order to study the load carrying capacity of the collected marl soils for different gradations, a more reliable test is needed to support the CBR results. The unconfined compressive strength (UCS) of a soil is used to identify the strength of the soil without confinement. For the three selected gradations, the UCS values were obtained for samples compacted at different moisture contents. Samples were prepared at densities approximately equal to those obtained using the CBR mold. Generally, soil specimen will attain high UCS value when its internal strength (a product of the firm binding between soil particles) has higher resistance to the applied static load.

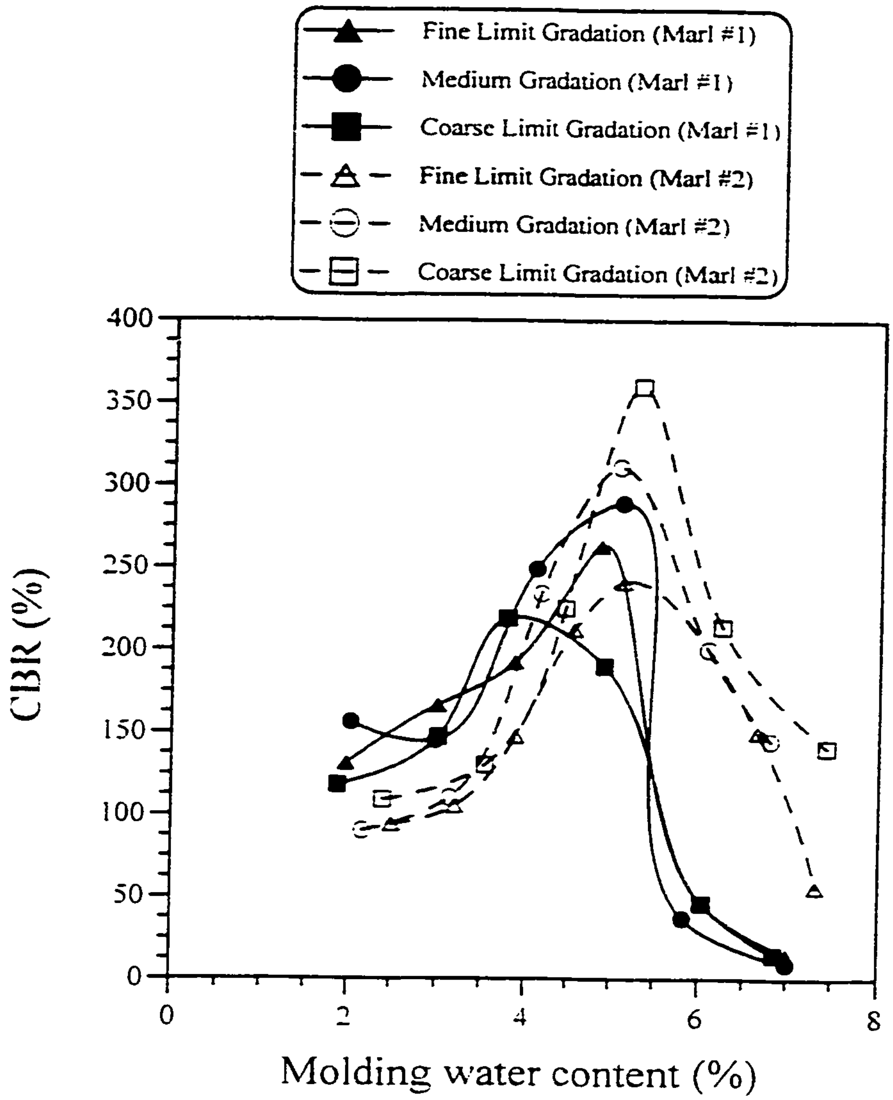


Figure 4.19: Unsoaked CBR-moisture relationships of the three gradations for both marls

It was observed that compacting a sample with a certain gradation in the UCS mold usually gives higher maximum dry density than compacting it in the standard CBR mold. This difference in densities was observed when the hammer for modified Proctor, drop height and number of blows were used. In order to achieve approximately equal dry densities, for the CBR samples and the unconfined compression samples, a series of samples were compacted in the UCS mold with different number of blows. It was found that, for the UCS mold, 32 blows on each layer are adequate to achieve dry densities approximately equal to those obtained using the CBR mold. The relation between the number of blows and the produced dry densities is shown in Fig. 4.20.

The variations of the dry density and the UCS with the molding moisture content for Marl #1 are shown in Figs. 4.21 to 4.23 for the three gradations. The maximum dry density, optimum moisture content and maximum UCS values, for the three gradations, are summarized in Table 4.3. The results shown in table 4.3, for the moisture-density relationships obtained from the UCS mold, showed that the maximum dry density and the optimum moisture content values obtained for the three sets are very close to each other. Hence, this again shows that the maximum dry density and the optimum moisture content values are independent of the selected gradations.

Results also show clearly that the maximum UCS values, for the samples tested at the selected gradations, were obtained at relatively low moisture contents i.e. on the dry side of optimum. For test specimens reconstituted to the fine limit gradation the maximum UCS value was attained at moisture content of 3.1%. Specimens reconstituted to the

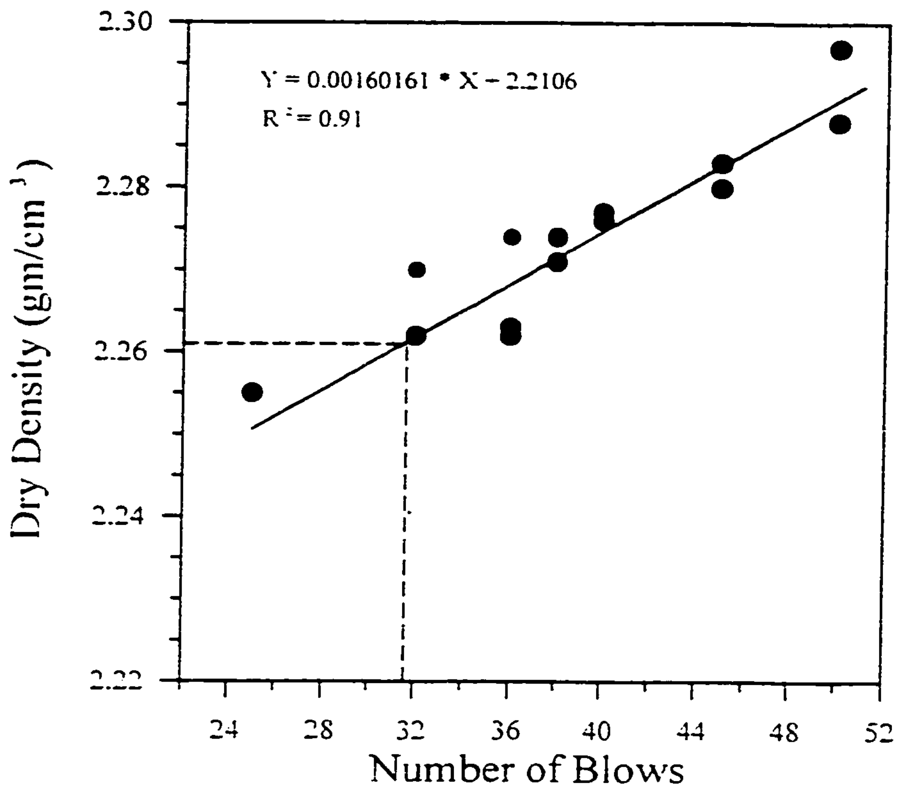


Figure 4.20: The relation between dry density and number of blows for UCS mold

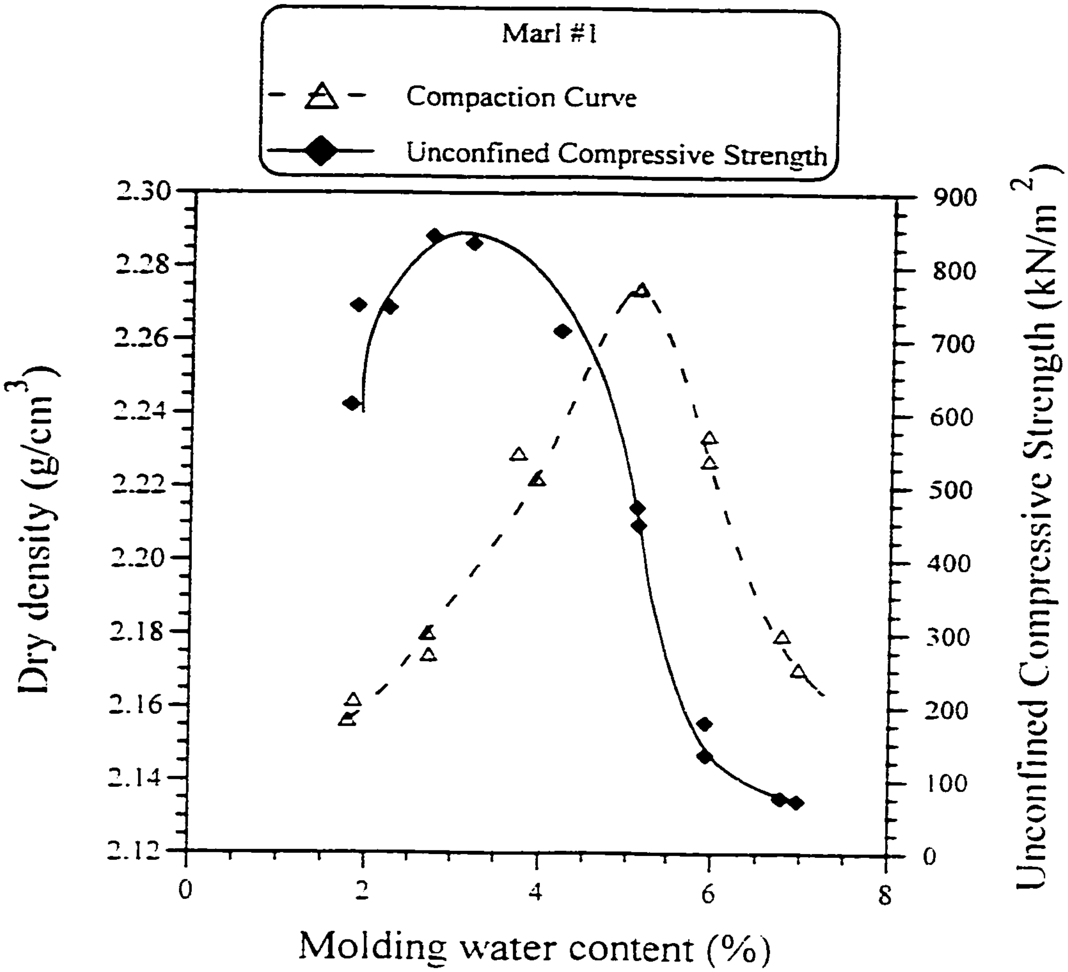


Figure 4.21: UCS-moisture-density relationships of the fine limit gradation for marl #1

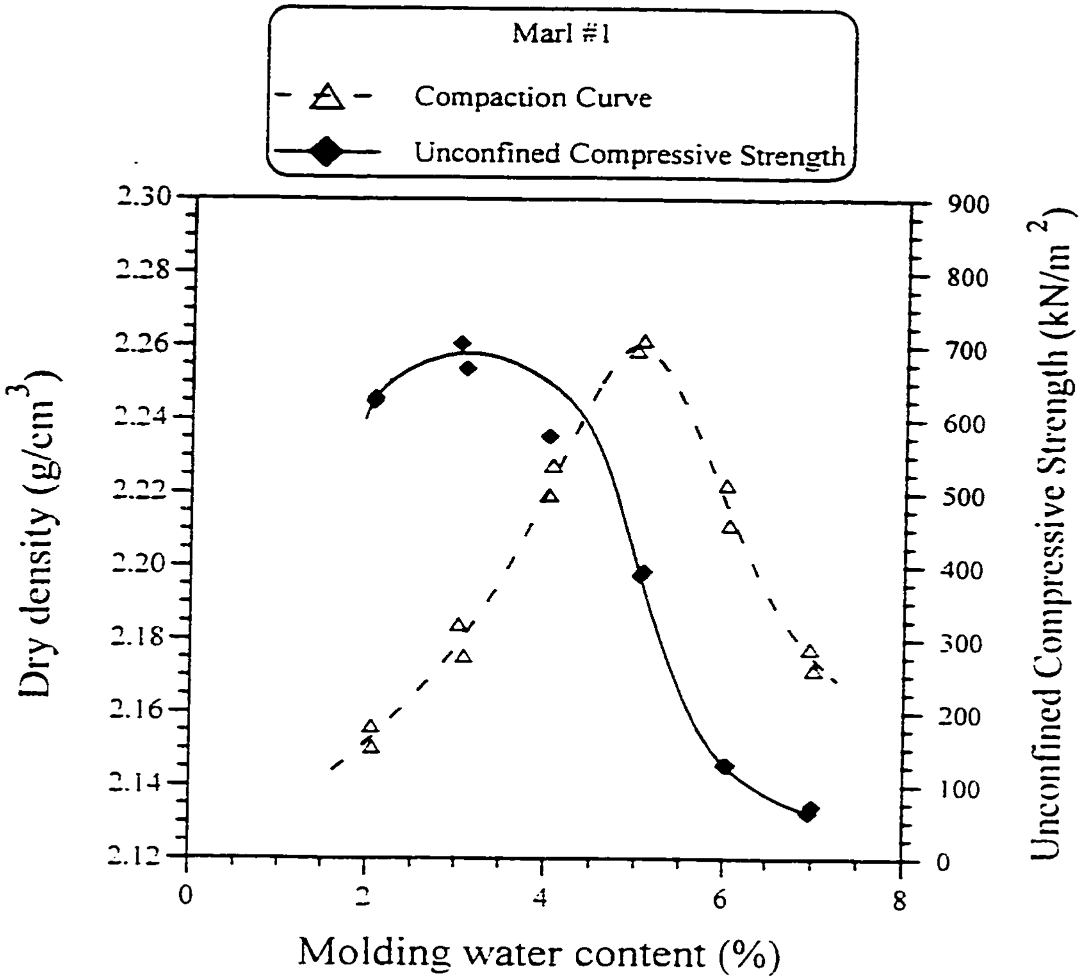


Figure 4.22: UCS-moisture-density relationships of the medium gradation for marl #1

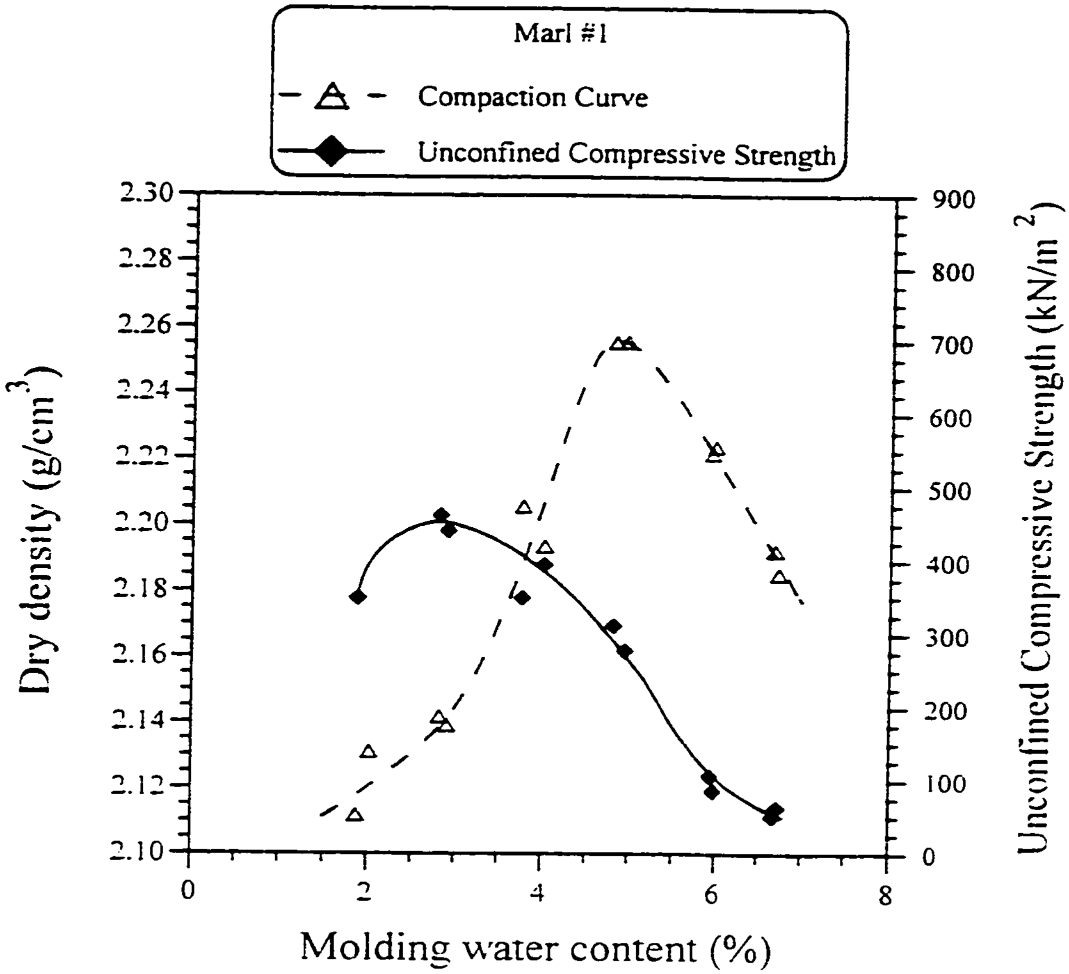


Figure 4.23: UCS-moisture-density relationships of the coarse limit gradation for marl #1

Table 4.3: The γ_{dmax}, w_{opt} and UCS_{max} values of the two marls for different gradations

Gradation	Marl #1			Marl #2		
	γ_{dmax} (gm/cm³)	w_{opt}	UCS_{max} (kN/m²)	γ_{dmax} (gm/cm³)	w_{opt}	UCS_{max} (kN/m²)
Fine Limit Gradation	2.27	5.1	845	2.23	6.3	348
Medium Gradation	2.26	5.1	690	2.25	6.3	343
Coarse Limit Gradation	2.26	4.8	454	2.23	6.3	240

medium gradation attained the maximum UCS value at moisture content of 3.2% and for specimens reconstituted to the coarse limit gradation the maximum UCS value was achieved at moisture content of 2.9%.

The UCS-moisture-density relationships for Marl #1 show that the maximum UCS values were obtained, for all gradations, at relatively low dry densities compared to the maximum dry density values and on the dry side of optimum. For the fine, medium and coarse gradations, the UCS_{max} was attained at dry densities of 2.19 gm\cm^3, 2.18 gm\cm^3 and 2.14 gm\cm^3, respectively. These values correspond to degrees of compaction of 96, 96, 95% for the three gradations, respectively. In addition, it is observed that at the optimum moisture content the UCS values are relatively low when compared to the maximum values. The fine limit gradation produced UCS of 448 kN\m^2, which is 53% of the UCS_{max}, while the medium gradation attained 387 kN\m^2 UCS at the optimum, which is only 56% of the UCS_{max}. The coarse limit gradation gave 280 kN\m^2 UCS, which is 62% of the UCS_{max}.

The UCS curves obtained for the three gradations for Marl #1 are shown in Fig. 4.24. The fine limit gradation shows, generally, the highest UCS values at all moisture contents while the coarse gradation shows the least UCS values. This is attributed to the high amount of fines in the fine gradation and relatively low amount of fines in the coarse gradation. The fine materials, especially when they possess some plasticity, work as a binding agent, which binds the coarse aggregates together, and this what produces cohesion. As shown in the figure, the UCS values for the three gradations, approach each

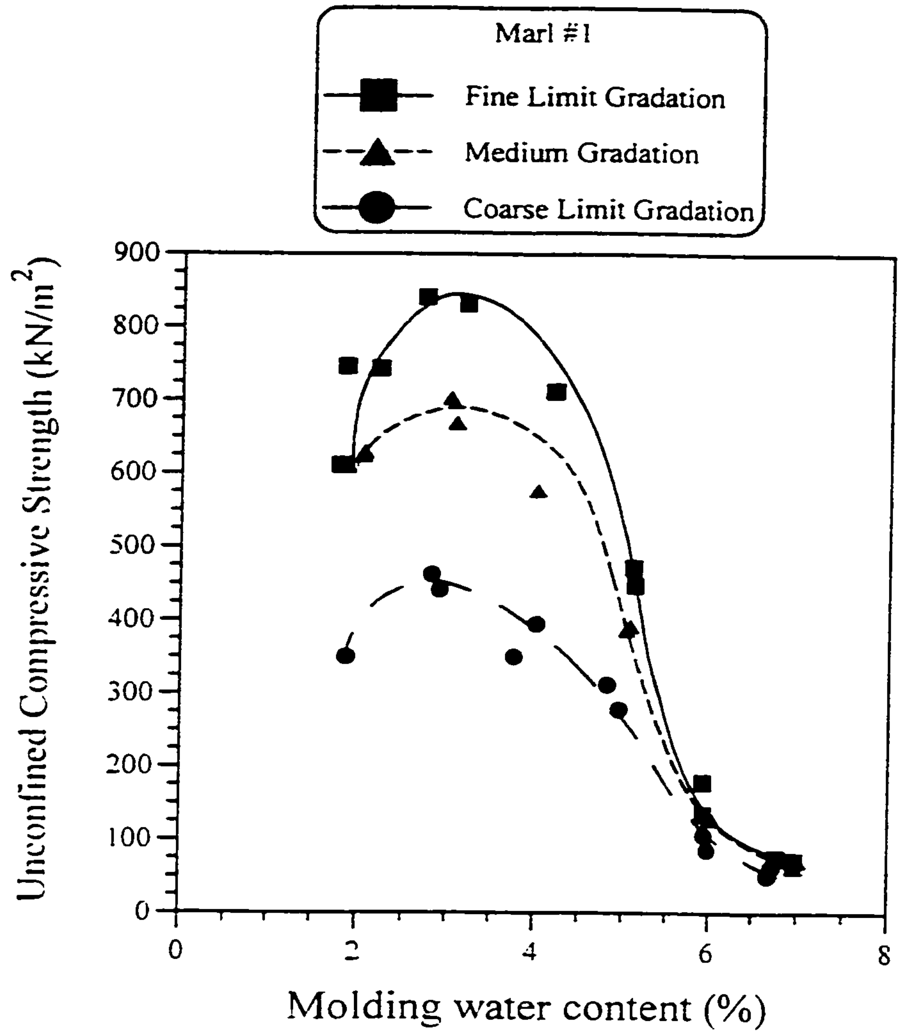


Figure 4.24: UCS-moisture relationships of the three gradations for marl #1

other on the wet side of optimum. Hence, for Marl #1 the UCS values at high moisture contents, i.e. on the wet side of the optimum, are independent of soil gradation.

For Marl #2, the UCS-moisture-density relationships are shown in Figs. 4.25 to 4.27. As shown in Table 4.3 the maximum dry density values obtained for the three gradations are approximately the same. In addition, the three gradations gave the same optimum moisture content values. This again substantiates the independency of the maximum dry density and optimum moisture content values on the selected gradations.

The maximum UCS values, for the samples tested at the three gradations, were also attained on the dry side of optimum. For test specimens reconstituted to the fine limit gradation the maximum UCS value was found at 4% moisture content. Specimens reconstituted to the medium gradation attained maximum UCS at 3.8% moisture content and for the third set of specimens, which were reconstituted to the coarse limit gradation the maximum UCS value was achieved at 4.3% moisture.

As shown in the UCS-moisture-density relationships, for Marl #2, the maximum UCS values were obtained, for all gradations at dry densities lower than the maximum dry densities. For the fine, medium and coarse gradations, the UCS_{max} was attained at dry densities of 2.16 gm\cm^3, 2.16 gm\cm^3 and 2.16 gm\cm^3, respectively. These values correspond to degrees of compaction of 97, 96, 97% for the three gradations, respectively. In addition, it is observed that at the optimum moisture content the UCS values are relatively low when compared to the maximum values. The fine limit gradation gave UCS of 275 kN\m^2, which is only 79% of UCS_{max} while for the medium gradation the UCS

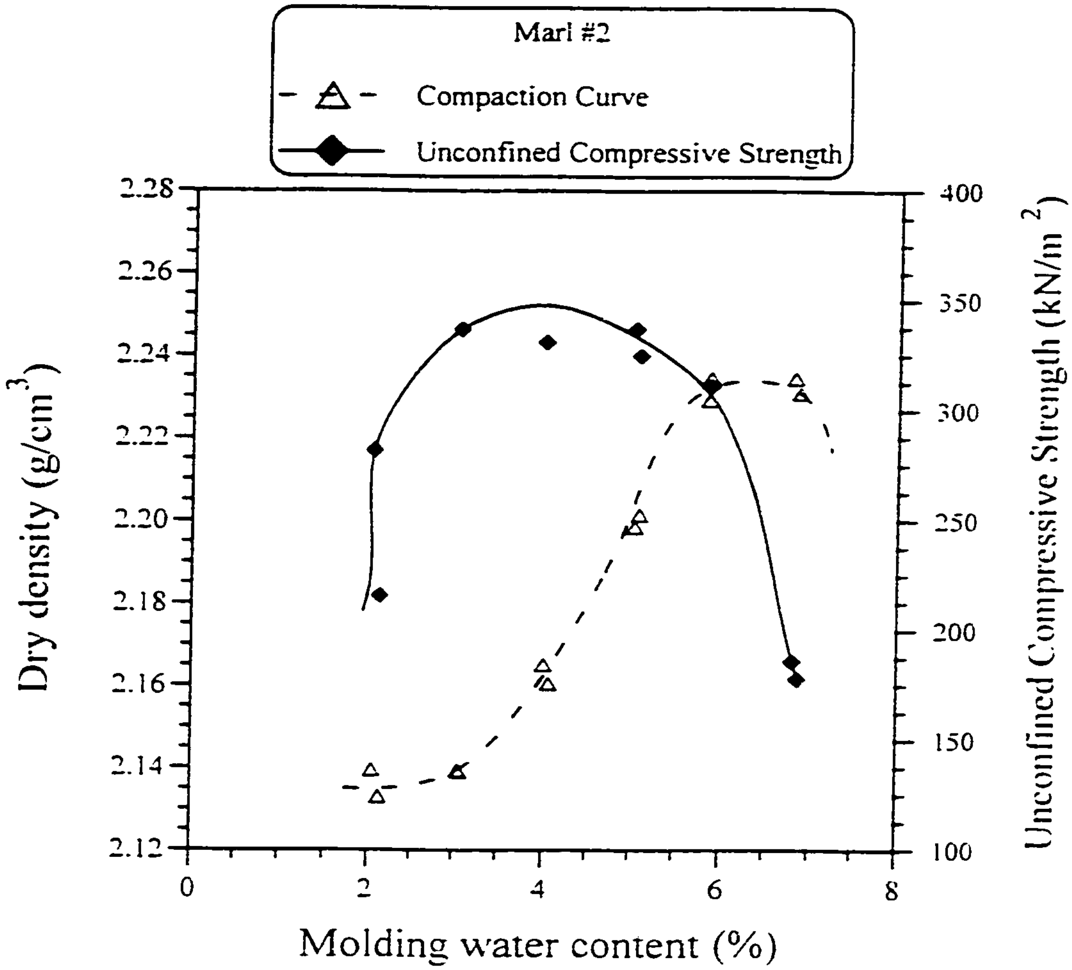


Figure 4.25 UCS-moisture-density relationships of the fine limit gradation for marl #2

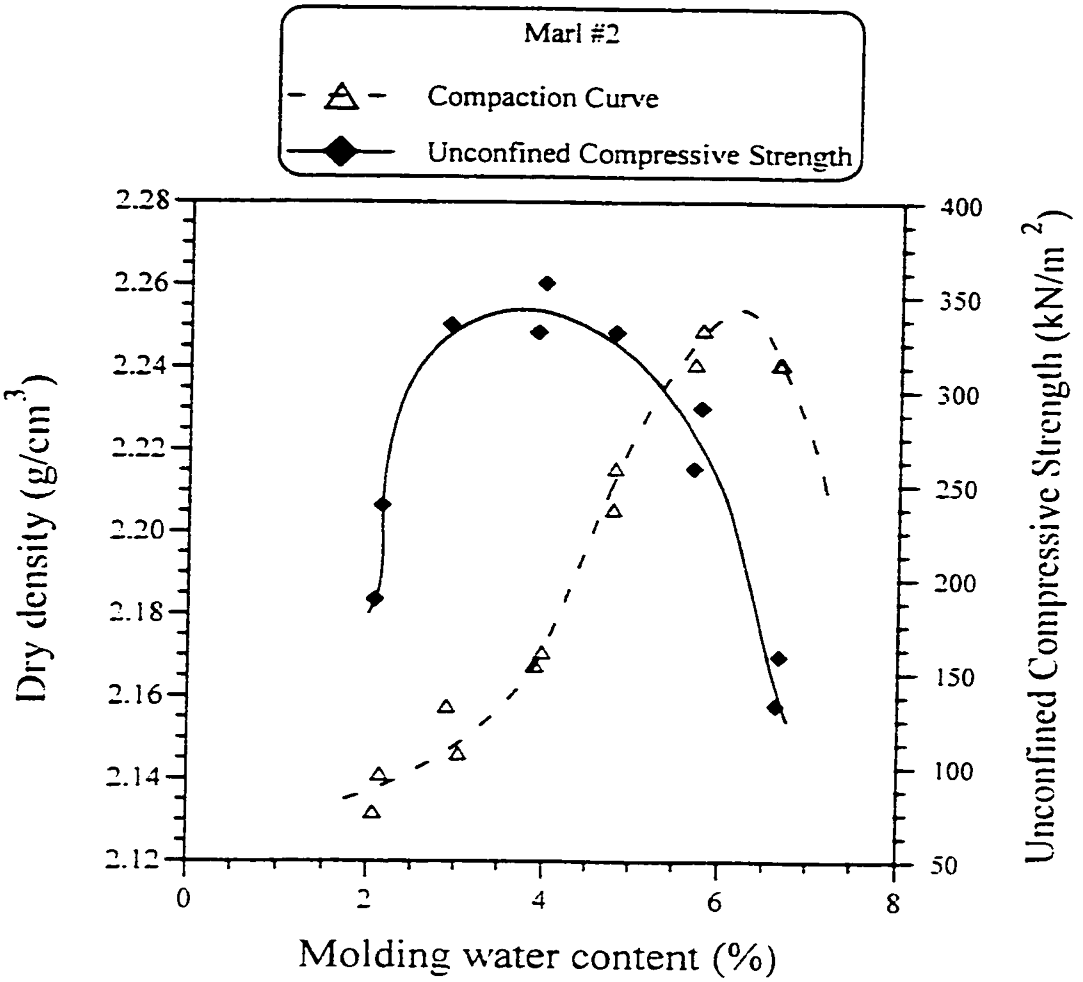


Figure 4.26: UCS-moisture-density relationships of the medium gradation for marl #2

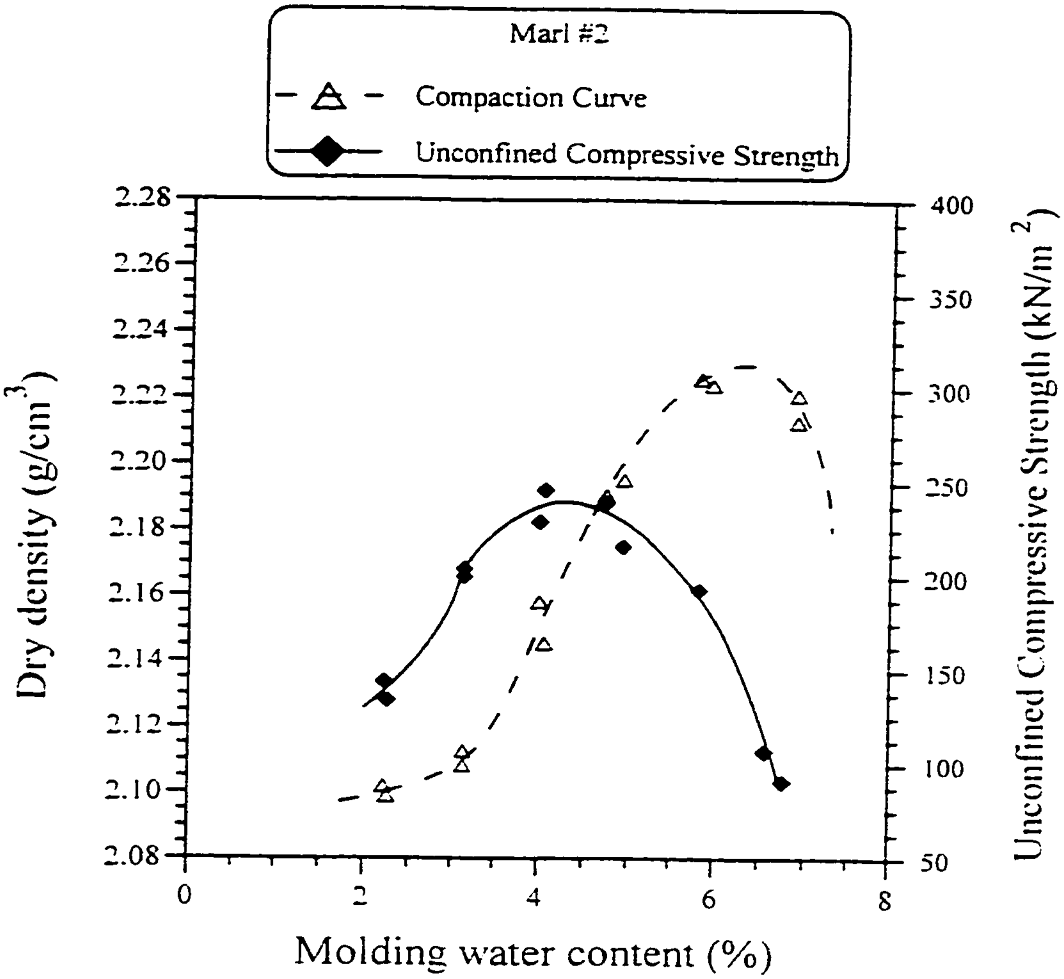


Figure 4.27: UCS-moisture-density relationships of the coarse limit gradation for marl #2

value at optimum was 214 kN\m^2, which is 62% of UCS_{max}. The coarse limit gradation obtained 138 kN\m^2 UCS at the optimum moisture content, which is 58% of UCS_{max}. Hence, for both marls, the UCS values at the optimum moisture content are relatively low compared to the maximum values. Hence, maximum dry densities are not sufficient condition for maximum UCS values.

The UCS curves for Marl #2, for the three selected gradations, show the same trend as the one observed for Marl #1. The fine limit gradation gave the highest UCS values, at all moisture contents, while the lowest UCS values were obtained for the coarse limit gradation, for all moisture contents. The amount of fines is the basic parameter, which lead to such ranking for the UCS curves. The UCS curves for the three gradations are shown in Fig. 4.28. It is clear that the UCS values for the fine and medium gradations are close to each other while the coarse gradation set lies much below these two. In addition, the variation between the UCS values for the three gradations is much lower at the extreme sides of the curves.

The variations of the UCS with the molding moisture content, for both marls, are shown in Fig. 4.29. Generally, Marl #1 shows higher UCS values on the dry side of optimum and at the optimum moisture content. This is caused by the adhesion of the plastic fines of Marl #1. While at the very wet side of optimum, all curves tend to merge close to each other. However, Marl #2 shows a relatively higher UCS value at higher water contents. This is attributed to the fact that Marl #2 is cohesionless and thus the effect of water (on the wet side of optimum) is not significant. In addition, the trends of

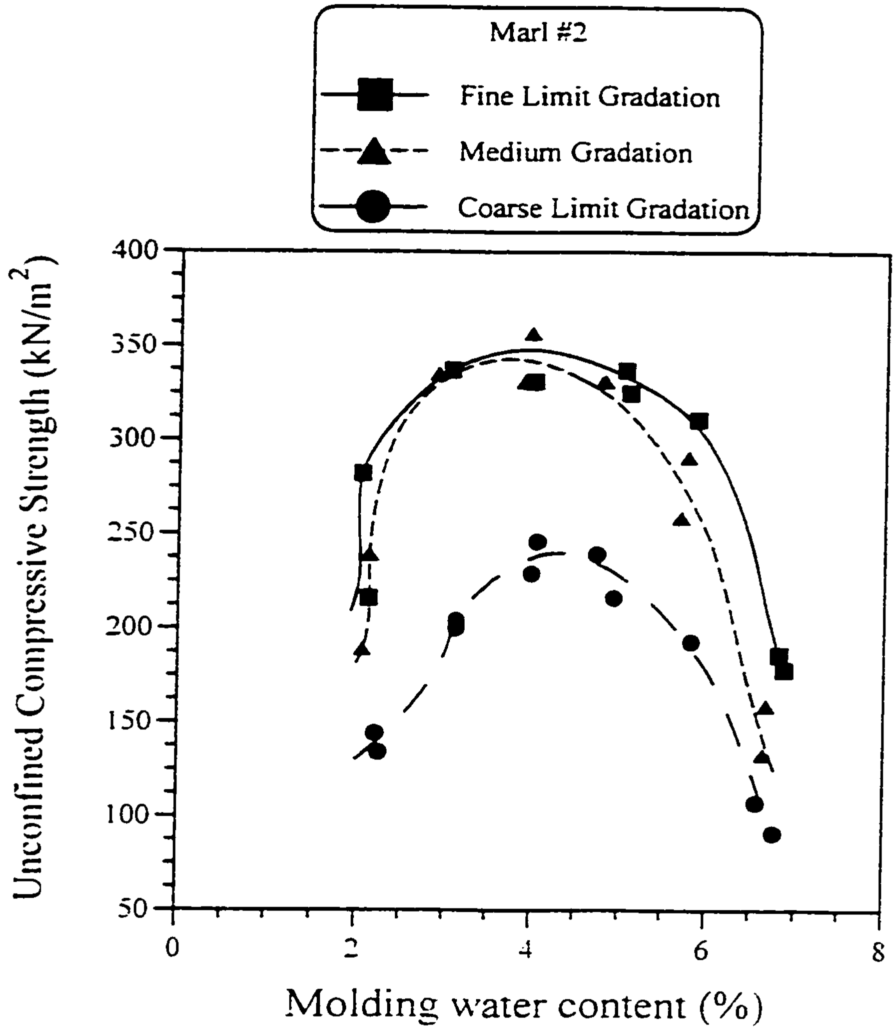


Figure 4.28: UCS-moisture relationships of the three gradations for marl #2

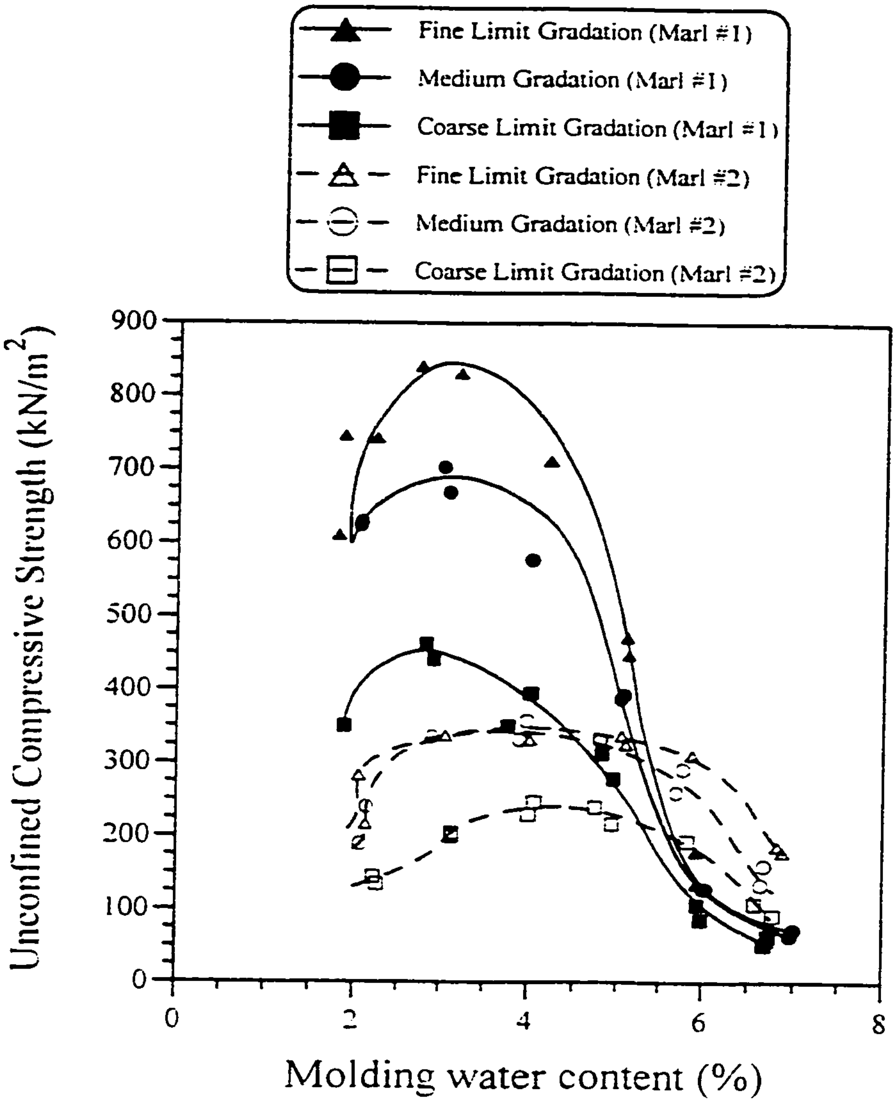


Figure 4.29: UCS-moisture relationships of the three gradations for both marls

the curves are observed to be different, for example, Marl #1 shows unsymmetrical curves with remarkable peaks, while Marl #2 curves are somewhat symmetrical and more flat. Furthermore, it is observed that the maximum UCS values for Marl #1 were obtained at lower moisture contents values compared to those obtained for Marl #2. Although a pair of specimens was tested, for each moisture content, some discrepancies appeared between the UCS results. Therefore, the best fitting UCS curves were obtained using the results produced from all specimens, for each gradation.

4.2.4 Clegg Hammer Tests

Clegg Hammer test was performed on the three gradations, in order to obtain another strength parameter, which can correlate with the CBR values. In general, reliable correlations between the CIV and the CBR values are expected since the two tests are usually performed under typical testing conditions using the same specimen.

The CIV-moisture-density relationships for Marl #1 are shown in Figs. 4.30 to 4.32, for the three gradations. All samples show that the maximum CIV values are either at the optimum or on the dry side of optimum. For the fine limit gradation the CIV_{max} was attained at moisture content of 4.9%. For specimens reconstituted to the medium gradation the CIV_{max} was attained at moisture content of 4.1%. For the set, which was reconstituted to the coarse limit gradation the CIV_{max} was achieved at moisture content of 3.8%. The maximum dry density, optimum moisture content and maximum CIV values, for the three gradations, are summarized in Table 4.4.

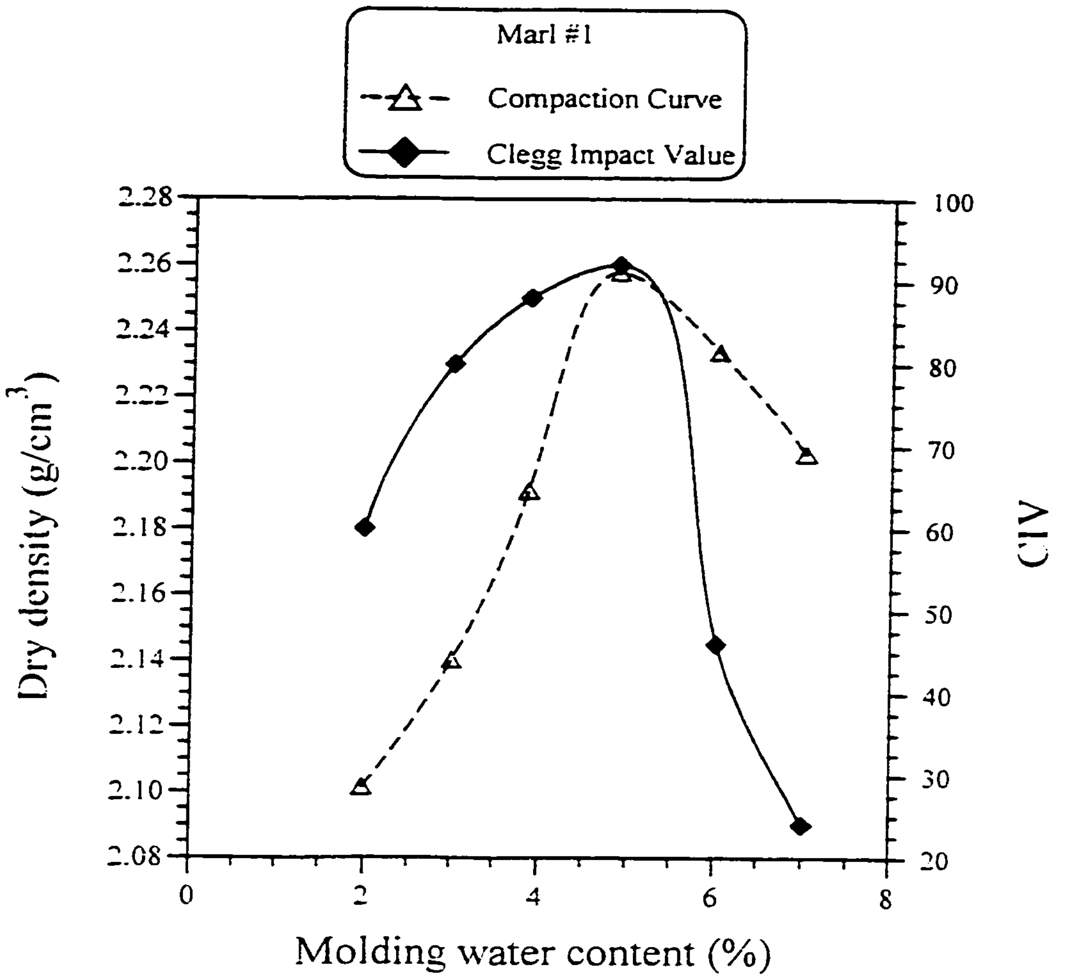


Figure 4.30: CIV-moisture-density relationships of the fine limit gradation for marl #1

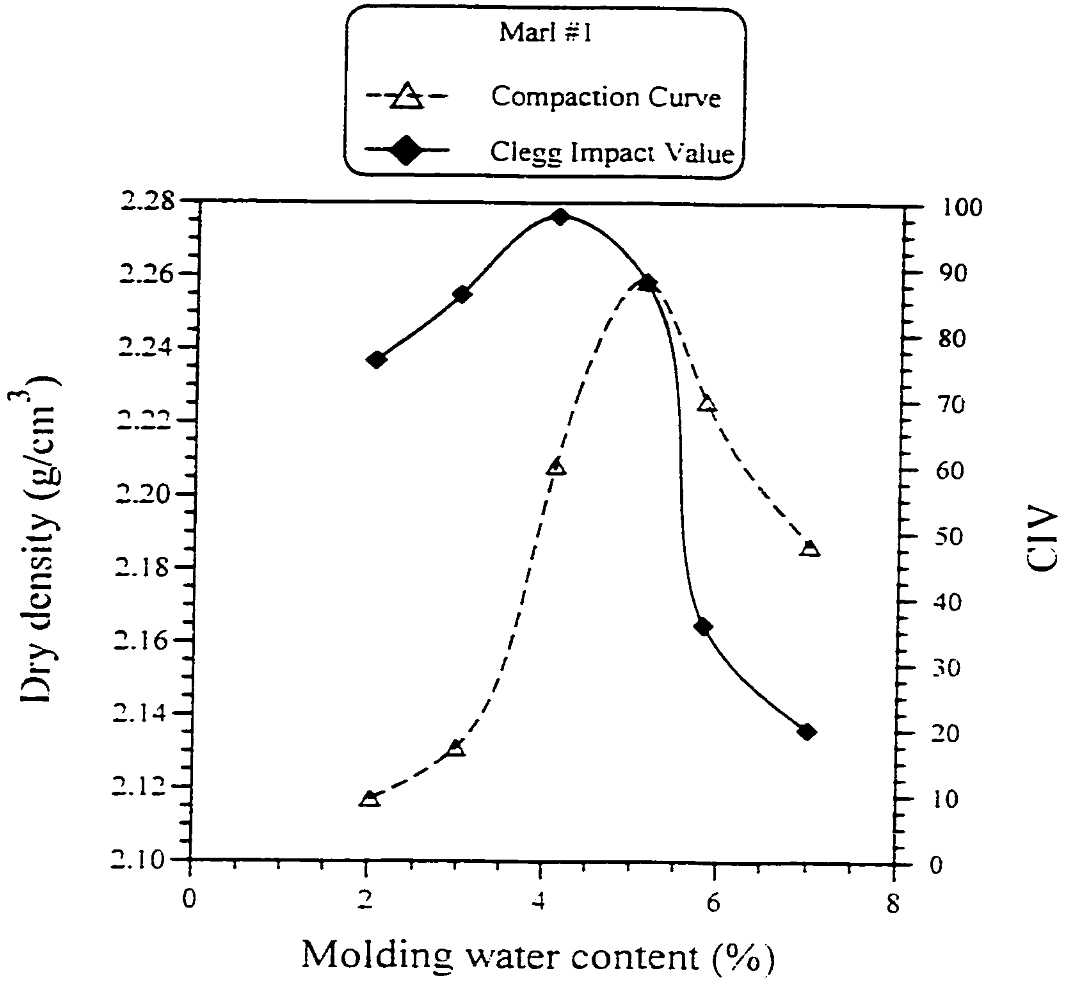


Figure 4.31: CIV-moisture-density relationships of the medium gradation for marl #1

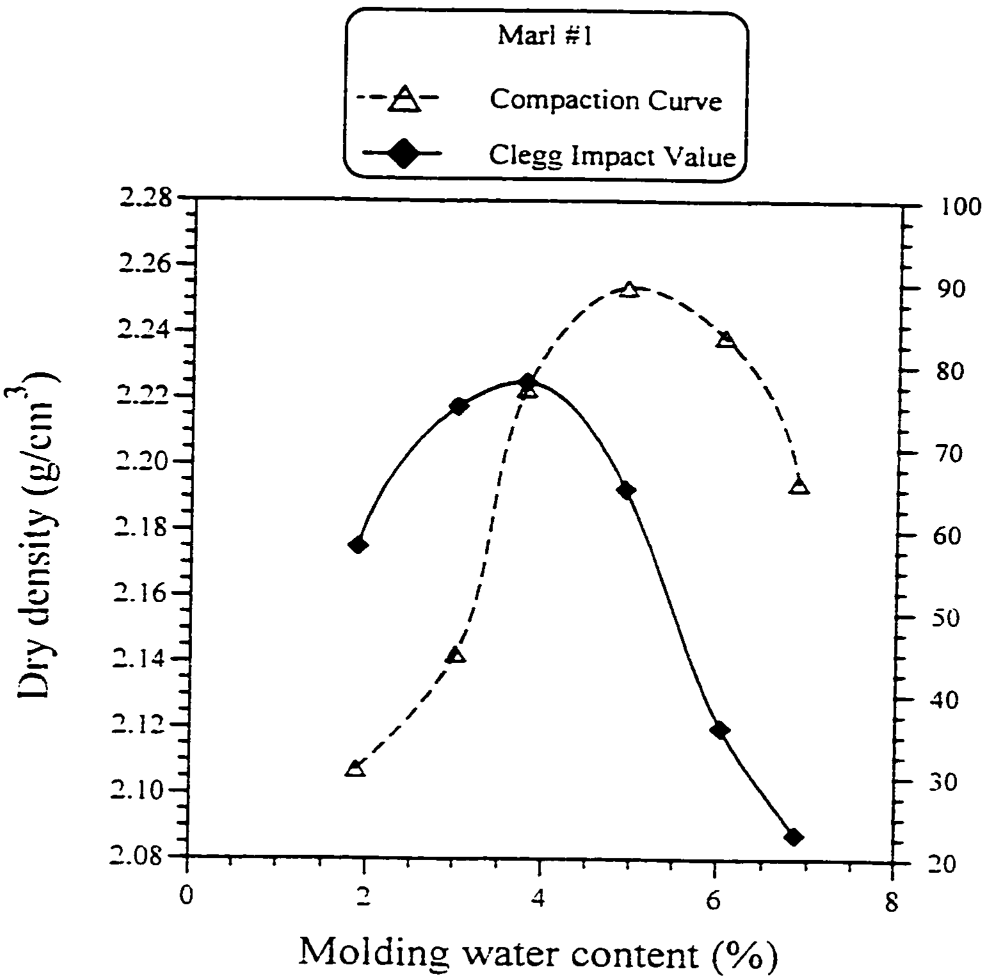


Figure 4.32: CIV-moisture-density relationships of the coarse limit gradation for marl #1

Table 4.4: The γ_{dmax} , w_{opt} and CIV_{max} values of the two marls for different gradations

Gradation	**Marl #1**			**Marl #2**		
	γ_{dmax} (gm/cm^3)	w_{opt}	CIV_{max}	γ_{dmax} (gm/cm^3)	w_{opt}	CIV_{max}
Fine Limit Gradation	2.26	4.9	92	2.22	6.1	92
Medium Gradation	2.26	5.1	98	2.24	5.3	89
Coarse Limit Gradation	2.25	4.9	78	2.22	5.5	94

The CIV curves for Marl #1 for the three gradations are shown in Fig. 4.33. It is clear that the medium gradation gave the highest maximum CIV value, while the coarse limit gradation gave the lowest maximum CIV value. The trend of CIV variation is similar to that shown for the unsoaked CBR. This substantiates the strong correlation between the two tests.

The CIV-moisture-density relationships, of the three gradations for Marl #2, are shown in Figs. 4.34 to 4.36. All sets of samples show that the maximum CIV values were observed to be on the dry side of optimum, which is similar to the unsoaked CBR curves. For the fine limit gradation the CIV_{max} was obtained at moisture content of 5.1%. For specimens reconstituted to the medium gradation the CIV_{max} was attained at moisture content of 5.1%. For the third set of specimens, where samples were reconstituted to the coarse limit gradation, the CIV_{max} was obtained at moisture content of 4.5%. Therefore, for all gradations, the CIV_{max} was attained at dry densities less than the maximum dry densities and on the dry side of optimum as shown in Table 4.4.

For marl #2, the maximum CIV value was obtained for the coarse limit gradation and the lowest maximum CIV value was obtained for the medium gradation, as shown in Fig. 4.37. However, the difference between the CIV values for the three gradations is very small. The variation of the CIV values with moisture content for the three gradations is similar to that of the dry density. Comparing CIV curves for the two marls as shown in Fig. 4.38, it is clear that Marl #1 shows higher CIV values on the dry side of optimum compared to Marl #2. This is reversed on the wet side of optimum when Marl #2 shows

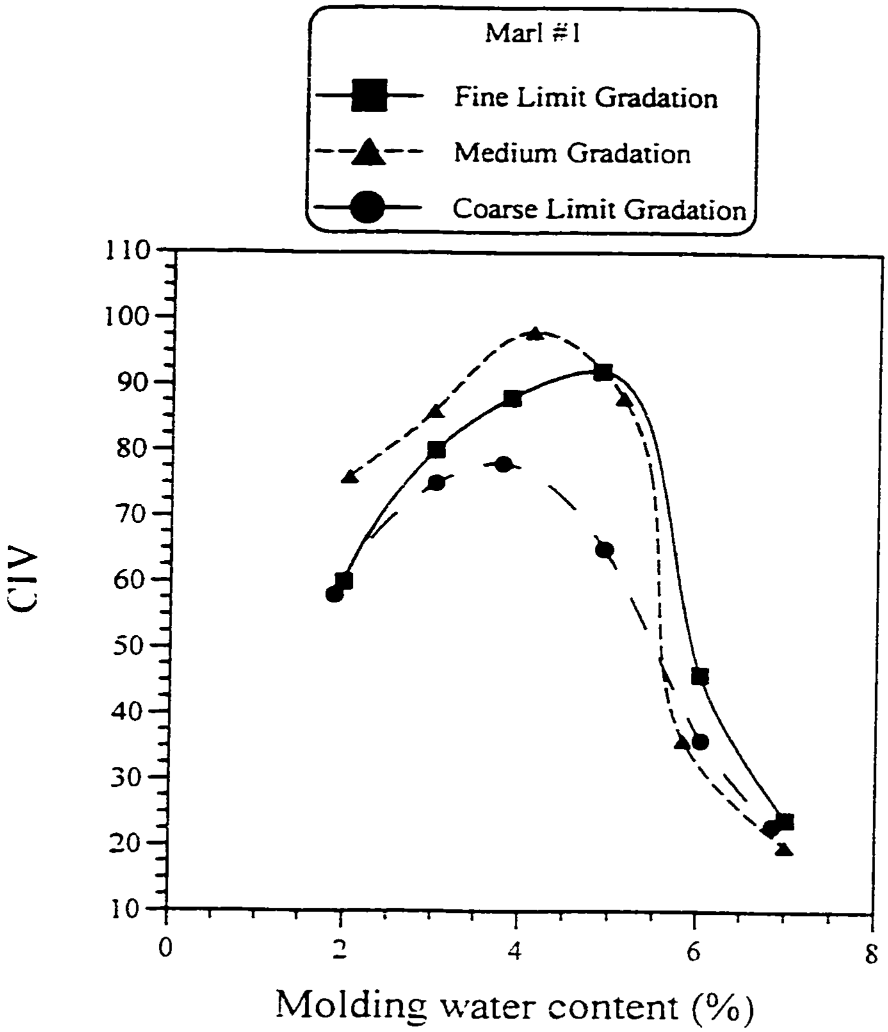


Figure 4.33: CIV-moisture relationships of the three gradations for marl #1

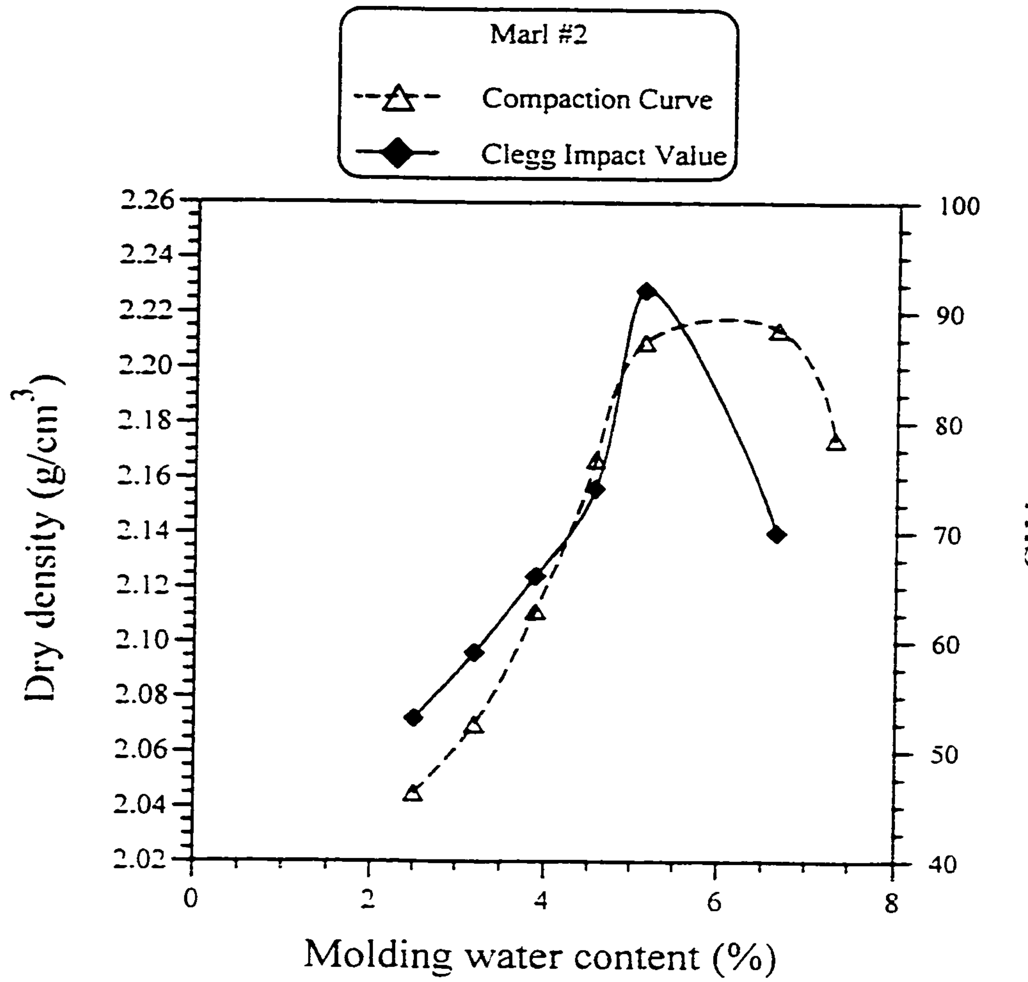


Figure 4.34: CIV-moisture-density relationships of the fine limit gradation for marl #2

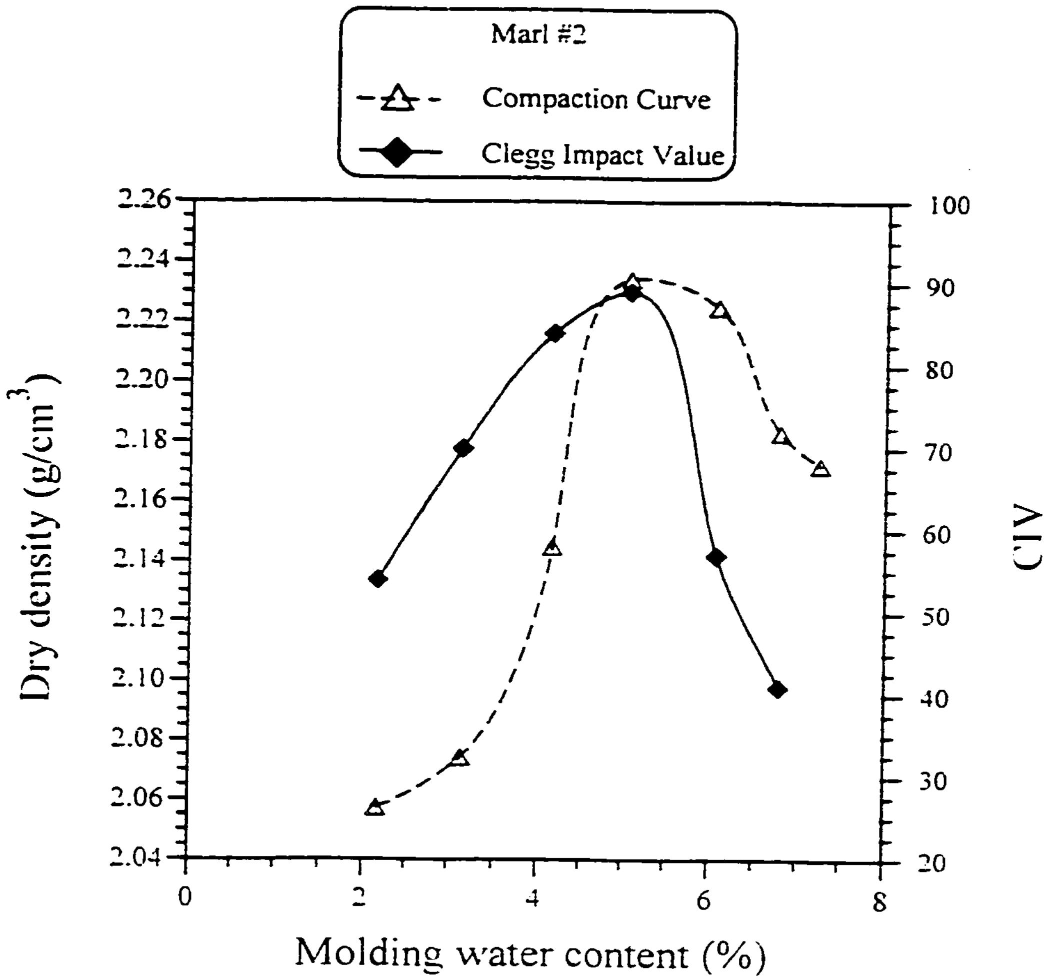


Figure 4.35: CIV-moisture-density relationships of the medium gradation for marl #2

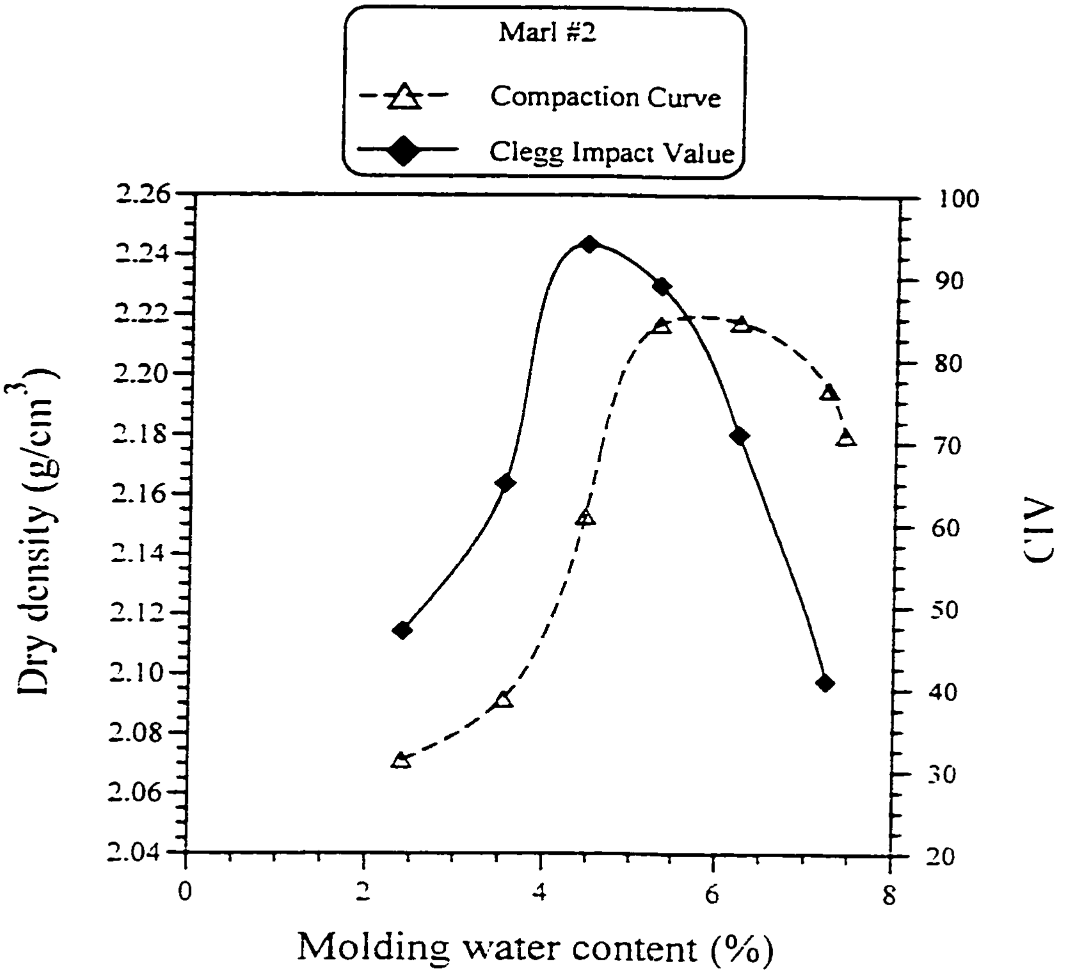


Figure 4.36: CIV-moisture-density relationships of the coarse limit gradation for marl #2

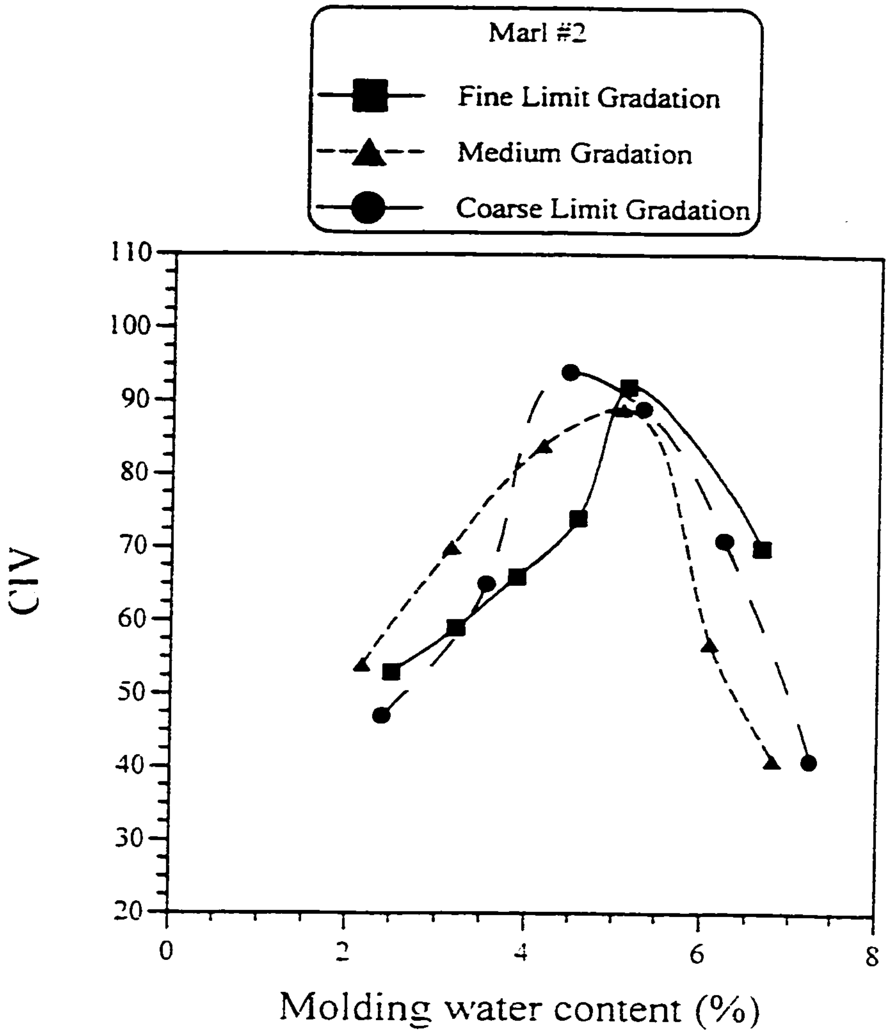


Figure 4.37: CIV-moisture relationships of the three gradations for marl #2

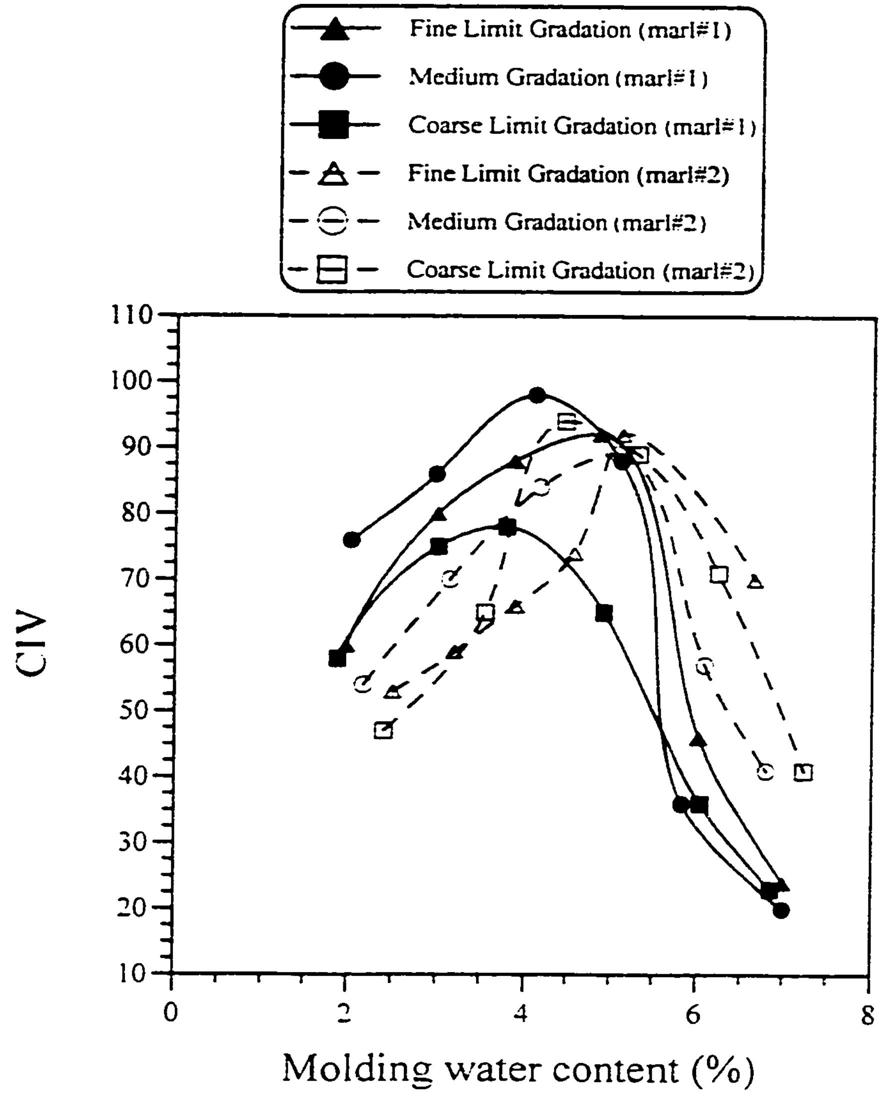


Figure 4.38: CIV-moisture relationships of the three gradations for both marls

higher CIV values. This phenomenon was observed also for the CBR and the UCS curves. Hence Marl #1 shows higher strength characteristics, compared to Marl #2, on the dry side of optimum. While Marl #2 shows higher strength characteristics on the wet side of the optimum moisture contents. This is attributed to the plastic behavior of Marl #1 (as shown in the characterization tests), which is absent in Marl #2. According to Aiban (1995), the cohesion of calcareous soils is highly dependent on the molding water content. Hence, a remarkable decrease in the strength of cohesive or plastic calcareous soils is observed with the increase of water content. In addition, as noticed for the CBR and the UCS tests, the curves for different gradations tend to merge close to each other at the very dry and the very wet sides of optimum.

In general, the maximum CIV and the maximum CBR values were obtained at the same dry density and moisture content values. However, there are some exceptions to this phenomenon and this is attributed to the differences between the procedures of the two tests despite their strong correlation.

4.3 Correlations Between the Strength Parameters

It is usually recommended to perform different types of tests in order to assess the strength properties of a certain soil. This will help in obtaining more reliable results and will compensate for any deficiency, which may be associated with certain type of testing procedures. In this study, the load carrying capacity for the different gradations was assessed using three types of tests, namely the California Bearing Ratio (CBR) Test, Unconfined Compressive Strength (UCS) Test and Clegg Hammer Test. The means by which the sample is loaded is different from one test to another. For example, a static load with a fixed rate is used for the CBR and UCS tests while a dynamic (impact) loading is used for the Clegg hammer test.

For both marls, the CBR, UCS, CIV curves were obtained for the three selected gradations. In addition, combinations of curves were obtained by superimposing each pair of curves representing two different strength parameters together for the same gradation. The correlations between different parameters were plotted using the values corresponding to certain moisture content values.

4.3.1 CBR-UCS Correlation

The CBR-UCS-moisture content curves for the different gradations, for each marl are shown in Figs. 4.39 to 4.44. Results clearly show that, the peak CBR and UCS values were observed to be at different moisture contents. Generally, the maximum UCS values were attained at moisture contents that are less than those at which the maximum CBR values were attained. In addition, it is observed that, for each pair of curves, the general trend of CBR-moisture content and UCS-moisture content variation is similar. The general trend of both curves are the increase of the strength until a peak value, at certain moisture content, is reached then the strength starts to drop.

The CBR-UCS correlations, obtained using the conventional mold for the three gradations, for each marl, are shown in Figs. 4.45 and 4.46. As shown in the figures, Marl #1 gave good correlation, while the data for Marl #2 are highly scattered and did not give reasonable correlation. For Marl #2, the samples gave high CBR values especially with

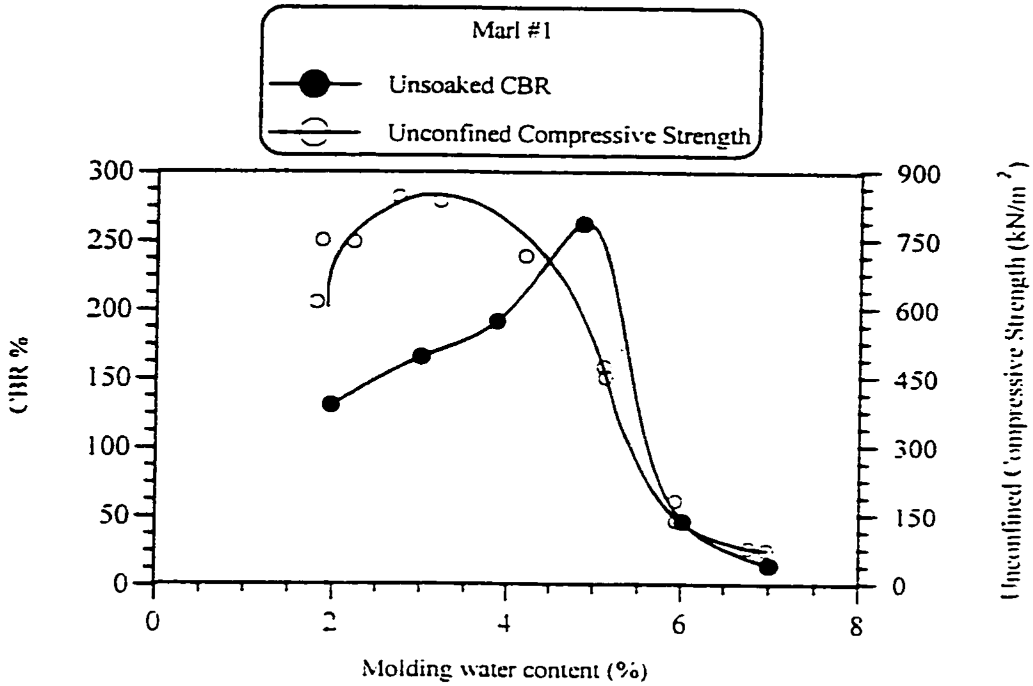


Figure 4.39: CBR-UCS-moisture relationships of the fine limit gradation for marl #1

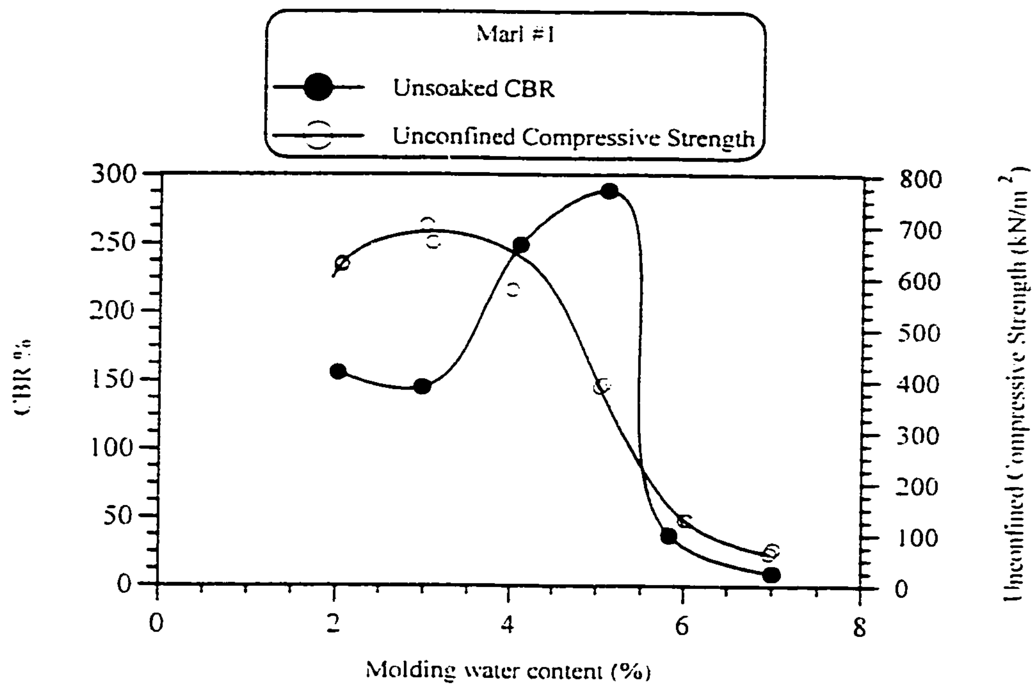


Figure 4.40: CBR-UCS-moisture relationships of the medium gradation for marl #1

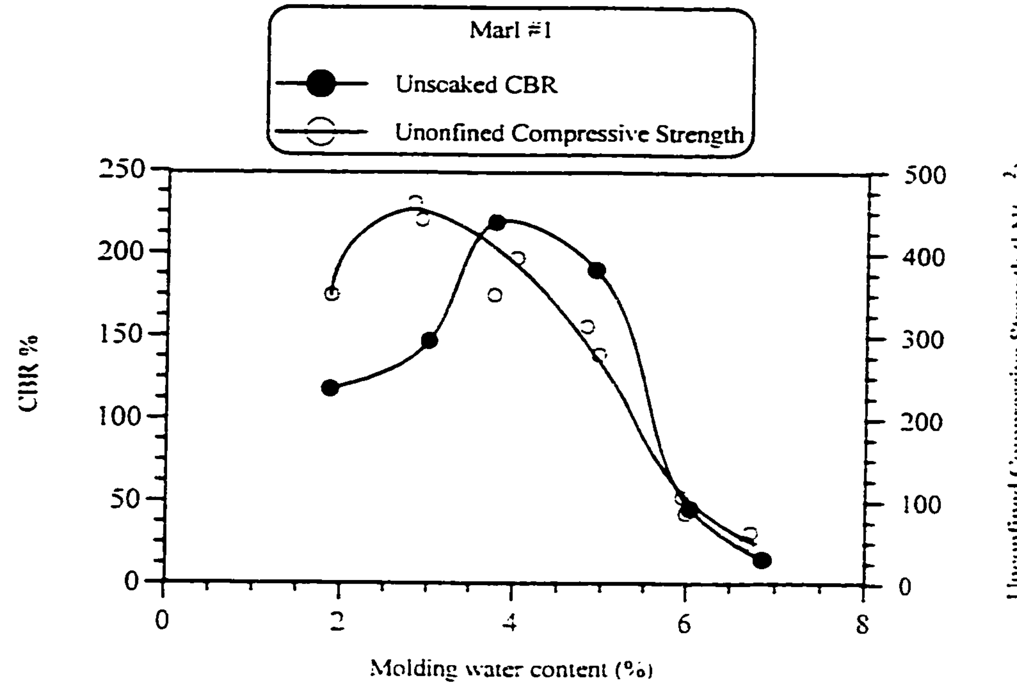


Figure 4.41: CBR-UCS-moisture relationships of the coarse limit gradation for marl #1

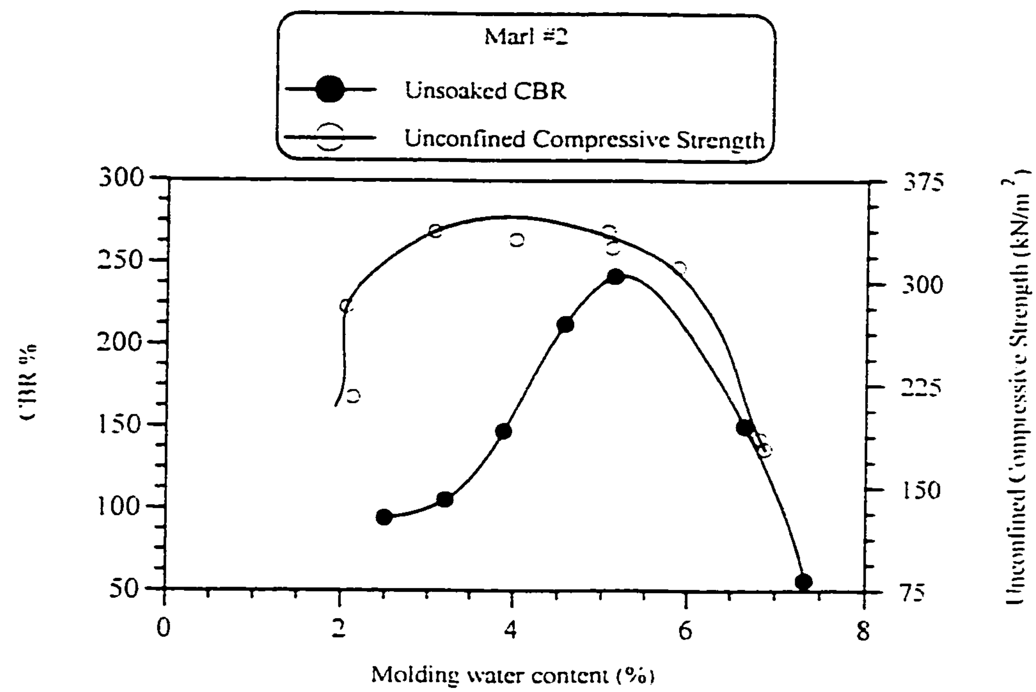


Figure 4.42: CBR-UCS-moisture relationships of the fine limit gradation for marl #2

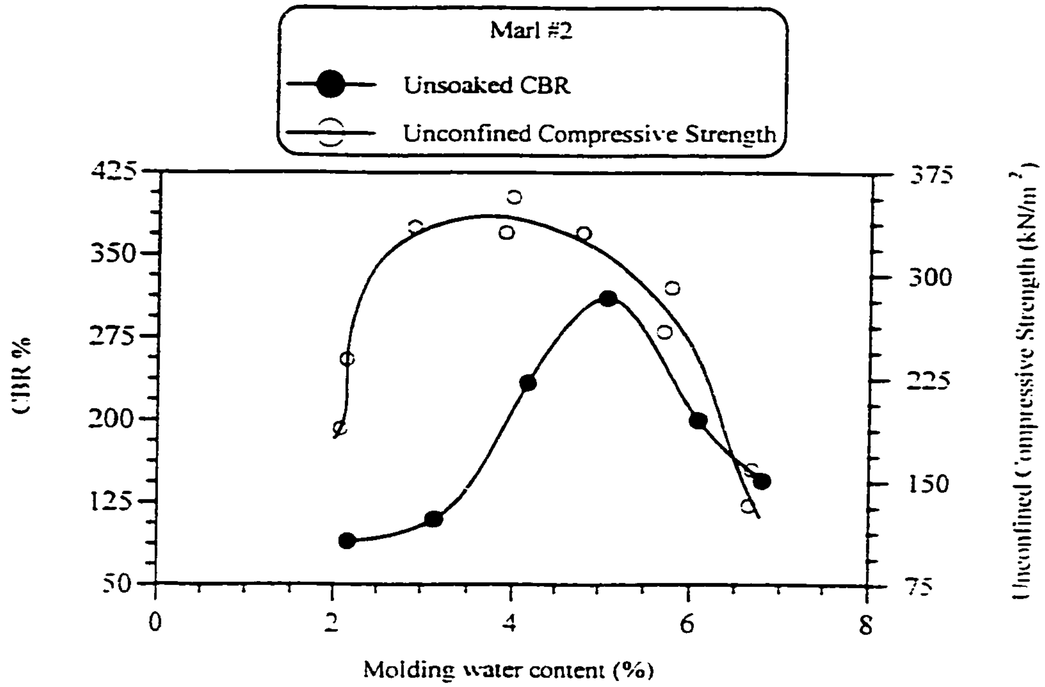


Figure 4.43: CBR-UCS-moisture relationships of the medium gradation for marl #2

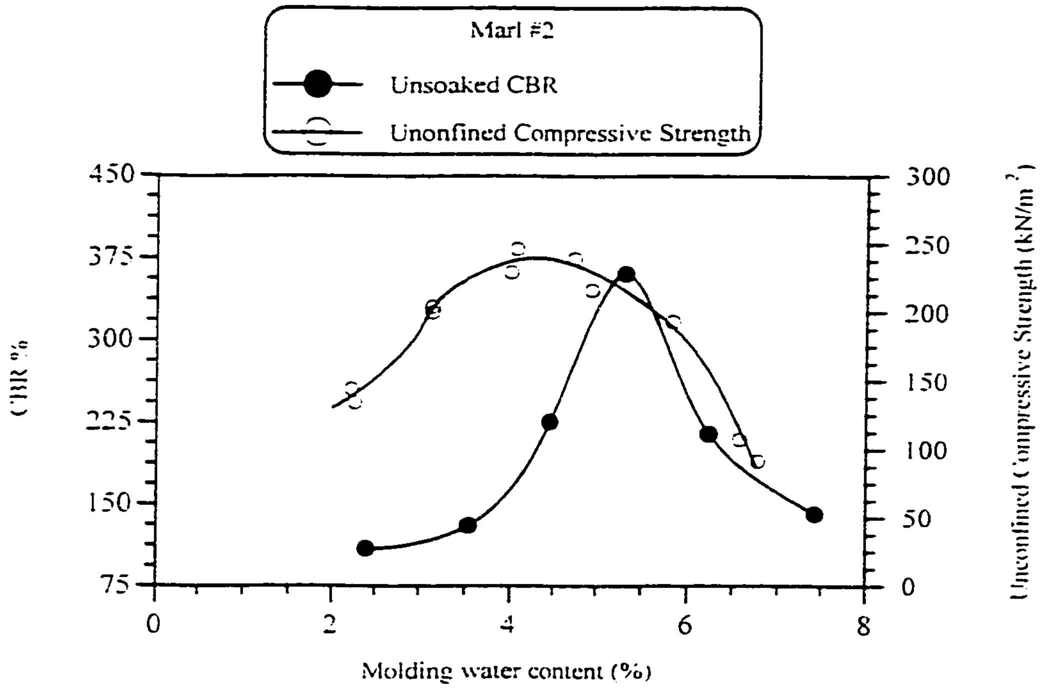


Figure 4.44: CBR-UCS-moisture relationships of the coarse limit gradation for marl #2

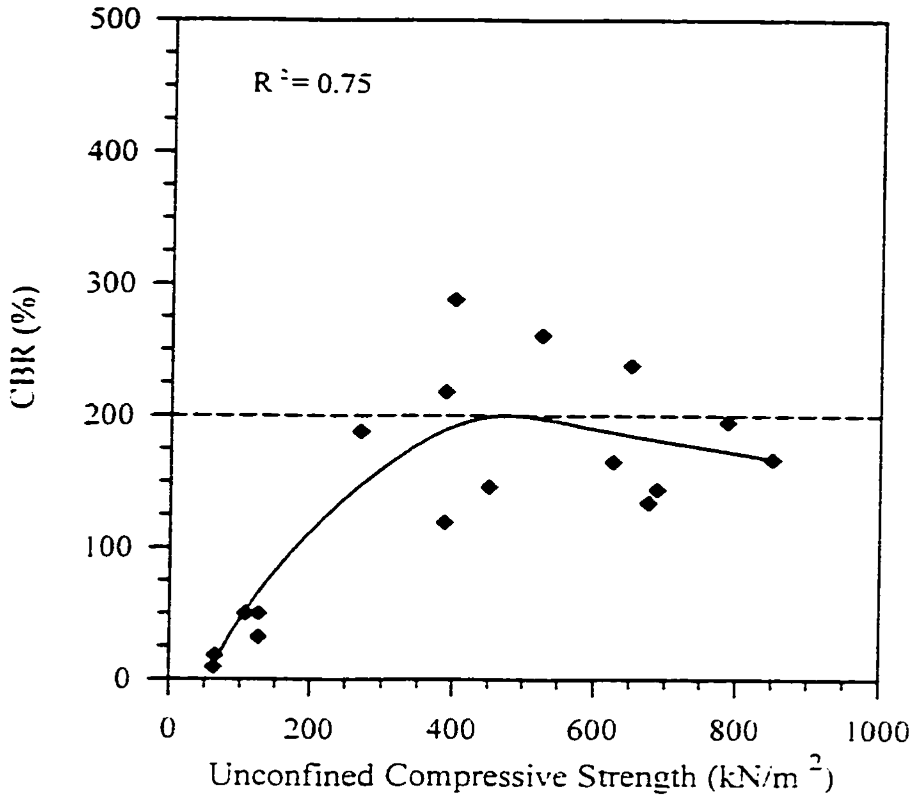


Figure 4.45: The correlation between the CBR and the UCS values for Marl #1

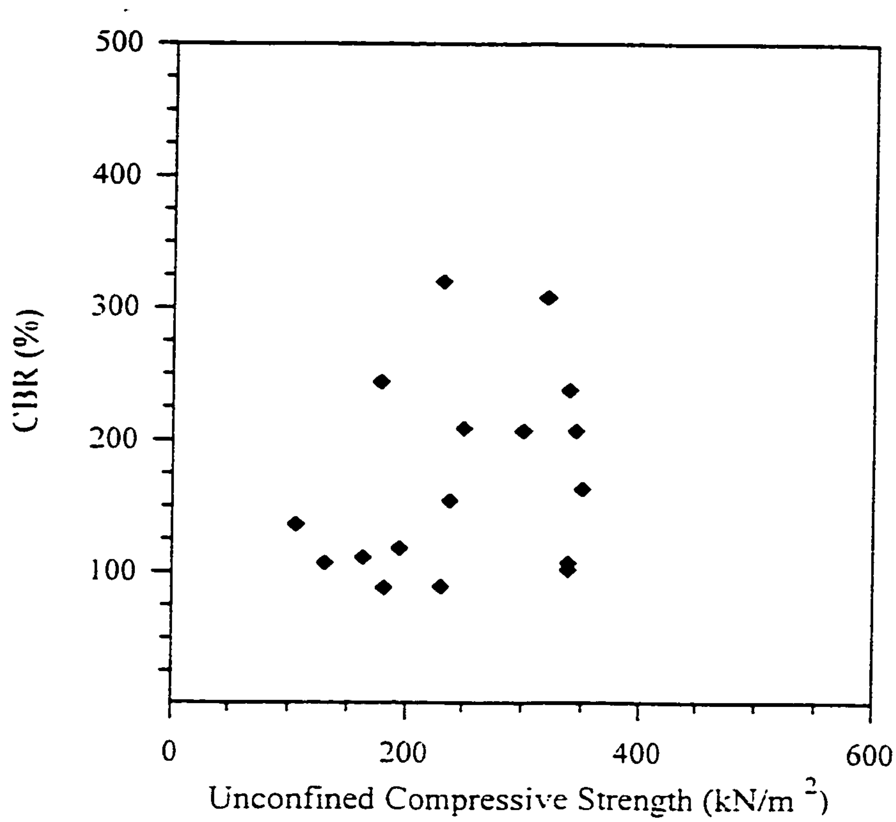


Figure 4.46: The relation between the CBR and the UCS values for Marl #2

the presence of high percentage of stony particles and this is attributed to the good quality of the aggregates. While the UCS values correspond to the same samples will be relatively low as a result of the low amount of fines and their lack of plasticity.

As shown in Fig. 4.45, the CBR values are proportional to the UCS values till a CBR value of 200% is reached. After this limit the UCS values started to decrease with further increase in the CBR values. The correlation between the CBR and the UCS for the two marls, using the three gradations is shown in Fig. 4.47. As shown in the figure, the CBR values increase with the increase of the UCS values till a certain optimum CBR limit of about 204% is reached. After this value the CBR and the UCS started to have an inversely proportional relation i.e. the CBR values started to decrease with further increase in the UCS values.

Furthermore, when correlating the UCS values with the CBR values obtained from the large-size mold, for the medium gradation, the correlation was reversed at a CBR of about 108% as shown in Fig. 4.48. Considering the UCS as a reference, and comparing the limits at which the CBR-UCS proportional relation was reversed, it is clear that a CBR of 200% using the conventional mold is equivalent to a CBR of about 100% using the large mold. Hence, CBR values greater than 100%, based on the large mold, may have questionable practicality, since the UCS may decrease beyond this value, for marls with low cohesion. Generally, the CBR increases with the increase of aggregate content as discussed before, while the high maximum UCS values were obtained for the fine gradations for both marls. Hence, with an excess amount of aggregates the CBR values

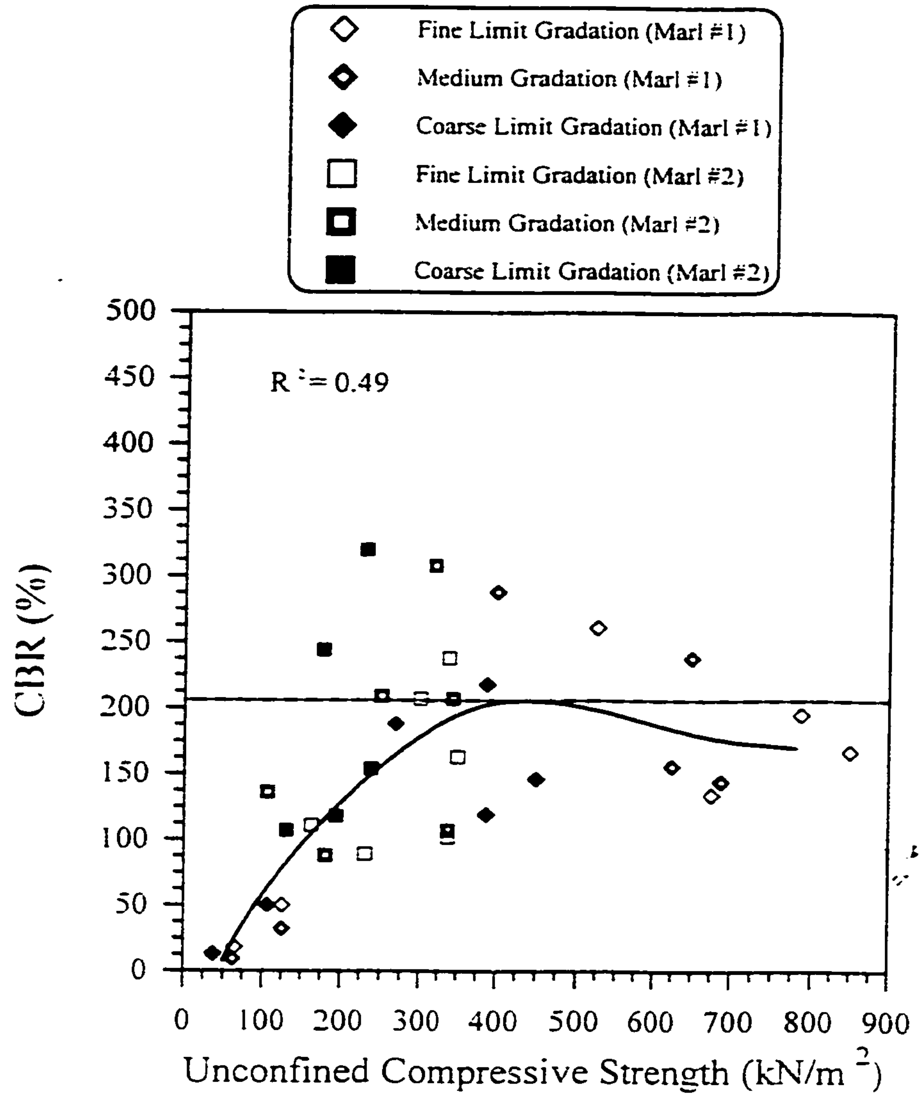


Figure 4.47: The correlation between the CBR and the UCS values for both marls

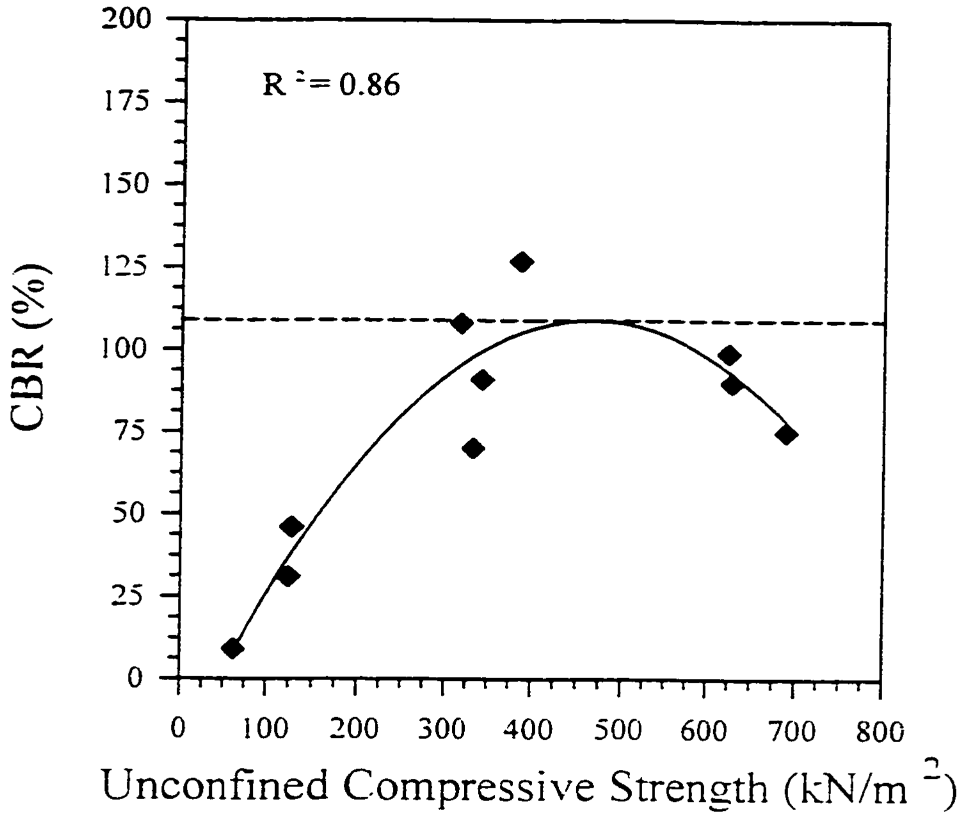


Figure 4.48: The correlation between the large mold CBR and the UCS for the medium gradation for both marls

tend to increase while the UCS values tend to decrease because of the lack of binding between the coarse grains especially with the absence of enough plastic fines to fill the voids between the aggregates. Therefore, the CBR-UCS correlation may not be appropriate for material with low cohesion, especially when it has coarse gradations.

4.3.2 CBR-CIV Correlation

The CBR-CIV-moisture content curves, obtained using the conventional mold, for the different gradations, for Marl #1 and Marl #2 are shown in Figs. 4.49 to 4.54, respectively. Generally, for each pair of curves the maximum CBR and CIV values were attained at almost the same moisture content except for only two pairs, where the maximum CIV values occurred at lower moisture contents than those for the maximum CBR values. A clear similarity between the trends of each pair was observed, and this indicates that a strong correlation between the two strength parameters is expected. The correlation between the CBR and the CIV for the two marls using the three gradations is shown in Fig. 4.55. The equation obtained from this correlation is given by:

$$CBR = 0.0587837 \times CIV^{1.86088}$$

or

$$CIV = 4.5854 \times CBR^{0.5374}$$

The CBR-CIV relations obtained using the medium gradation for both marls, for the small-size and large-size molds, are shown in Figs. 4.56 and 4.57, respectively. The equation obtained from the correlation for the small mold for the medium gradation is given by:

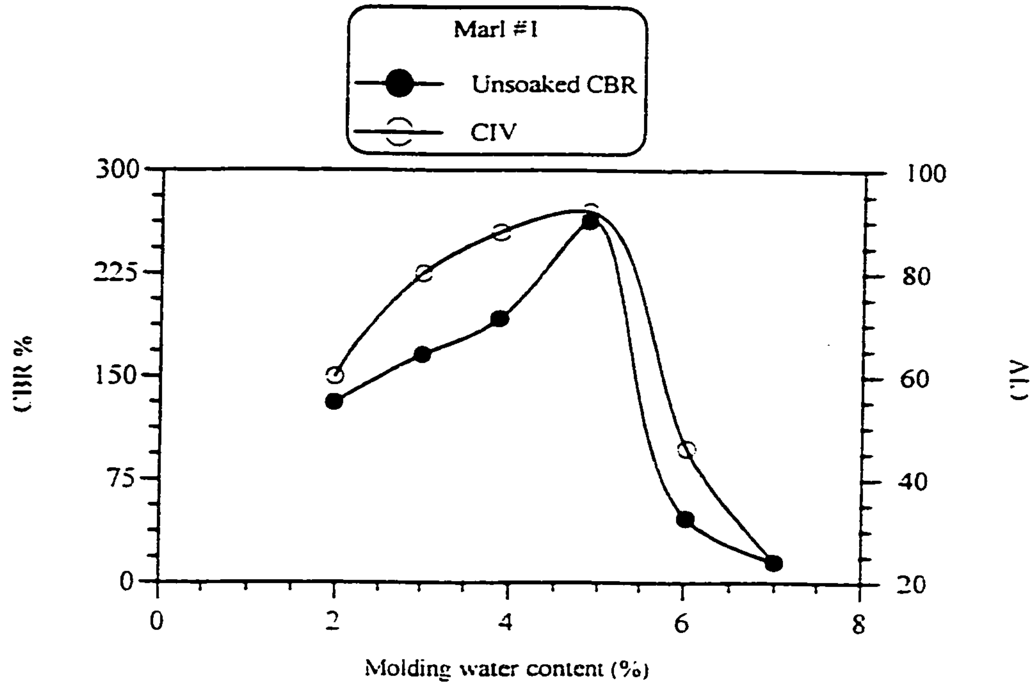


Figure 4.49: CBR-CIV-moisture relationships of the fine limit gradation for marl #1

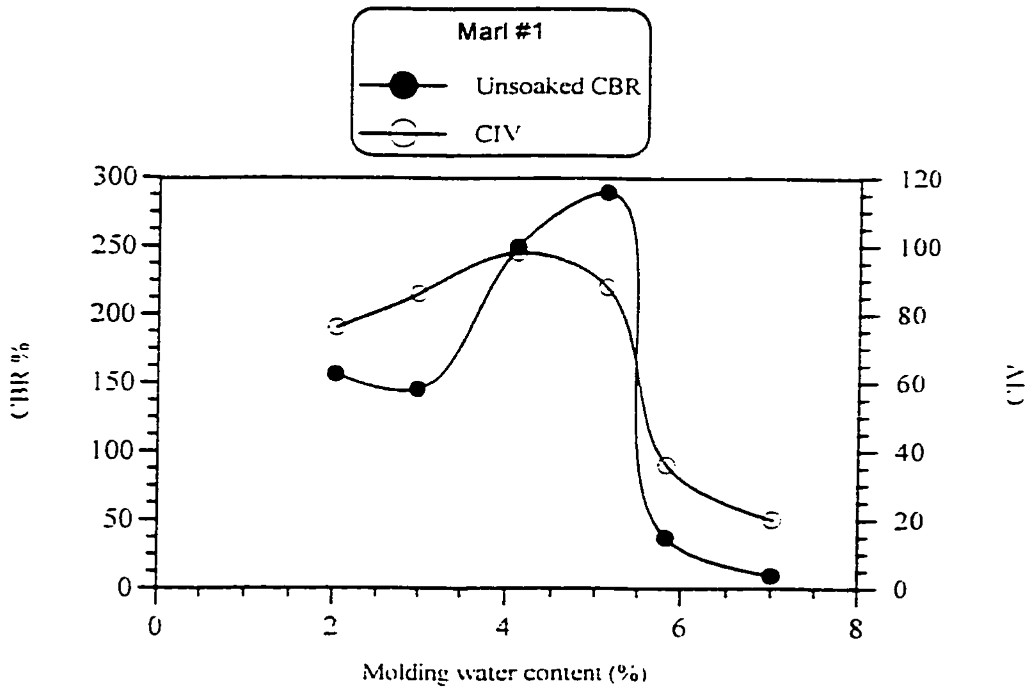


Figure 4.50: CBR-CIV-moisture relationships of the medium gradation for marl #1

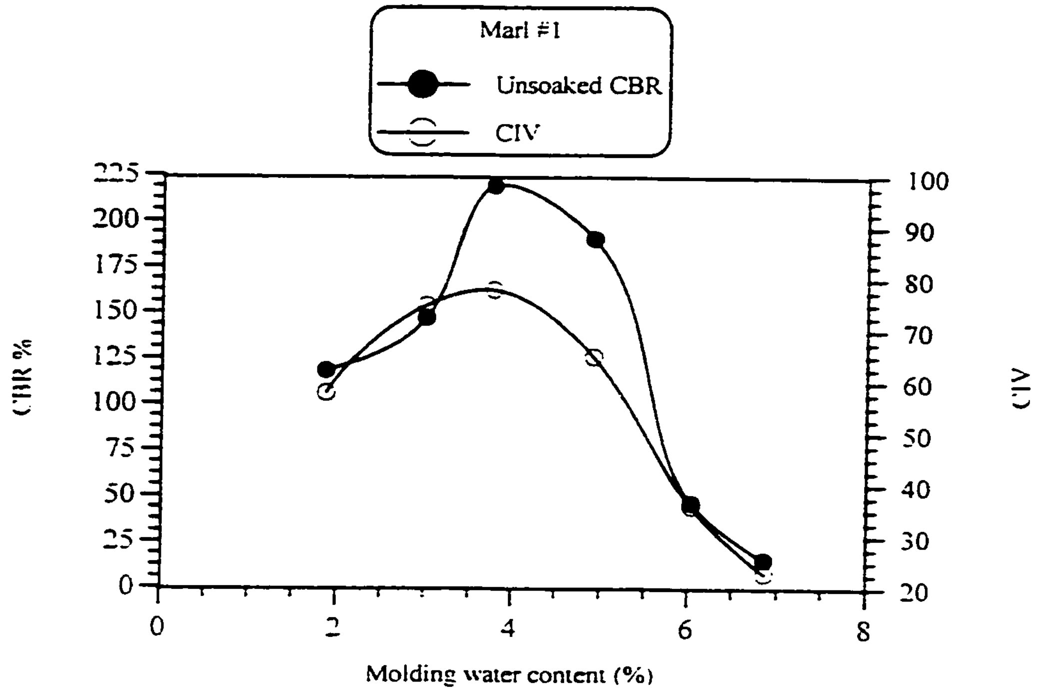


Figure 4.51: CBR-CIV-moisture relationships of the coarse limit gradation for marl #1

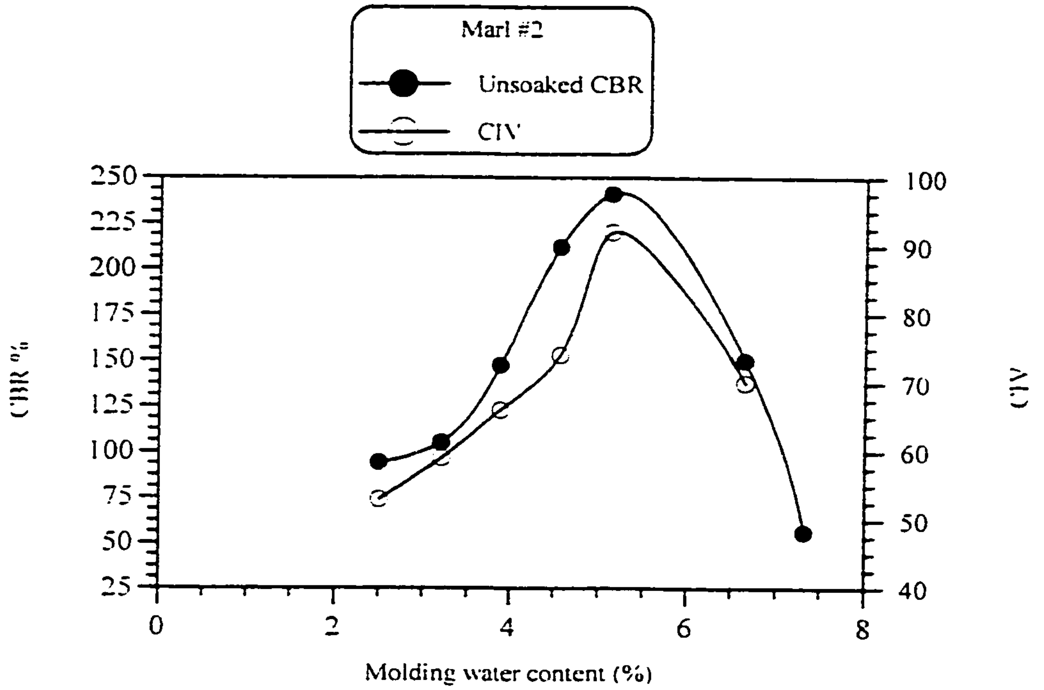


Figure 4.52: CBR-CIV-moisture relationships of the fine limit gradation for marl #2

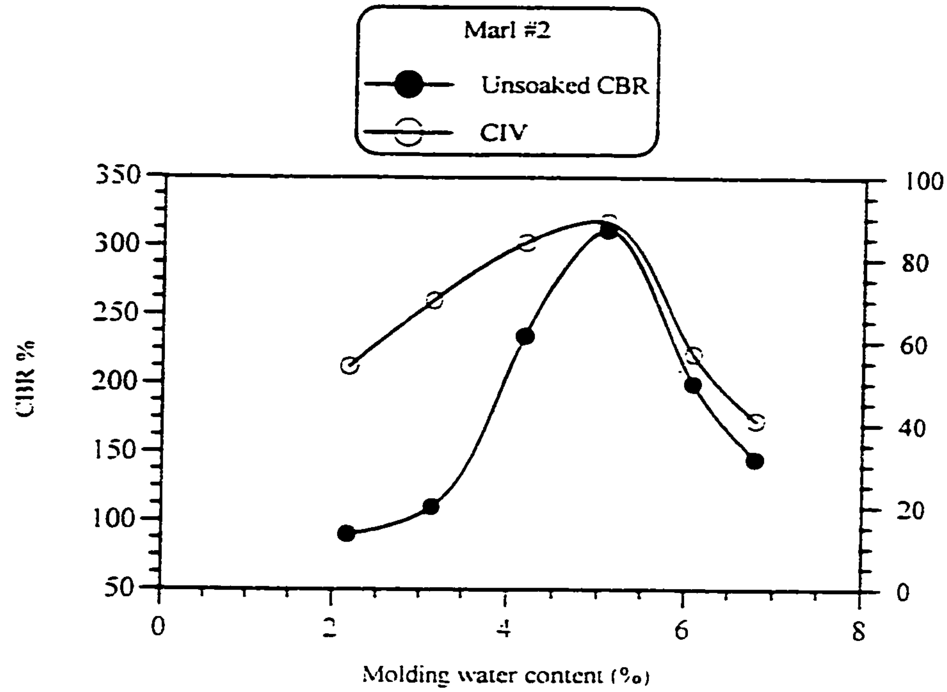


Figure 4.53: CBR-CIV-moisture relationships of the medium gradation for marl #2

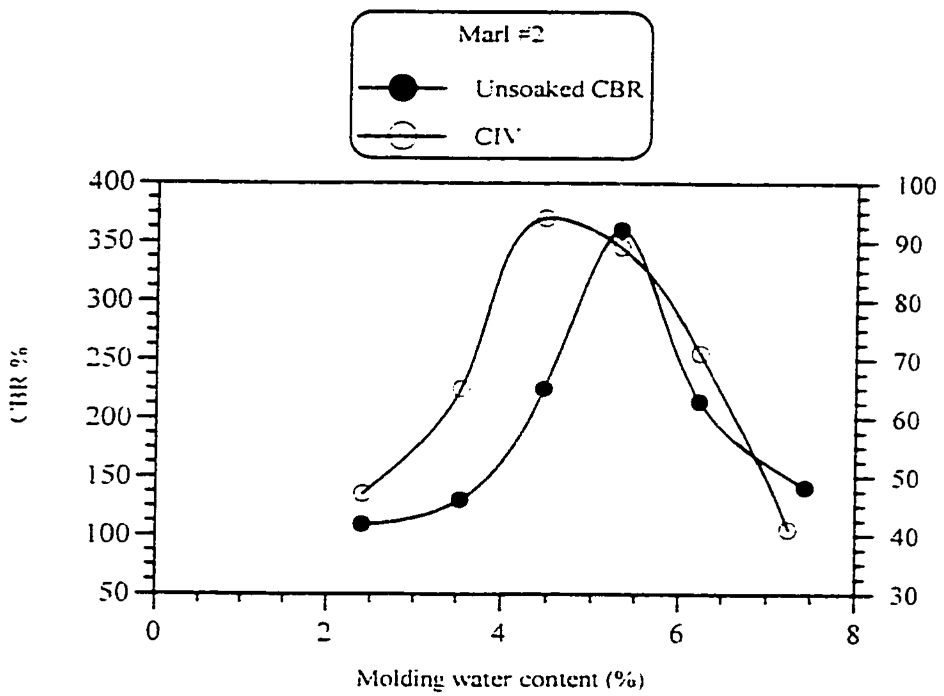


Figure 4.54: CBR-CIV-moisture relationships of the coarse limit gradation for marl #2

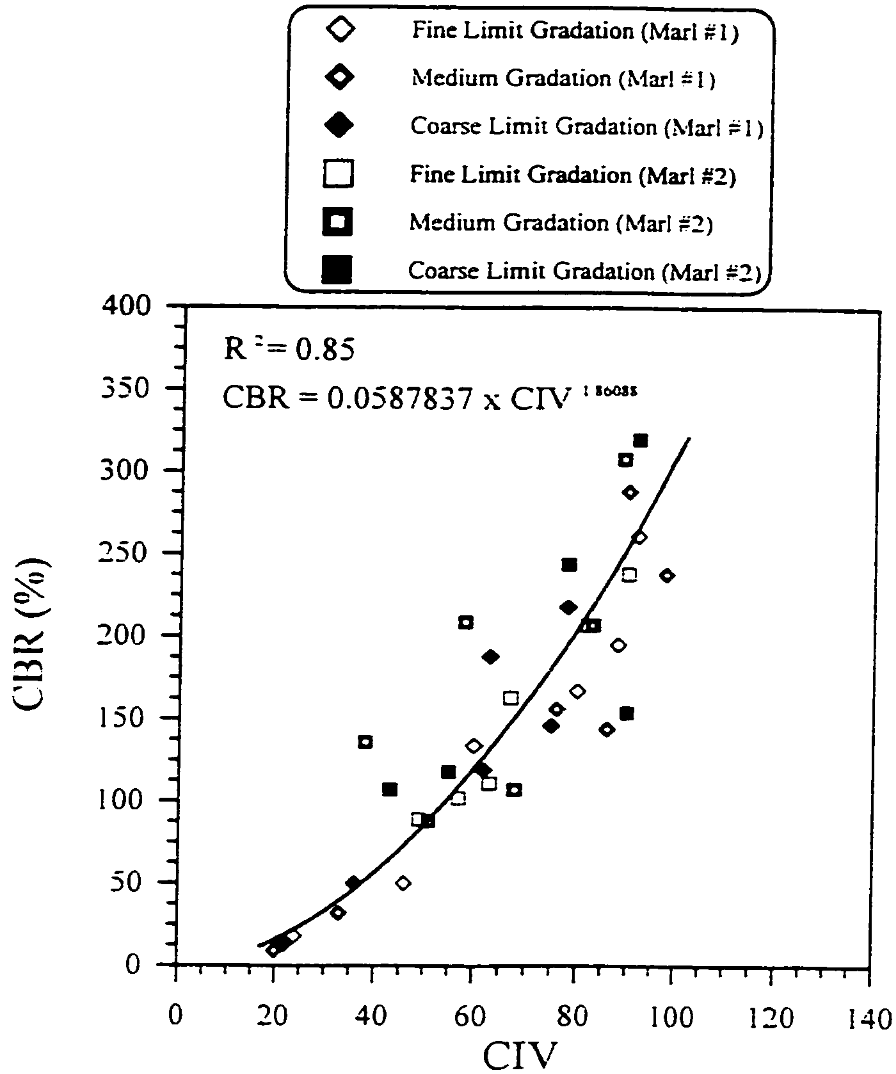


Figure 4.55: The correlation between the unsoaked CBR and the CIV values

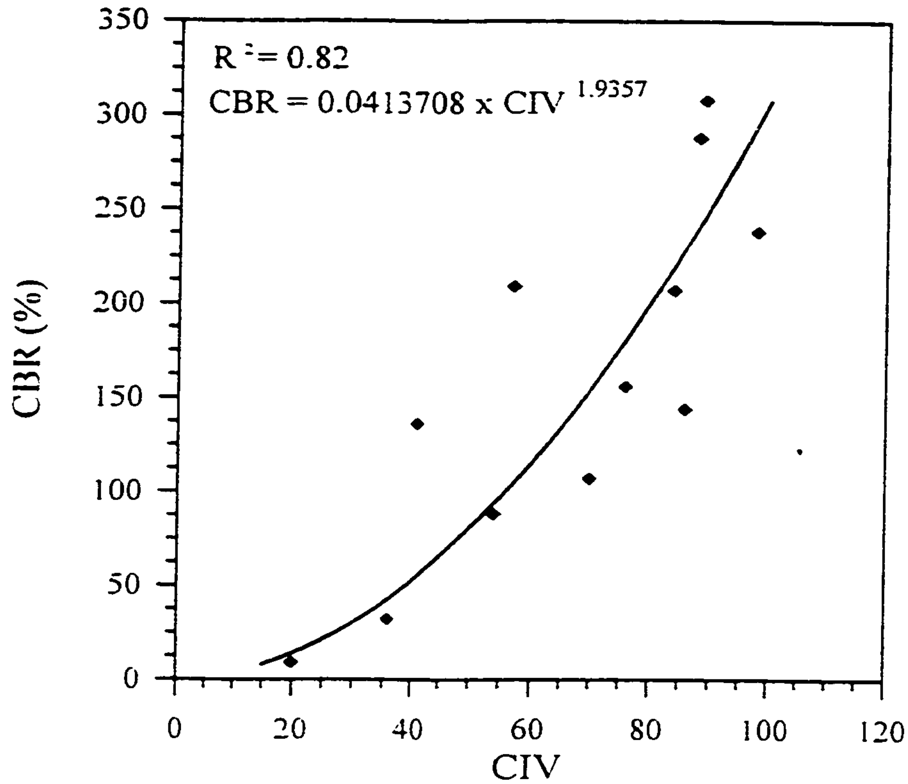


Figure 4.56: The correlation between the unsoaked CBR and the CIV values for the medium gradation using the small mold for both marls

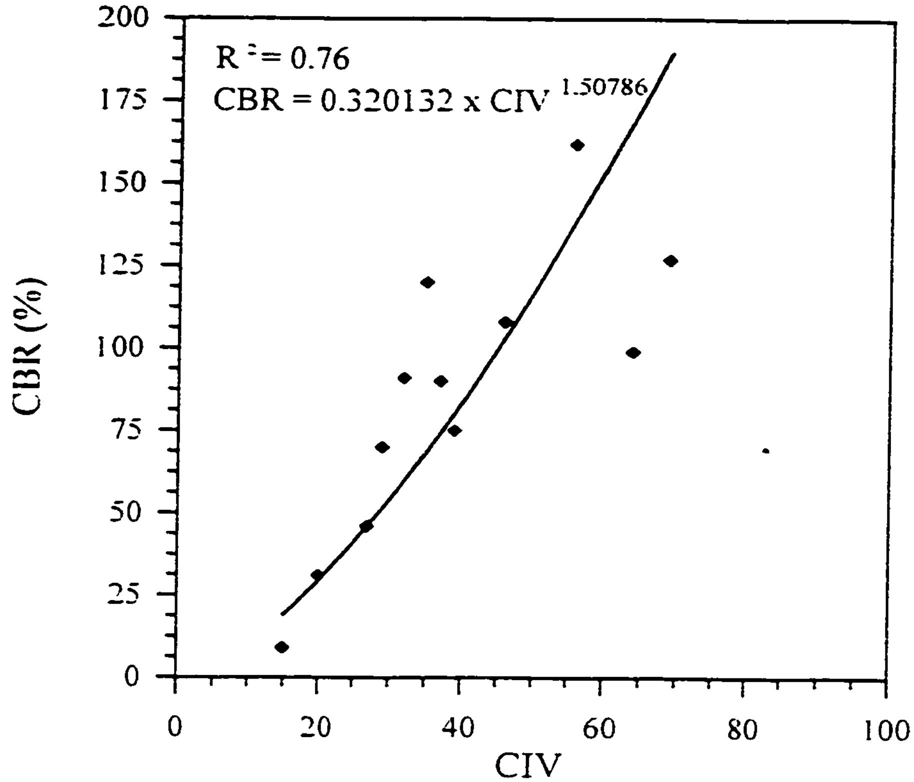


Figure 4.57: The correlation between the unsoaked CBR and the CIV values for the medium gradation using the large mold for both marls

$$CBR = 0.0413708 \text{ x } CIV^{1.9357}$$

or

$$CIV = 5.1836 \text{ x } CBR^{0.5166}$$

While for the large mold the obtained equation for the medium gradation is given by:

$$CBR = 0.320132 \text{ x } CIV^{1.50786}$$

or

$$CIV = 2.1284 \text{ x } CBR^{0.6632}$$

It is clear that the CBR-CIV correlations are not typical for the two molds, even when the samples were reconstituted to the same gradation and tested at approximately equal molding water contents. The variation between the equations is attributed to the differences in samples size and boundary conditions. In order to compare the obtained CBR-CIV correlations, for both mold sizes, the three equations were plotted in Fig. 4.58. As shown in the figure, the correlation obtained for the small mold, using the three gradations and the medium gradation only, are close to each other, hence the medium gradation correlation can be considered representative for the CBR-CIV correlation obtained from the different gradations. In addition it was also observed that the correlation obtained for the large mold is shifted to the left of the correlations obtained for the small mold and almost parallel to them. Hence, for a certain CIV value, the corresponding large mold CBR value is higher than the corresponding small mold CBR value. Therefore, obtaining the field CBR, using the CBR-CIV correlation from the conventional small mold, may not be accurate.

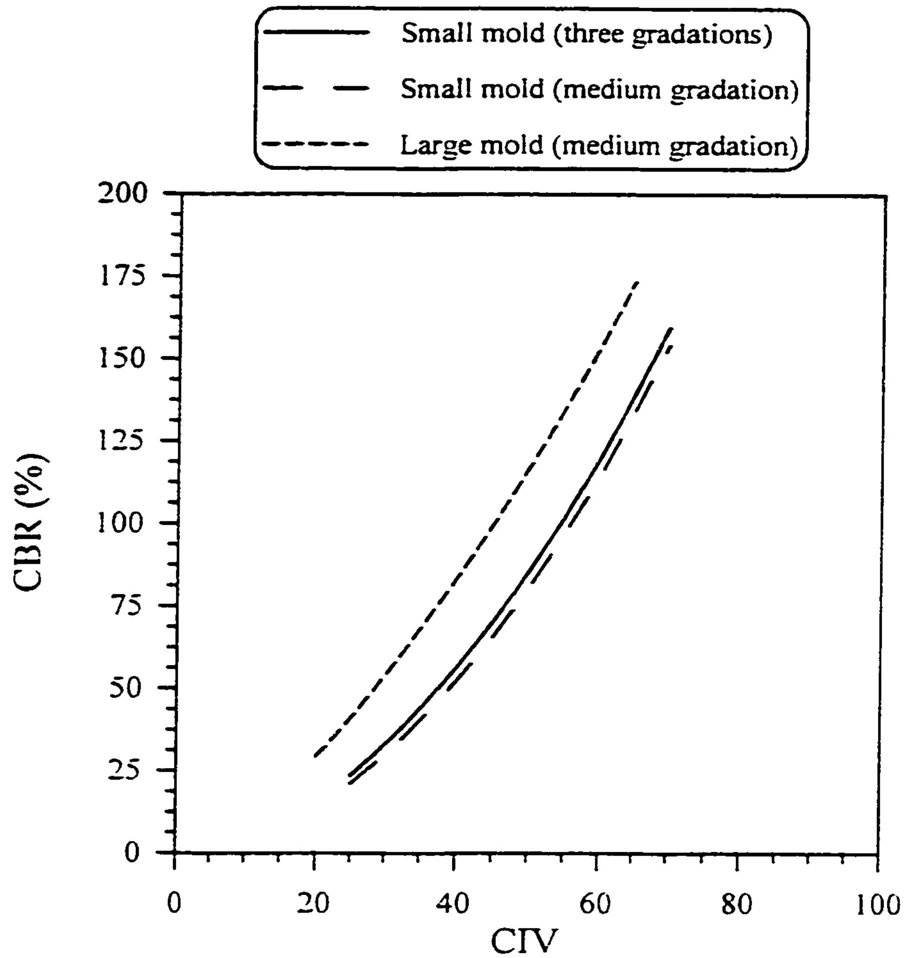


Figure 4.58: Comparison between the CBR-CIV correlations obtained using small and large size molds

4.3.3 CIV-UCS Correlation

The CIV-UCS-moisture content curves for the different gradations, for each marl are shown in Figs. 4.59 to 4.64, obtained using the conventional mold. The peak UCS and CIV values were obtained at different moisture contents. Generally, the maximum UCS values were obtained at moisture contents less than those at which the maximum CIV values were obtained. It is also observed that the trends of curves for each pair are close to each other, which may indicate a possibility of a reasonable correlation. The correlation between the UCS and the CIV for the two marls using the three different gradations is shown in Fig. 4.65. The CIV values increase with the increase of the UCS values till a CIV value of about 88 is reached, after which a drop in the CIV values is observed with the increase in the UCS values. When a CIV value of 88 is considered, the corresponding CBR value from the CIV-CBR correlation is 244%. Hence it is clear that high CBR (above 200%) and CIV (above 88) values may not indicate a practical strength of the soil.

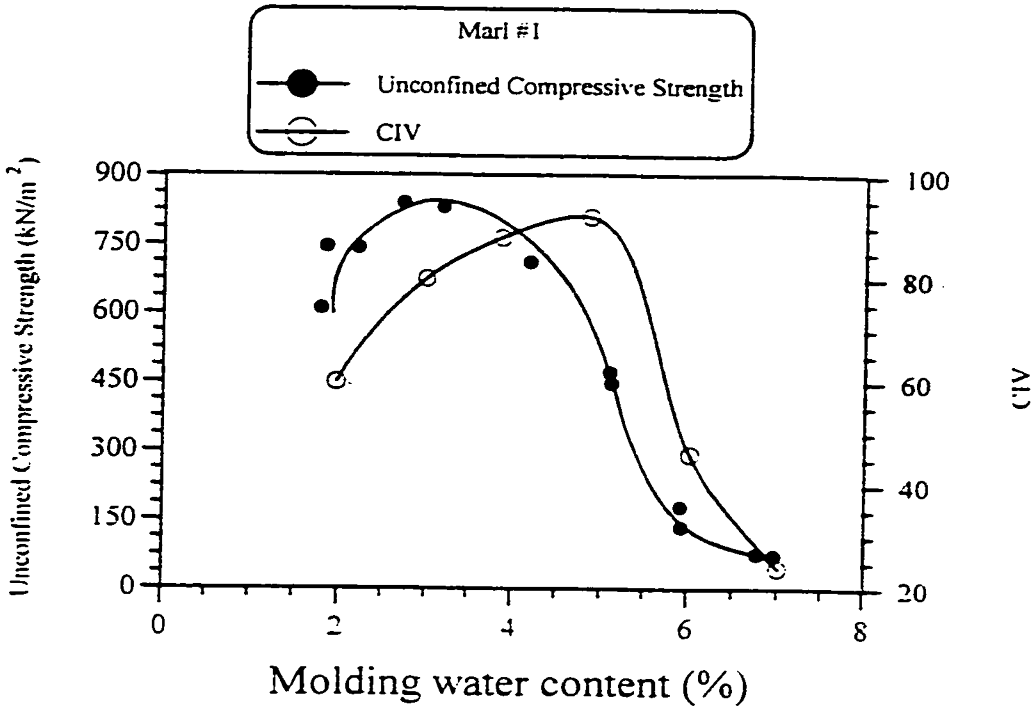


Figure 4.59: UCS-CIV-moisture relationships of the fine limit gradation for marl #1

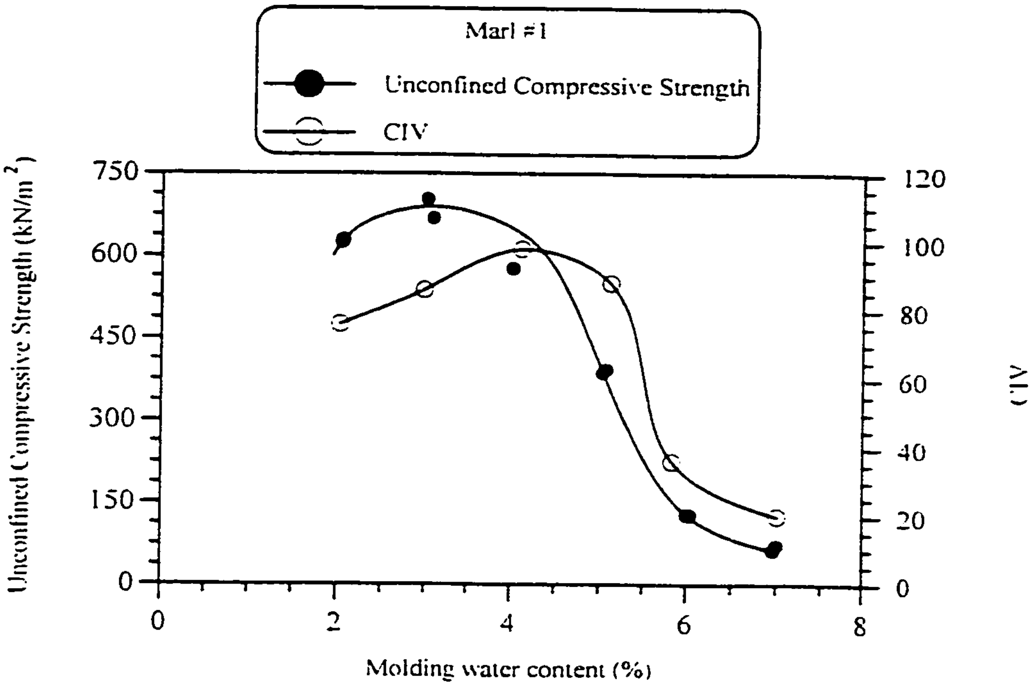


Figure 4.60: UCS-CIV-moisture relationships of the medium gradation for marl #1

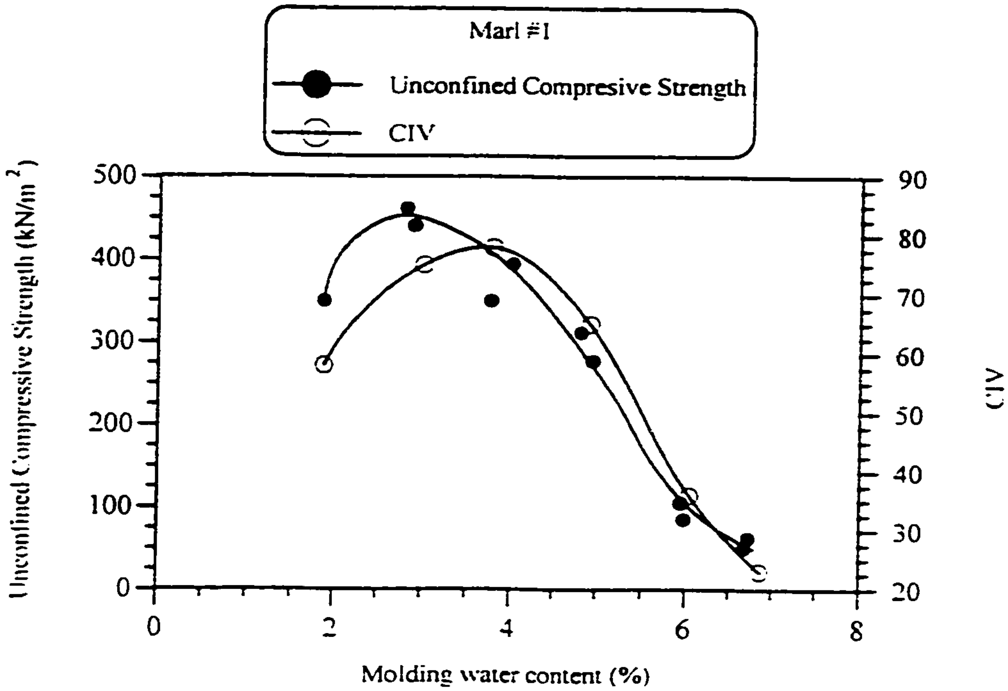


Figure 4.61: UCS-CIV-moisture relationships of the coarse limit gradation for marl #1

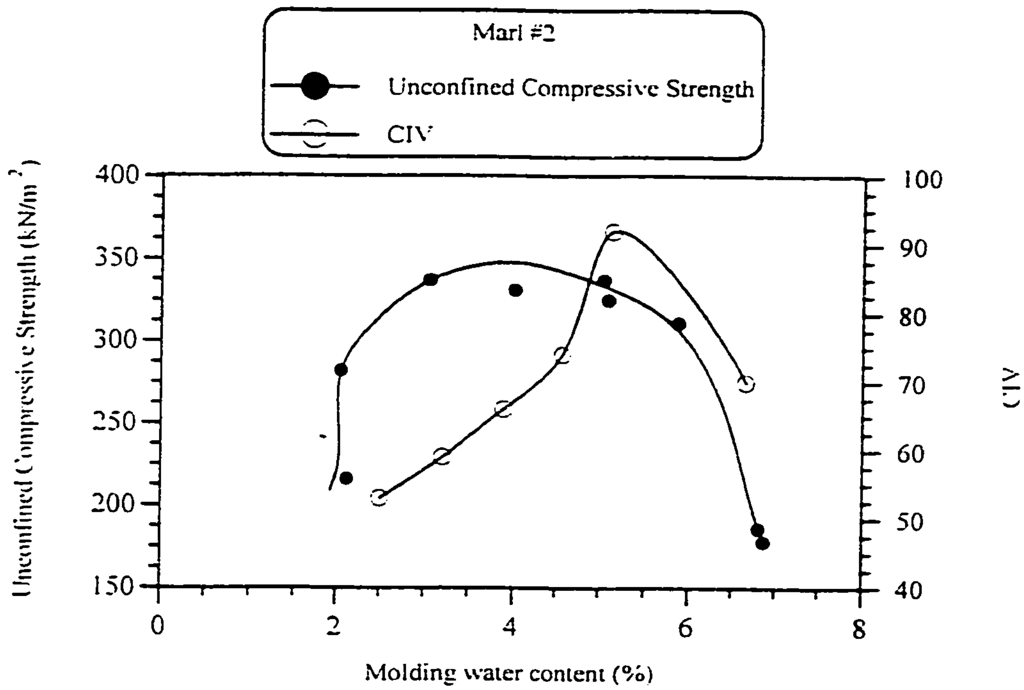


Figure 4.62: UCS-CIV-moisture relationships of the fine limit gradation for marl #2

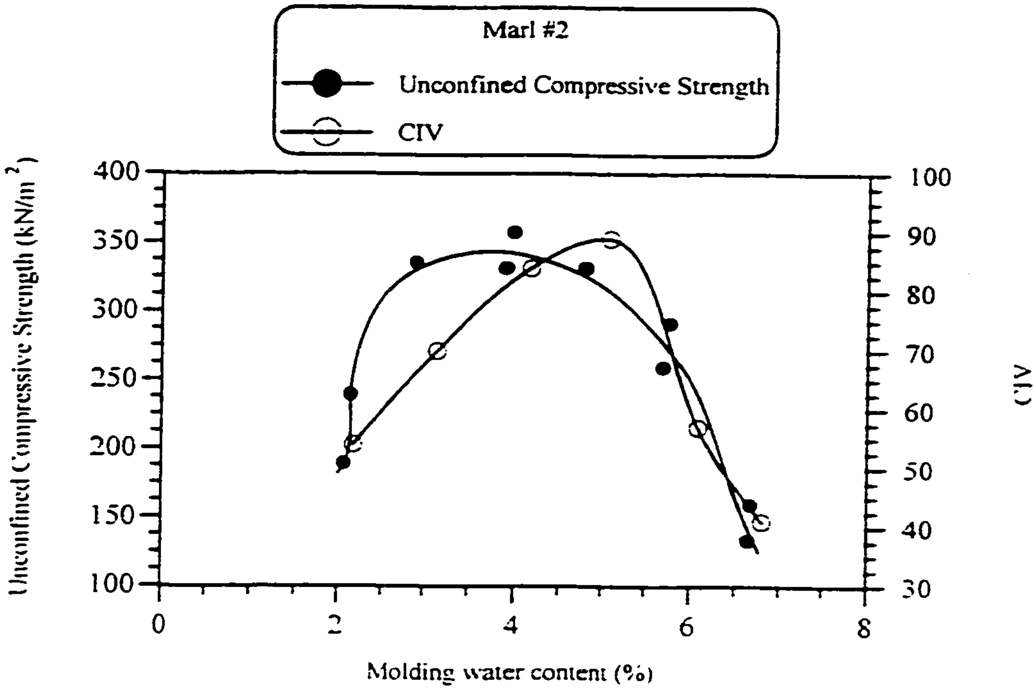


Figure 4.63: UCS-CIV-moisture relationships of the medium gradation for marl #2

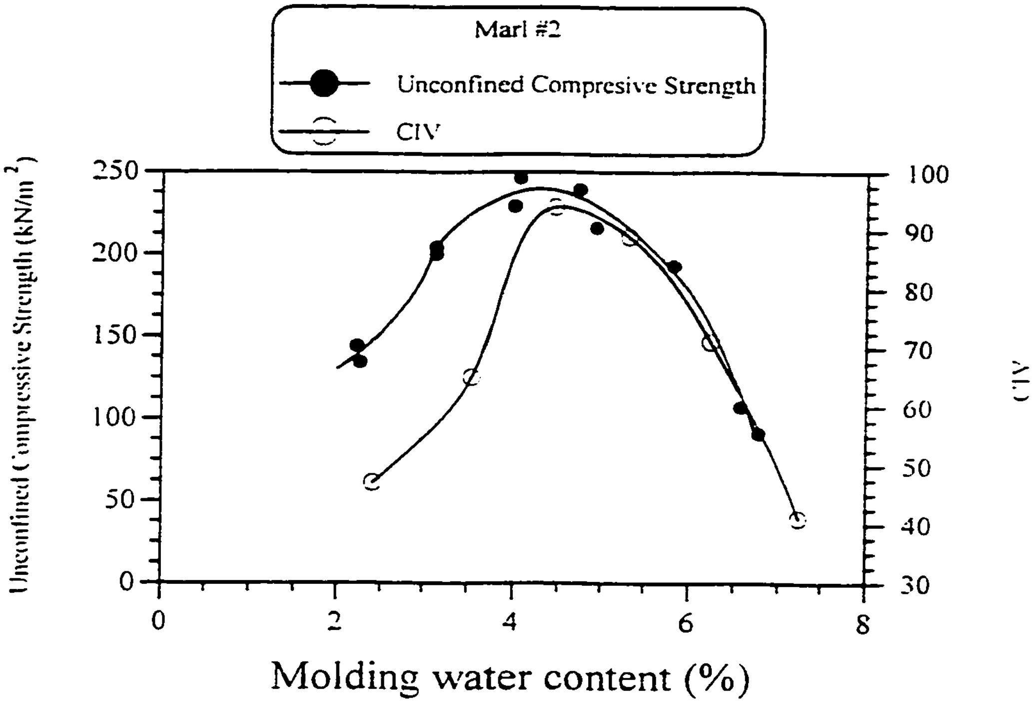


Figure 4.64: UCS-CIV-moisture relationships of the coarse limit gradation for marl #2

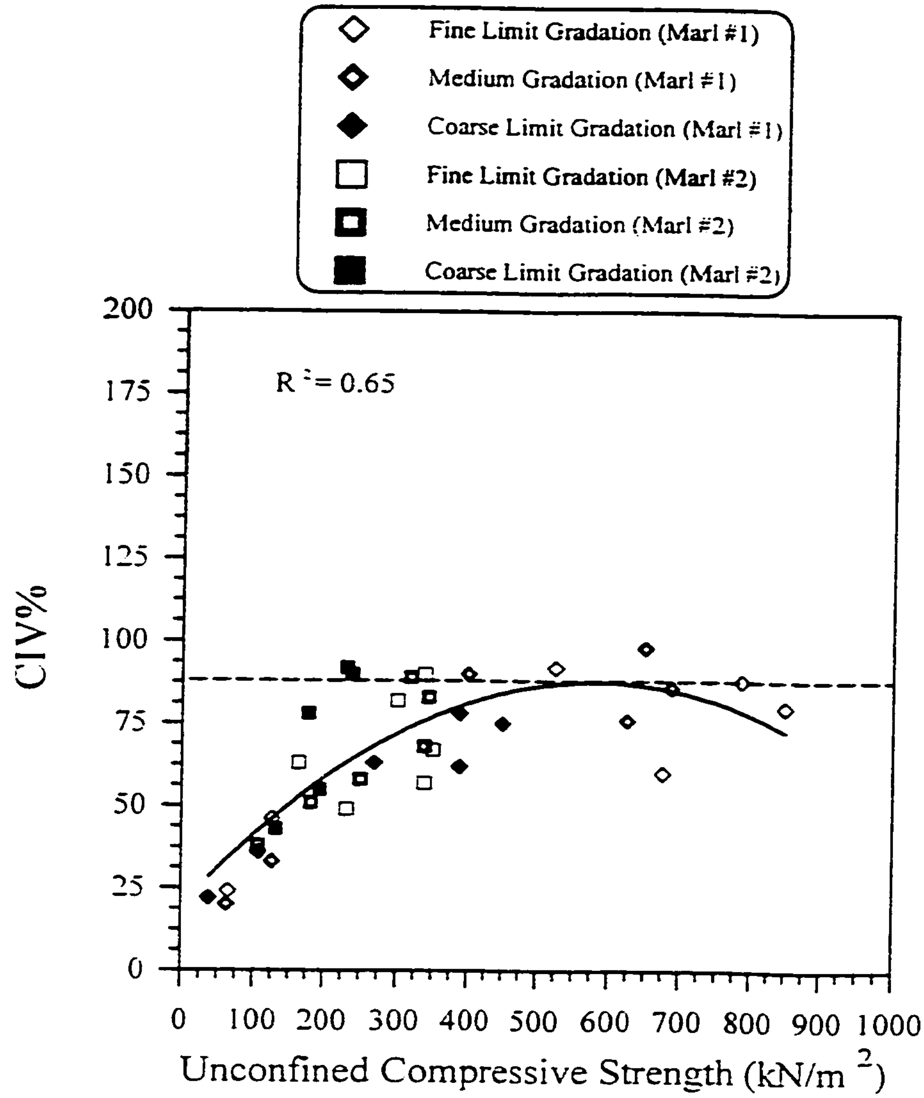


Figure 4.65: The correlation between the UCS and the CIV values obtained using the standard CBR mold

4.4 Testing Program using Large size Setup

Marl soils usually contain high contents of gravels, and this necessitates that oversize particles should be discarded from the soil sample prior testing, when using the standard compaction mold. Excluding such large amount of material will affect the results accuracy and consequently the reliability of the testing procedure. In this research program, a large size compaction mold was used in order to study the effect of excluding the large size particles from the test specimen. In order to compare the results obtained from the small size and the large size setups, the tests were performed using similar gradations for both mold sizes. The medium gradation was used to perform the comparison between the large size and standard size tests. The tests performed using the large setup were, the compaction test, the CBR test and the CIV test.

4.4.1 Compaction Test

The moisture-density relation was obtained using the large size specimens (large mold). Two sets of specimens were tested for each marl type. The first set was prepared using the scalp and replace method, which is used for the small conventional "modified Proctor" setup, by excluding the over size material and replacing it with the same amount of stones passing the ¾ in. (19 mm) sieve and retained on the No. 4 sieve. The second set was prepared using the actual gradation, which includes all sizes up to 2 in. (50.8 mm). The large setup was calibrated in order to obtain approximately the same moisture density relations, which was obtained using the small size "modified Proctor" mold for the same gradation.

For the two marls, maximum differences of less than 0.025 gm/cm^3 were occurred between the dry densities obtained, using "modified Proctor" and large size molds, for material prepared using scalp and replace method, as shown in Figs. 4.66 and 4.67. However, Garga and Madureira (1985) reported that Donaghe and Townsend showed a maximum difference of 0.06 gm/cm^3. As shown in the figures, the large setup and the small setup gave dry density values that are almost equal. Therefore, the large set up can be used as a possible replacement of the small setup. This has the advantage that, larger particles can be included in the specimen, prepared in the large mold. The entire gradation was used for both marls when reconstituting the soil into the medium gradation, but without discarding any of the large particles.

The moisture-density curves for the entire gradation for both marls, compared to the moisture density curves for the material prepared by scalp and replace method, are shown in Figs. 4.68 and 4.69. The maximum dry density and the optimum moisture content values, using the two preparation methods, for both marls are shown in Table 4.5. It is noticed that the entire gradation for both marls gave slightly higher maximum dry densities at higher moisture content values compared to the maximum values obtained for material prepared with the replacement method. According to Fragaszy et al (1990), Donaghe and Townsend reported similar observations after they noticed that scalp and replace method could give lower maximum dry unit weights compared to those obtained from samples prepared using the entire gradation.

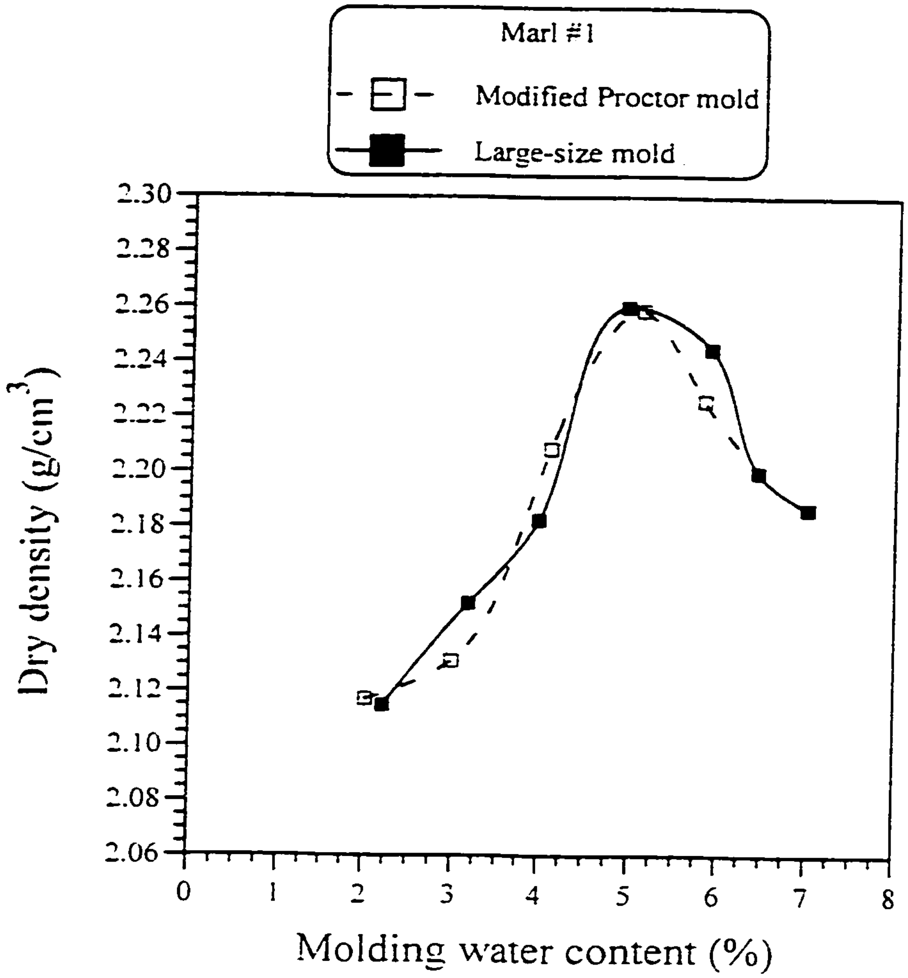


Figure 4.66: The moisture-density relationships for marl #1 obtained using standard and large molds for the medium gradation

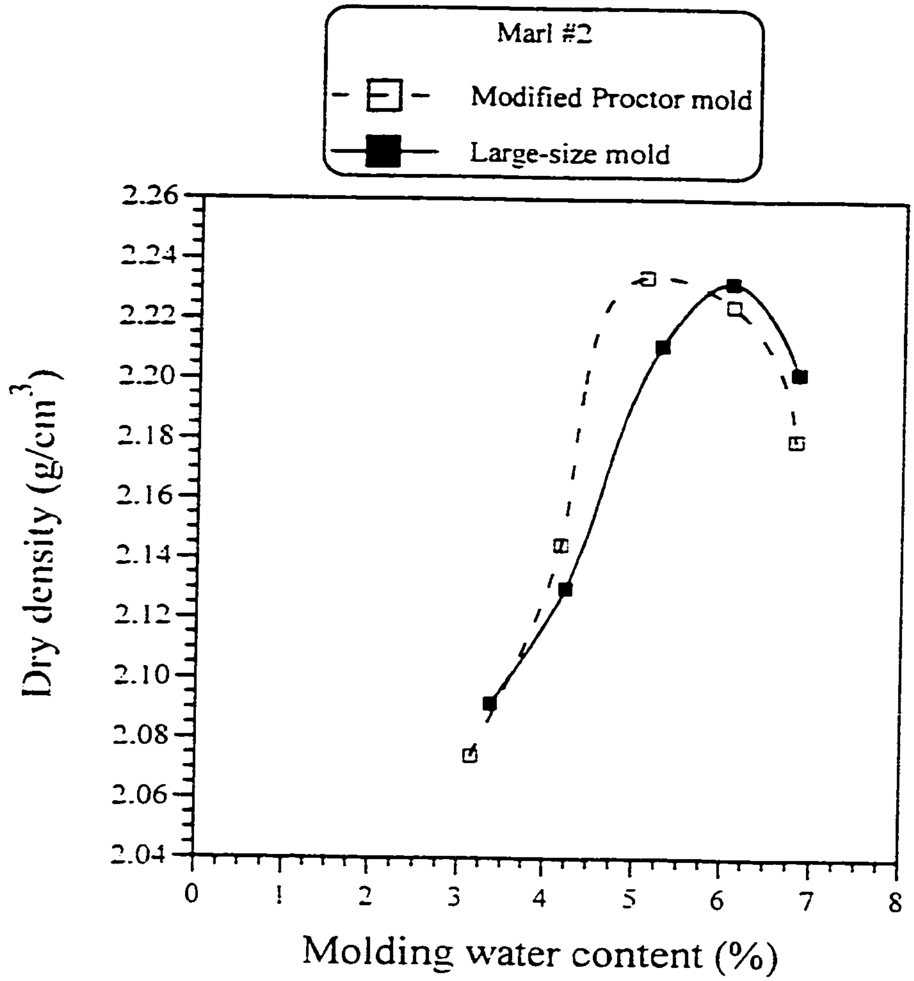


Figure 4.67: The moisture-density relationships for marl #2 obtained using standard and large molds for the medium gradation

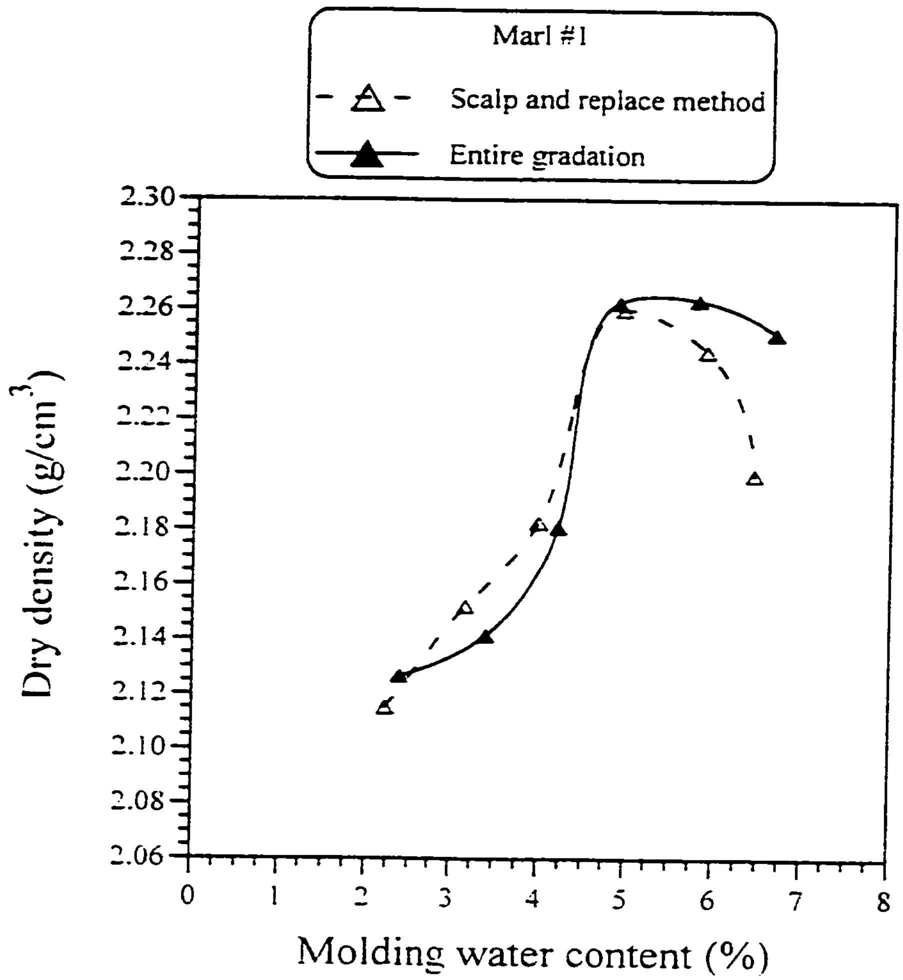


Figure 4.68: The moisture-density relationships for material prepared with scalp and replace method and entire gradation for marl #1 using large mold

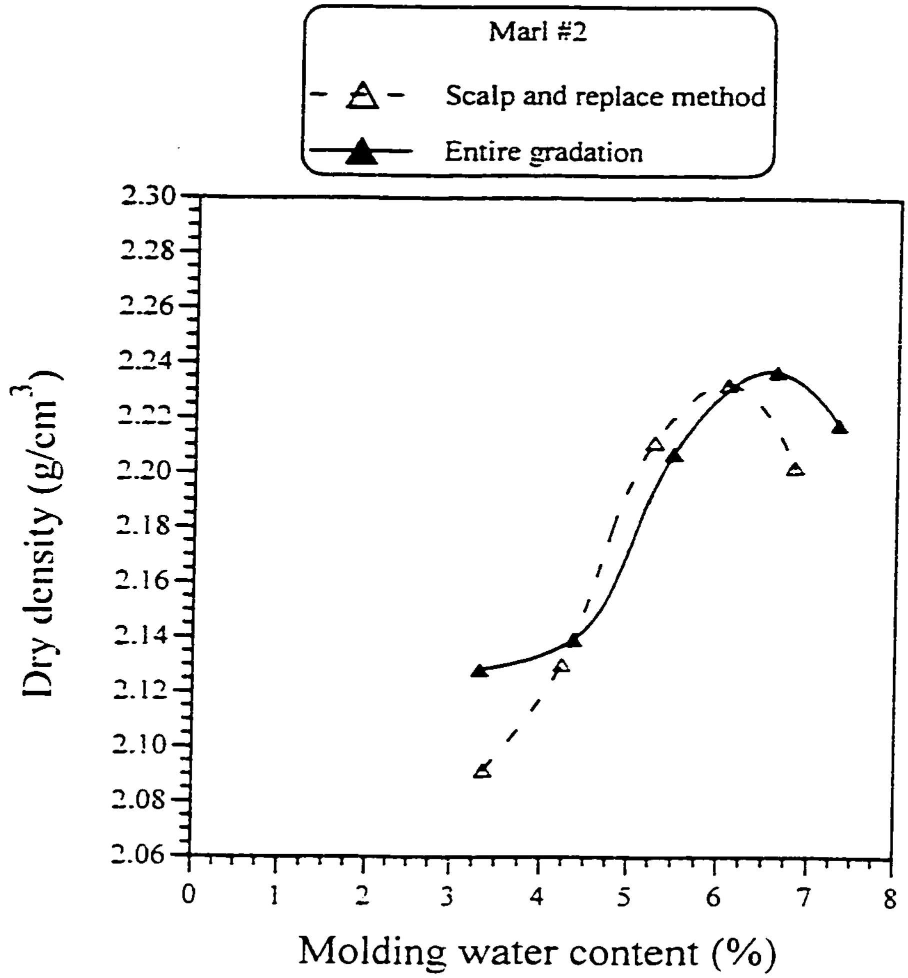


Figure 4.69: The moisture-density relationships for material prepared with scalp and replace method and entire gradation for marl #2 using large mold

Table 4.5: The γ_{dmax} and w_{opt} values of the two marls using large size mold for material prepared using scalp and replace method and entire gradation for the medium gradation

Preparation Method	Marl #1		Marl #2	
	γ_{dmax} (gm/cm^3)	w_{opt}	γ_{dmax} (gm/cm^3)	w_{opt}
Scalp and Replace Method	2.27	5.0	2.24	6.1
Entire Gradation (without replacement)	2.26	5.3	2.23	6.6

It is also observed that for both marls, the material prepared with replacement, show higher dry densities on the dry side of the optimum moisture contents. Generally the differences between the dry densities of the two sets are less that 1% for both marls. However, a remarkable change in the optimum moisture content value was observed. For Marl #1, replacing the large particles with the same mass of the smaller ones, caused the optimum moisture content to drop from 5.3% to 5.0%, while for Marl #2 a drop from 6.6% to 6.1% was observed. In general, replacing the coarse particles with finer ones will affect the amount of water required to achieve the maximum dry density and hence the optimum moisture content value will differ. This will have a significant practical consequence, since the optimum moisture content is critical for such material when it comes to field construction. It is known that the range of compaction moisture content is narrow and a little deviation may be critical since it may result in CBR values that are less than the acceptable ones.

4.4.2 Unsoaked CBR Test

Unsoaked CBR tests were performed on samples prepared using the large size mold to study the effect of the mold size on the CBR values and to make a comparison with the values obtained from the small mold. Samples were prepared for both small and large mold sizes, using the same gradation (the medium gradation). The large mold CBR tests were performed on samples prepared using scalp and replace method and on samples prepared using the entire gradation.

The CBR-moisture-density relationships for Marl #1, for samples prepared using scalp and replace method and samples prepared utilizing the entire gradation are shown in Figs. 4.70 and 4.71, respectively. In order to compare the CBR curves obtained using the two preparation methods, the two CBR curves are shown in Fig. 4.72. It is clear that the material prepared using the entire gradation (without excluding the oversize particles) has higher maximum CBR values. This is caused by the presence of the large particles in the sample, which increase the resistance to penetration of the plunger into the soil. It is shown in the figure that the differences between the CBR values for the two curves decrease as the moisture content increases on the wet side of optimum.

The CBR values for specimens compacted using scalp and replace method, using small and large molds, are shown in Fig. 4.73. A sharp decrease in CBR values was obtained when using small molds as compared to the same material tested using the large mold. The maximum CBR obtained from the small mold was 289% while maximum CBR value obtained from the large mold did not exceed 127%. This corresponds to a decrease of about 56% in the maximum CBR value obtained from the conventional mold. However, the differences in the CBR values were reduced on the wet side of optimum. This remarkable difference between the two curves is attributed to the effect of mold restraining effect or confinement in the case of small mold, which is absent or at least reduced when the large mold was used.

The CBR-moisture-density relationships for Marl #2, for samples prepared using scalp and replace method and samples prepared utilizing the entire gradation are shown in

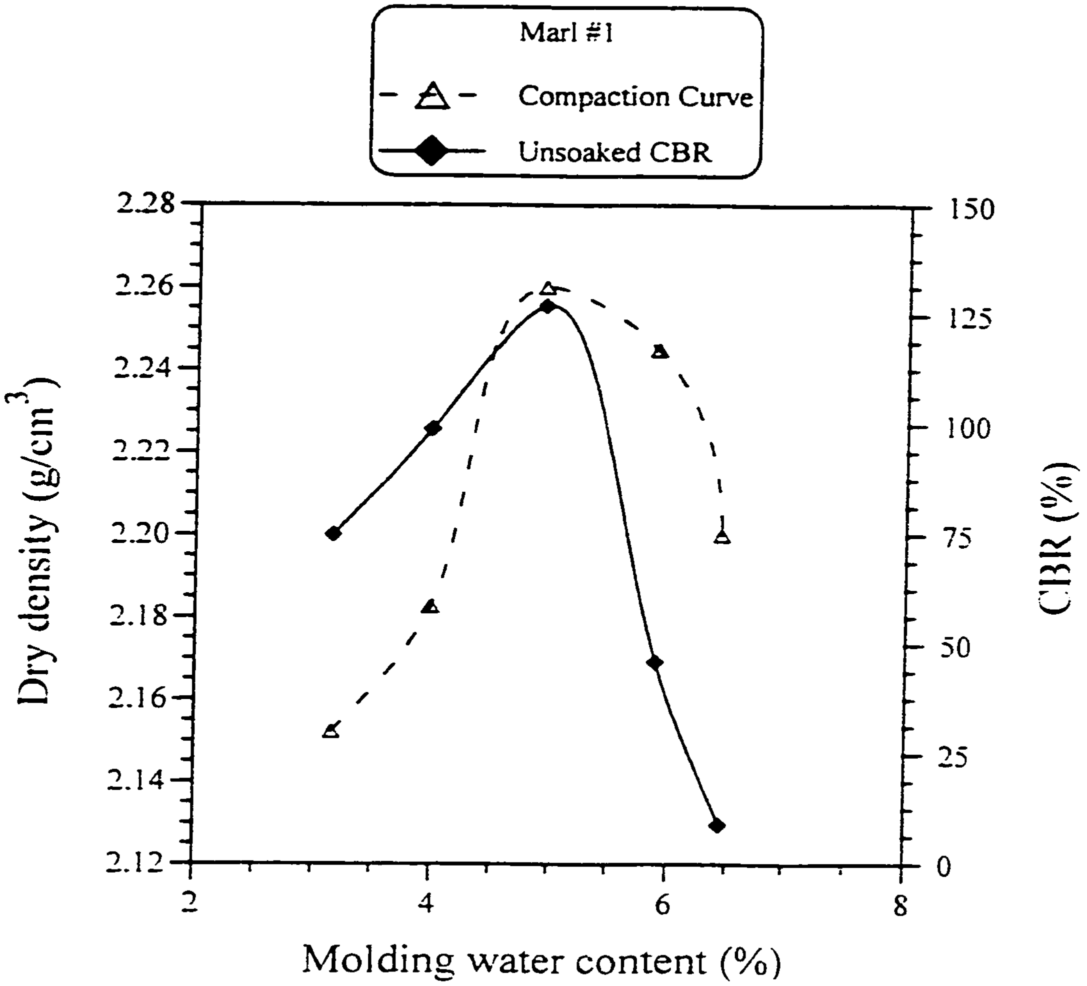


Figure 4.70: Unsoaked CBR-moisture-density relationships of sample prepared in the large mold using scalp and replace method for the medium gradation for marl #1

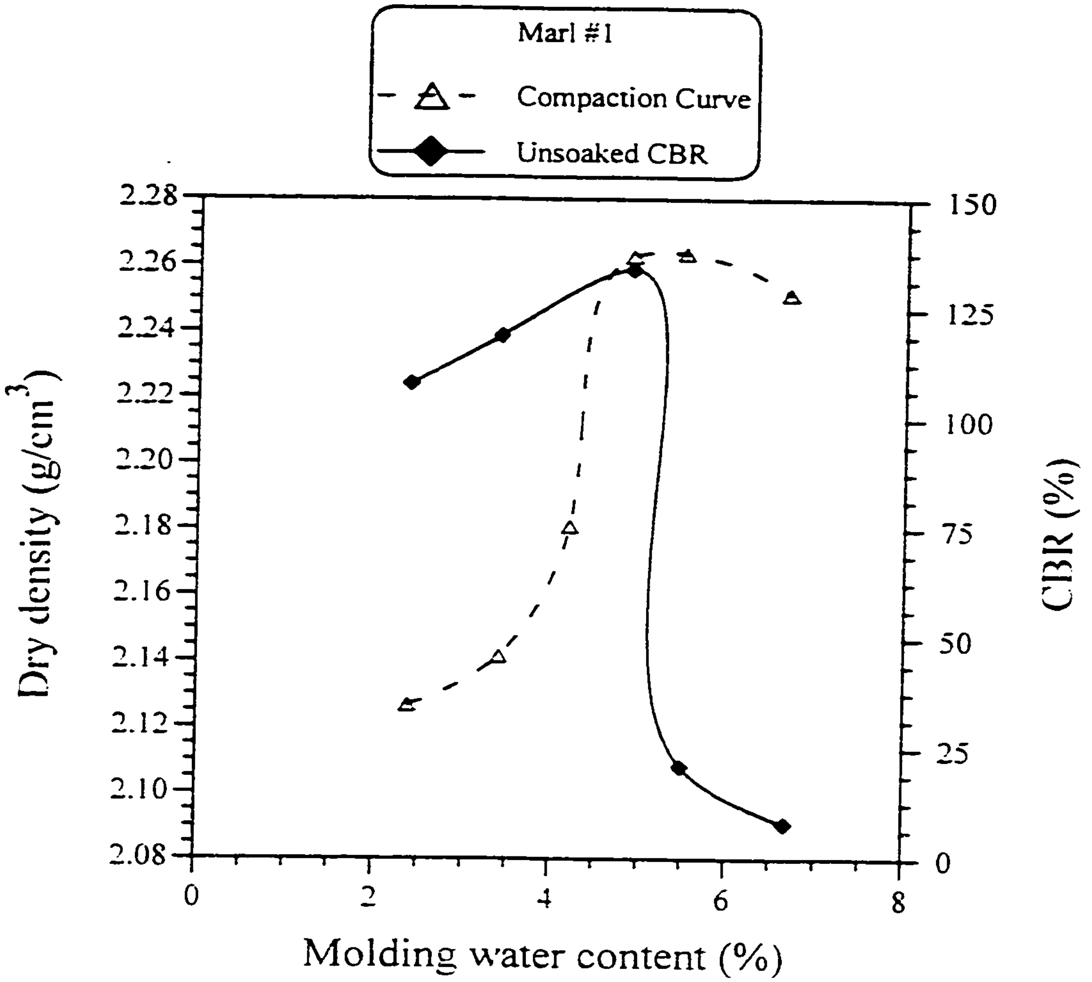


Figure 4.71: Unsoaked CBR-moisture-density relationships of sample prepared in the large mold using the entire material for the medium gradation for marl #1

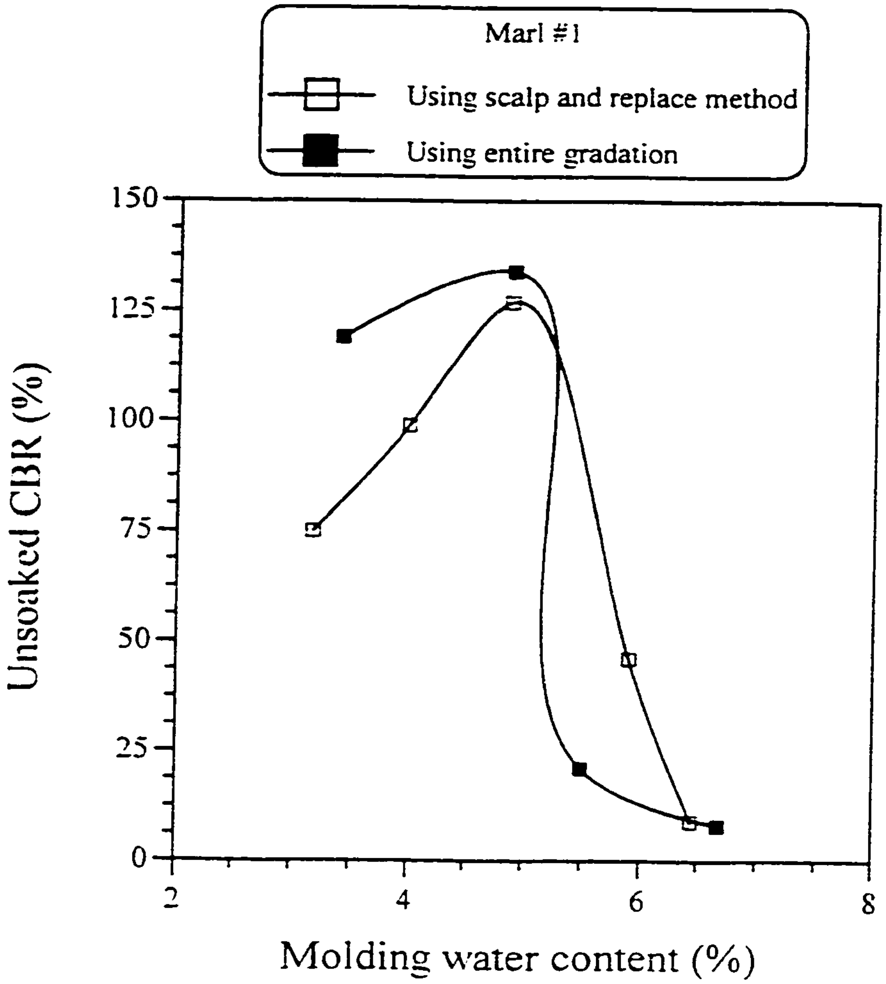


Figure 4.72: Unsoaked CBR-moisture relationships of sample prepared in the large mold using scalp and replace method and entire material for the medium gradation for marl #1

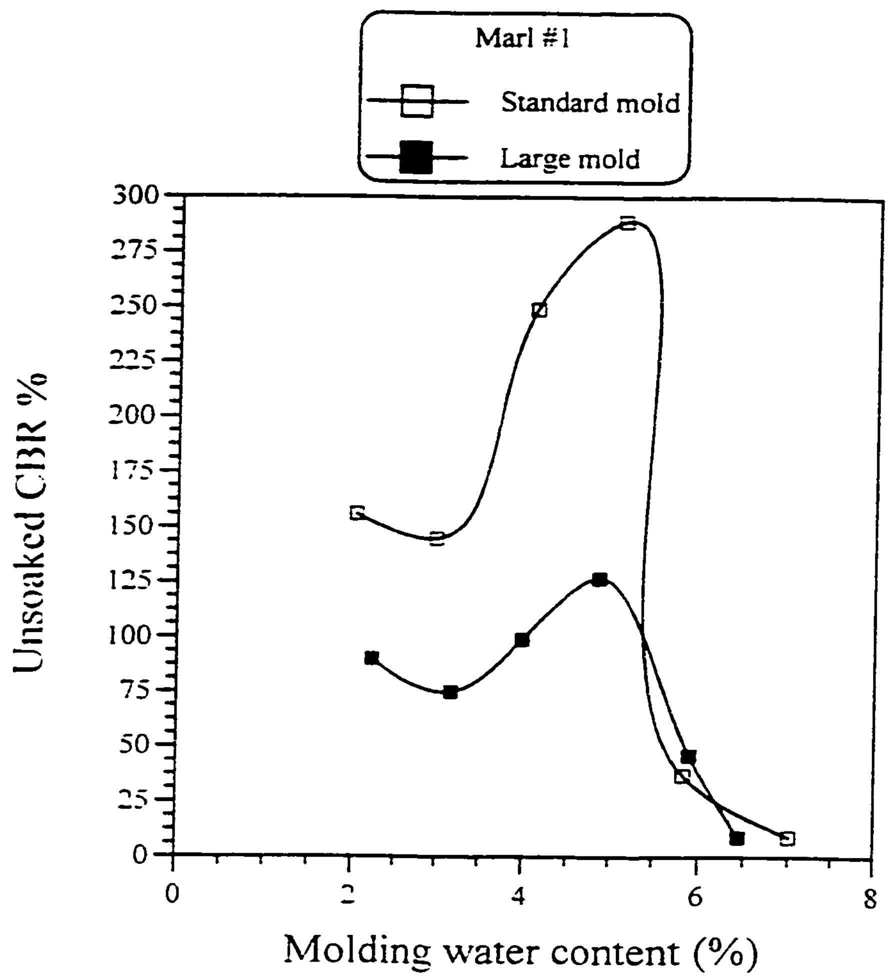


Figure 4.73: Unsoaked CBR-moisture relationships of the material prepared using scalp and replace method using standard and large molds for marl #1

Figs. 4.74 and 4.75, respectively. The CBR curves obtained from the two preparation methods are shown in Fig. 4.76. It is clear that the material prepared using the entire gradation has higher CBR values at all molding water contents, which is caused by the presence of large particles in the sample. As shown in the figure, the differences between the CBR values for the two curves are greater on the dry side of optimum compared to the differences on the wet side of optimum moisture content. On the wet side of optimum the presence of excess water will reduce the effective strength of the sample and hence the preparation method will not have significant effect on the strength.

The CBR values for specimens compacted using scalp and replace method. for samples prepared using the small and large molds. are shown in Fig. 4.77. A sever decrease in CBR values resulted upon the use of the large mold, for the same material. when compared to the CBR values obtained from the small mold. The maximum CBR value obtained from the small mold was 311% and the maximum CBR value obtained from the large mold was 162%. This gives a decrease of about 50% in the maximum CBR value when using the large mold.

It is clear that the CBR values obtained using the large mold. for samples compacted on the dry side of optimum. were almost half of that obtained using the conventional small mold for the two selected marls as shown in Figs. 4.73 and 4.77. To study the effect of wall restraining on the CBR values, a series of CBR tests were conducted at points lying at half the distance between the wall and the center of the mold. The results given in Figs. 4.78 to 4.81 clearly show that the CBR values of the off center points are generally

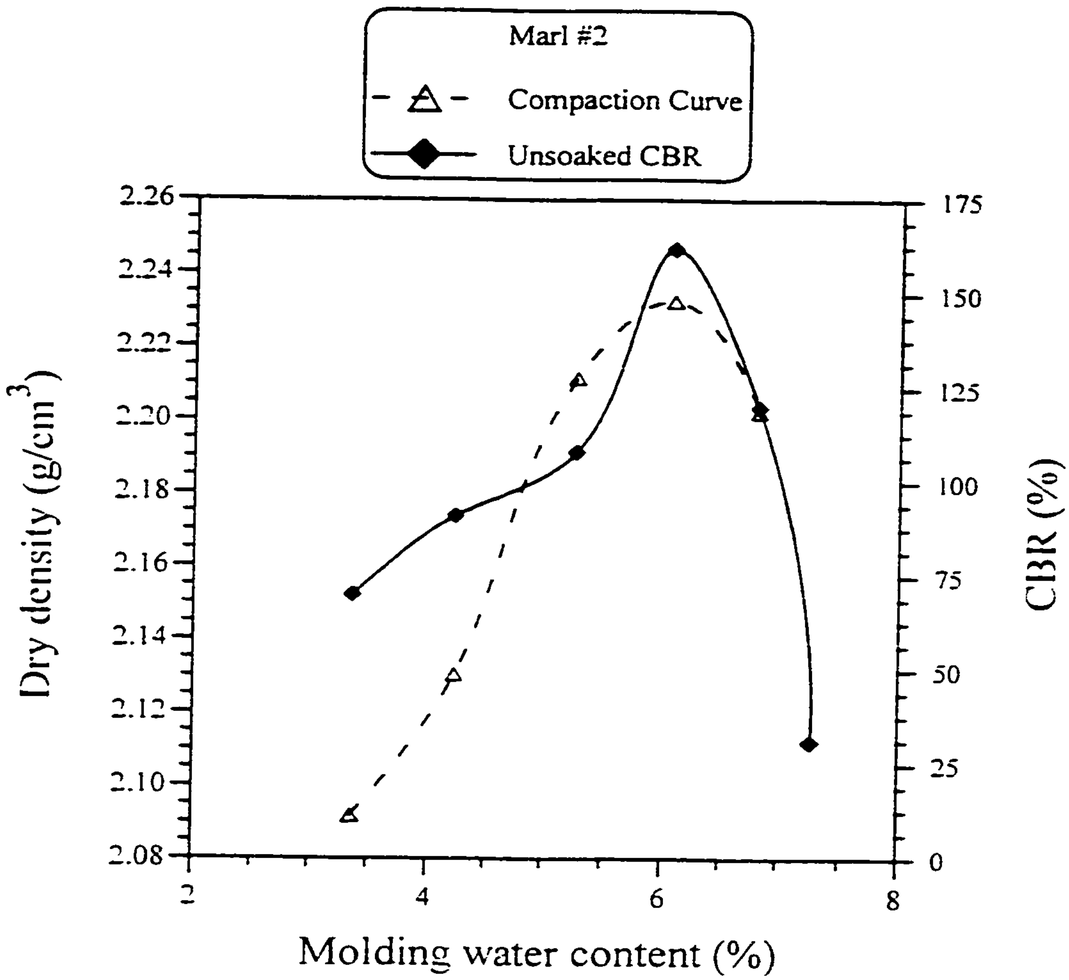


Figure 4.74: Unsoaked CBR-moisture-density relationships of sample prepared in the large mold using scalp and replace method for the medium gradation for marl #2

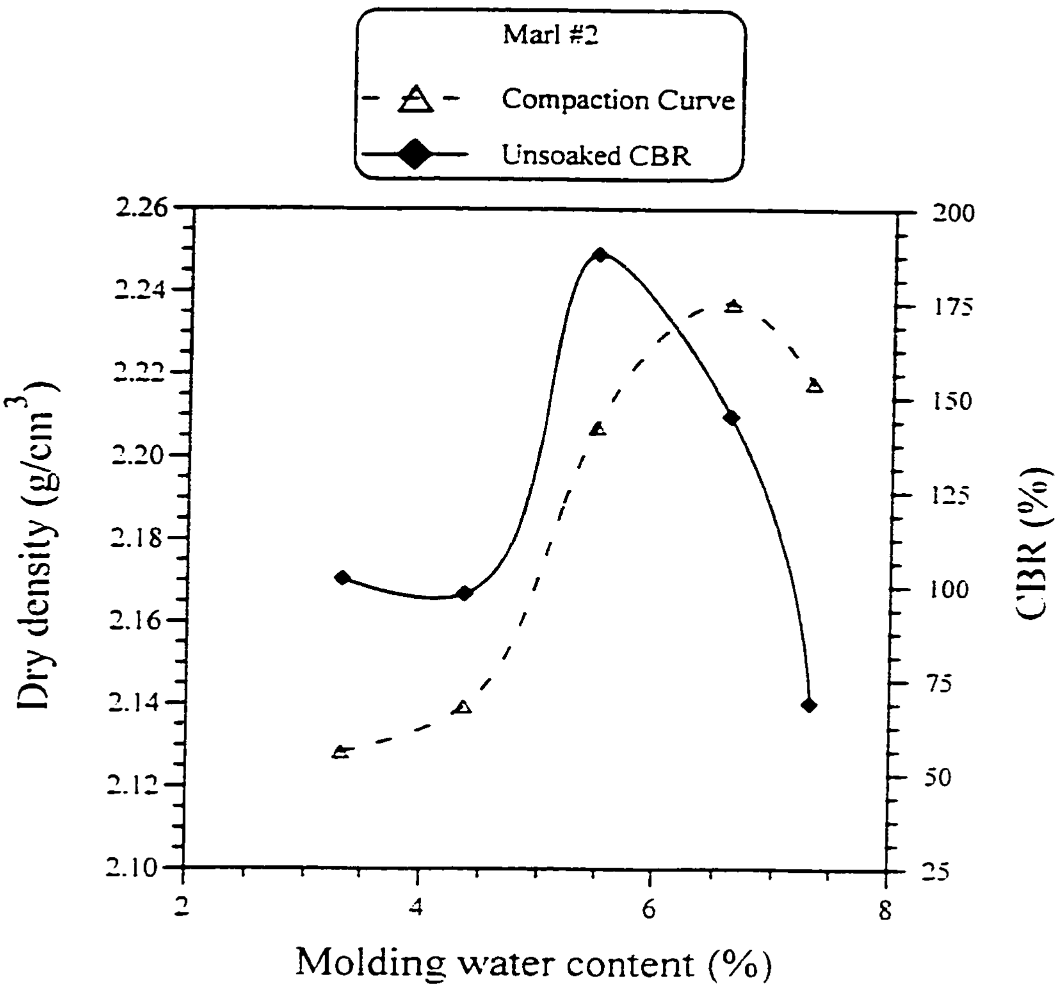


Figure 4.75: Unsoaked CBR-moisture-density relationships of sample prepared in the large mold using the entire material for the medium gradation for marl #2

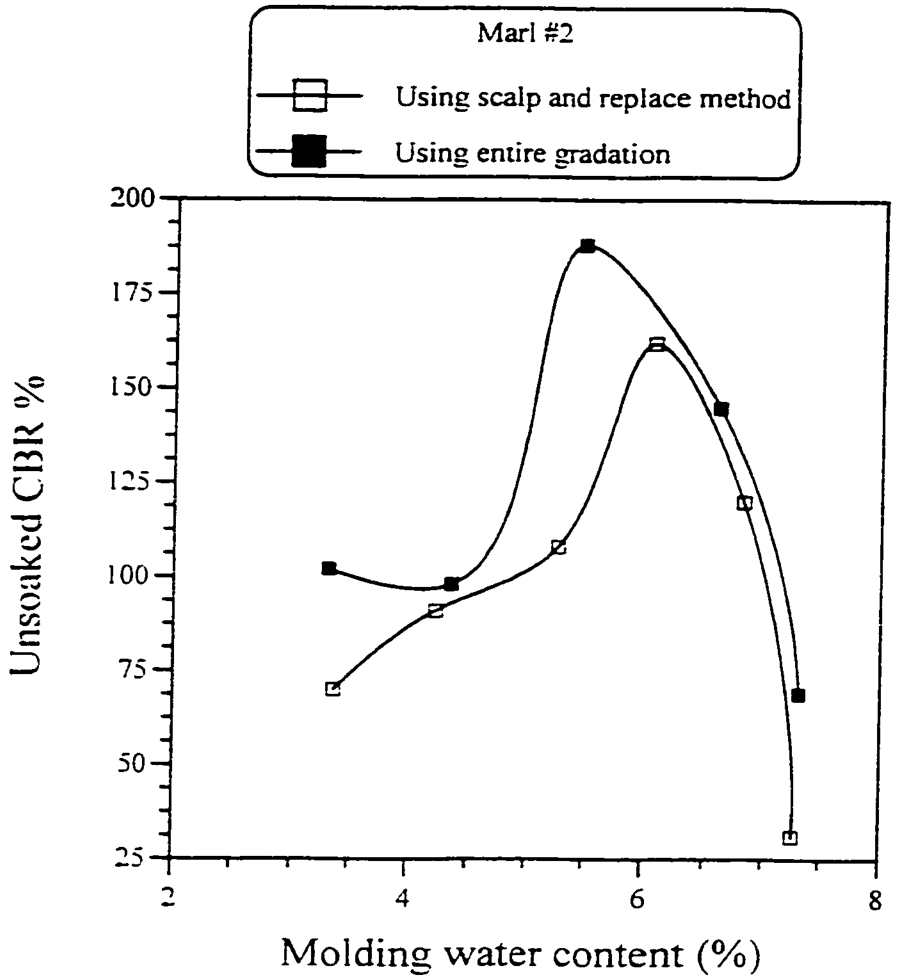


Figure 4.76: Unsoaked CBR-moisture-density relationships of sample prepared in the large mold using scalp and replace method and entire material for the medium gradation for marl #2

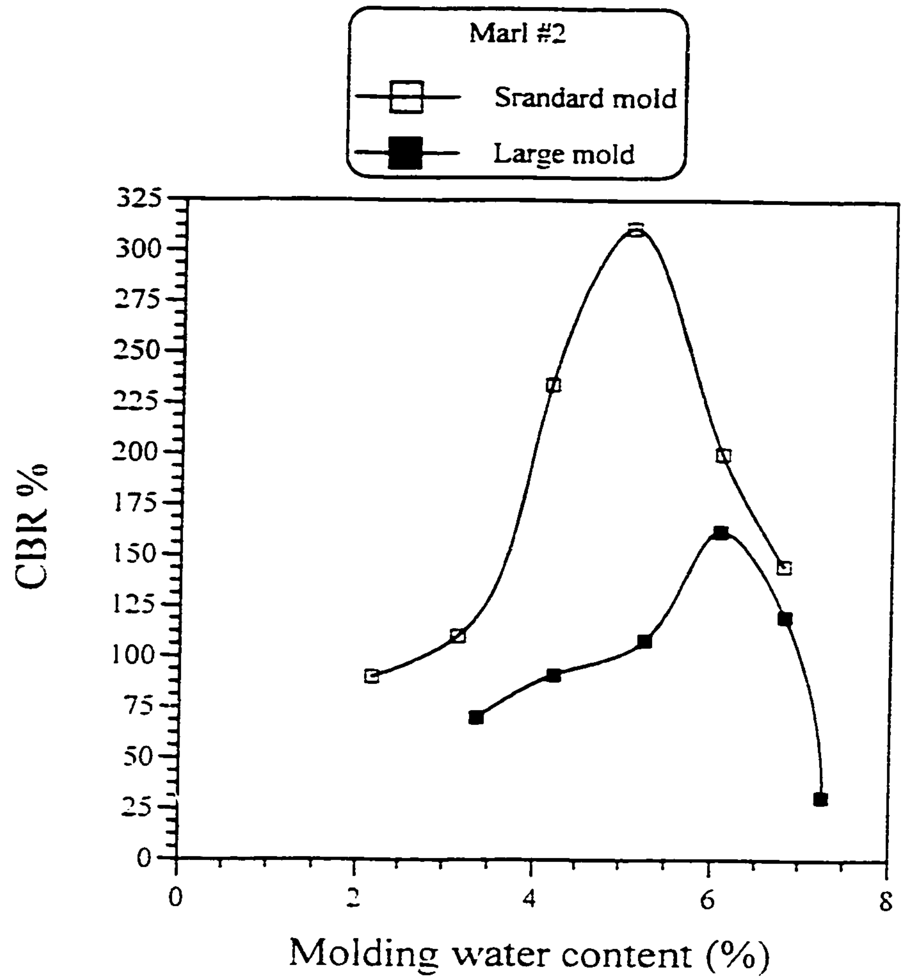


Figure 4.77: Unsoaked CBR-moisture relationships of the material prepared using scalp and replace method using standard and large molds for marl #2

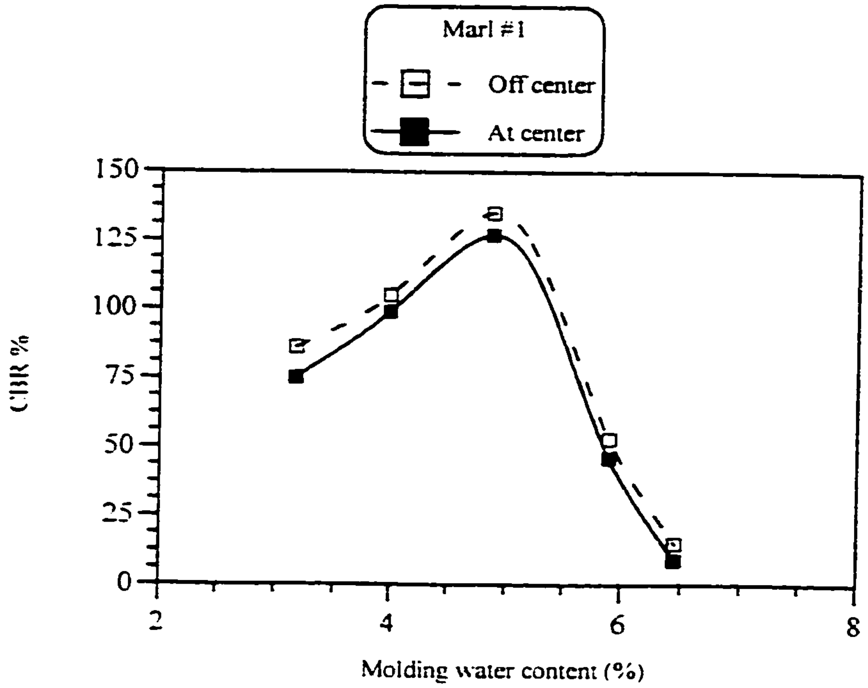


Figure 4.78: Unsoaked CBR-moisture relationships at center and off center points for material prepared by scalp and replace method using the large mold for marl #1

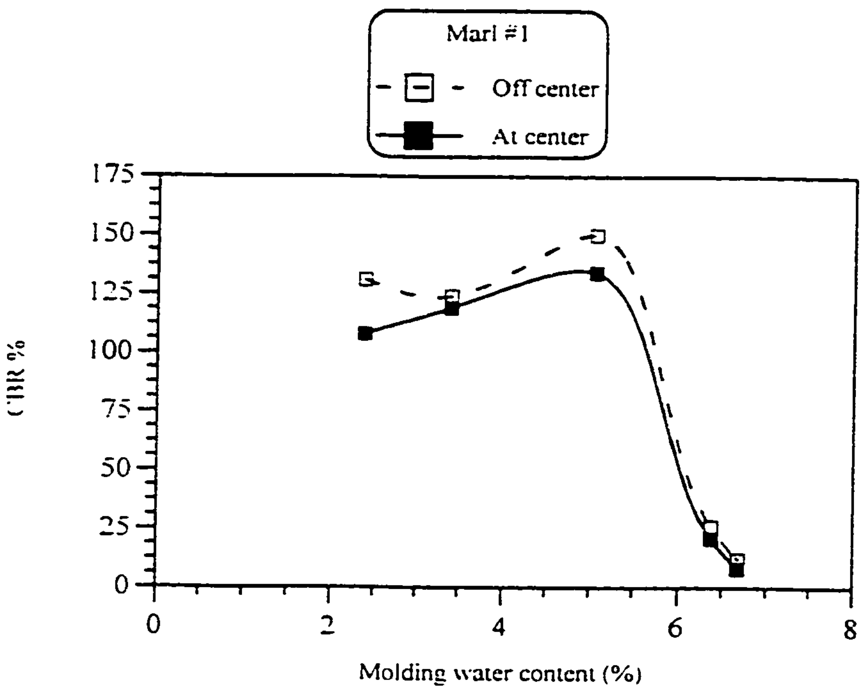


Figure 4.79: Unsoaked CBR-moisture relationships at center and off center points for material prepared in the large mold using the entire gradation for marl #1

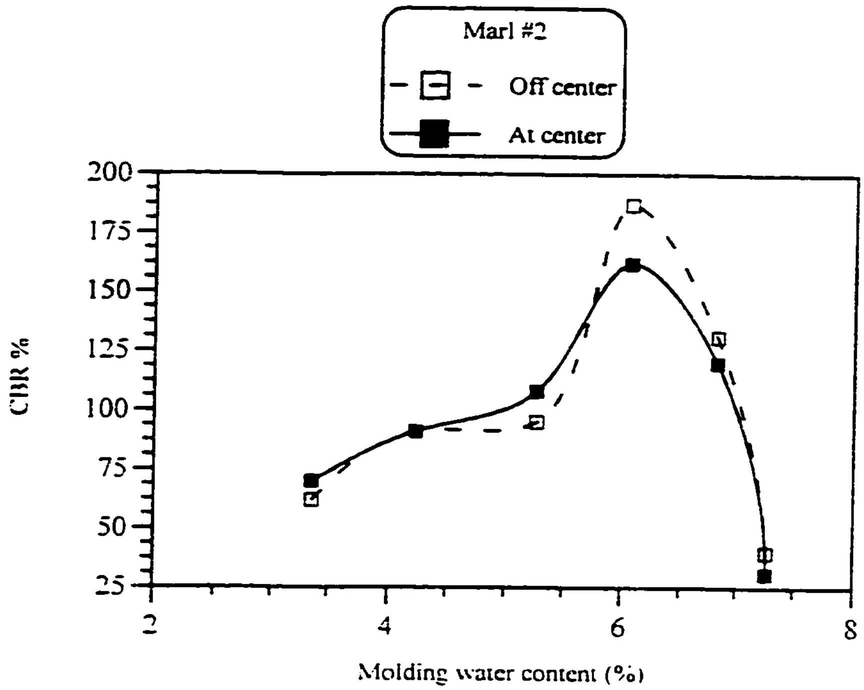


Figure 4.80: Unsoaked CBR-moisture relationships at center and off center points for material prepared by scalp and replace method using the large mold for marl #2

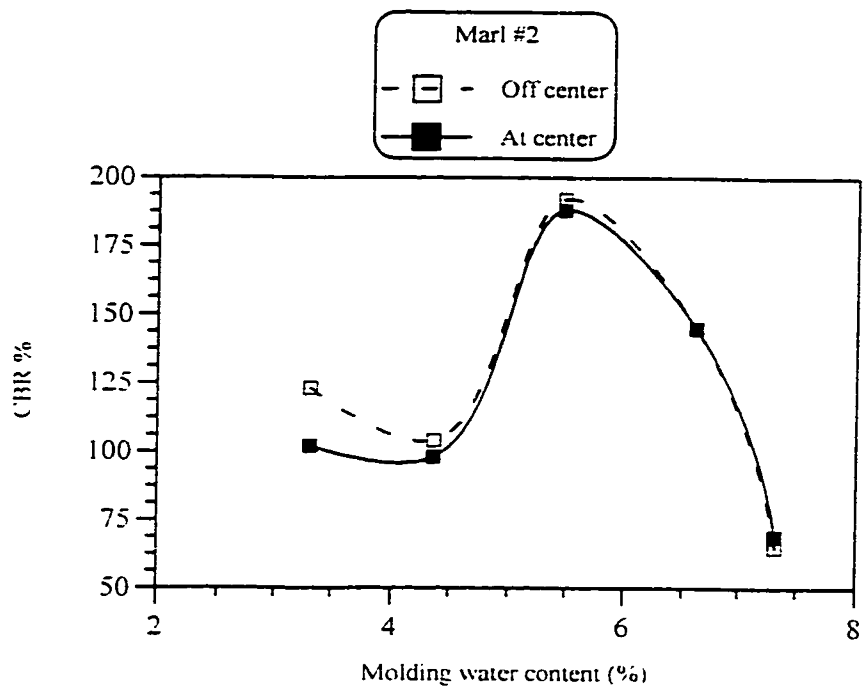


Figure 4.81: Unsoaked CBR-moisture relationships at center and off center points for material prepared in the large mold using the entire gradation for marl #2

higher than those obtained at the center, although the two points are about 3 in. apart only. This is caused by the boundary conditions and restraining effect of the mold. This could explain the differences in the CBR values between the standard (small) mold and the large mold.

4.4.3 Clegg Hammer Test

Clegg Hammer tests were performed on the large specimens to study the effect of the mold size on the CIV values. Samples were prepared for both the standard (small) and the large mold sizes, using the medium gradation. The Clegg hammer tests were conducted on samples prepared using scalp and replace method in addition to samples prepared using the entire gradation.

The CIV-moisture-density relationships for Marl #1, for the two marls compacted in the large size mold for the two preparation methods utilizing the medium gradation are shown in Figs. 4.82 and 4.83. In order to compare the CIV values obtained using the two preparation methods, the two CIV curves are shown together in Fig. 4.84. The material prepared using the scalp and replace method show higher maximum CIV value compared to the material prepared using the entire gradation, except at low moisture contents. The results have shown that the differences between the CIV values for the two curves are more on the dry side of optimum. However, the two curves merge close to each other on the wet side of optimum. The CIV values for specimens prepared using scalp and replace method, compacted in the standard and large molds, are shown in Fig. 4.85. Generally, the large mold gave lower CIV values compared to the small mold. The maximum CIV

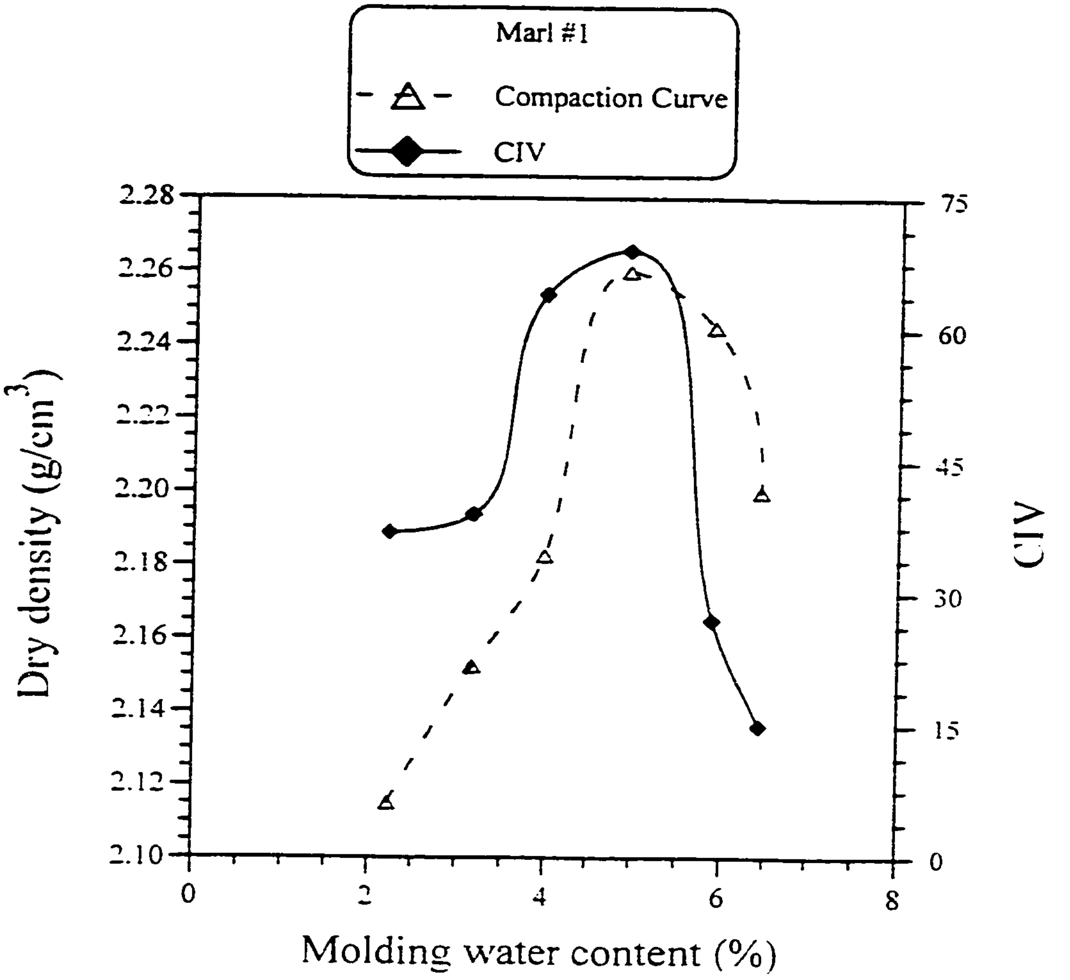


Figure 4.82: CIV-moisture-density relationships of sample prepared in the large mold using scalp and replace method for the medium gradation for marl #1

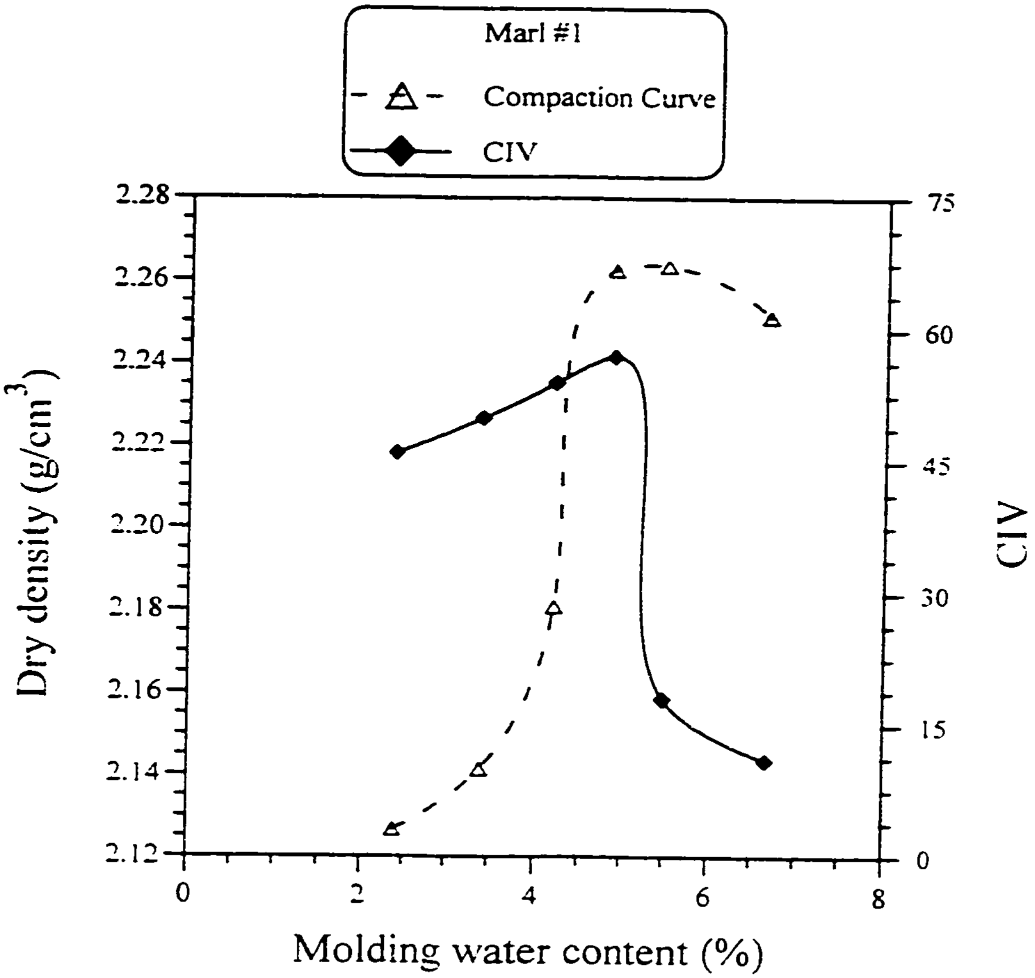


Figure 4.83: CIV-moisture-density relationships of sample prepared in the large mold using the entire material for the medium gradation for marl #1

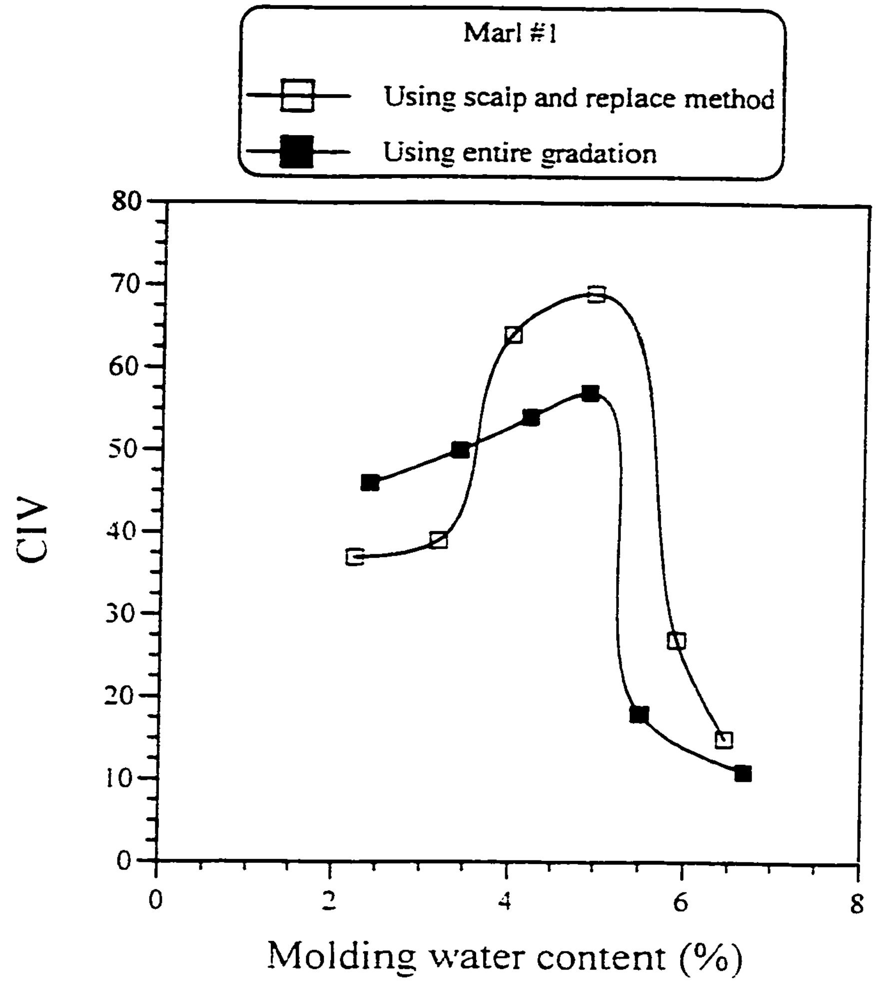


Figure 4.84: CIV-moisture relationships of sample prepared in the large mold using scalp and replace method and entire material for the medium gradation for marl #1

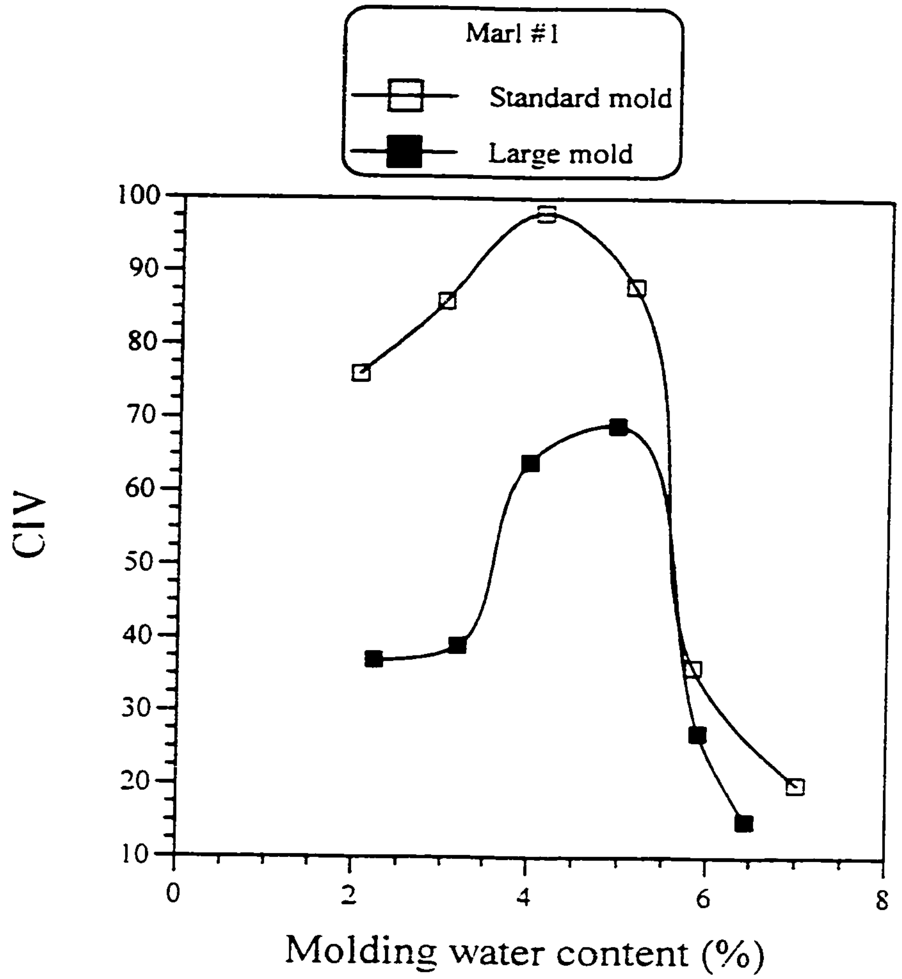


Figure 4.85: CIV-moisture relationships of the material prepared using scalp and replace method using standard and large molds for marl #1

obtained from the small mold was 98 while the maximum CIV value obtained from the large mold was 69. This gives a decrease of about 30% in the maximum CIV value. The differences between the CIV values, for the two preparation methods, decreased on the wet side of optimum as shown in the figure.

The CIV-moisture-density relationships for Marl #2, for the two preparation methods using the large size mold are shown in Figs. 4.86 and 4.87. To Compare the CIV values obtained from the two preparation methods, the two CIV curves are shown in Fig. 4.88. It is shown that the material prepared using scalp and replace method has higher maximum CIV, as observed for Marl #1. As shown in the figure, the differences between the CIV values for the two curves are greater on the dry side of optimum, while there are slight differences on the wet side of the optimum moisture content. However, the differences are small when compared to those for the CBR values. Hence, the CIV show less sensitivity to specimen maximum particle size compared to the CBR value.

The CIV curves for specimens prepared using scalp and replace method, compacted in small and large molds, are shown in Fig. 4.89. Generally the large mold gave lower CIV values compared to the small mold. The maximum CIV obtained using the small mold was 89 while the maximum CIV value obtained from the large mold was 56, this corresponds to a decrease of about 37% in the maximum CIV value. Hence, as observed for the two selected marls, the CIV obtained using the large mold is about two thirds of that obtained using the conventional standard mold.

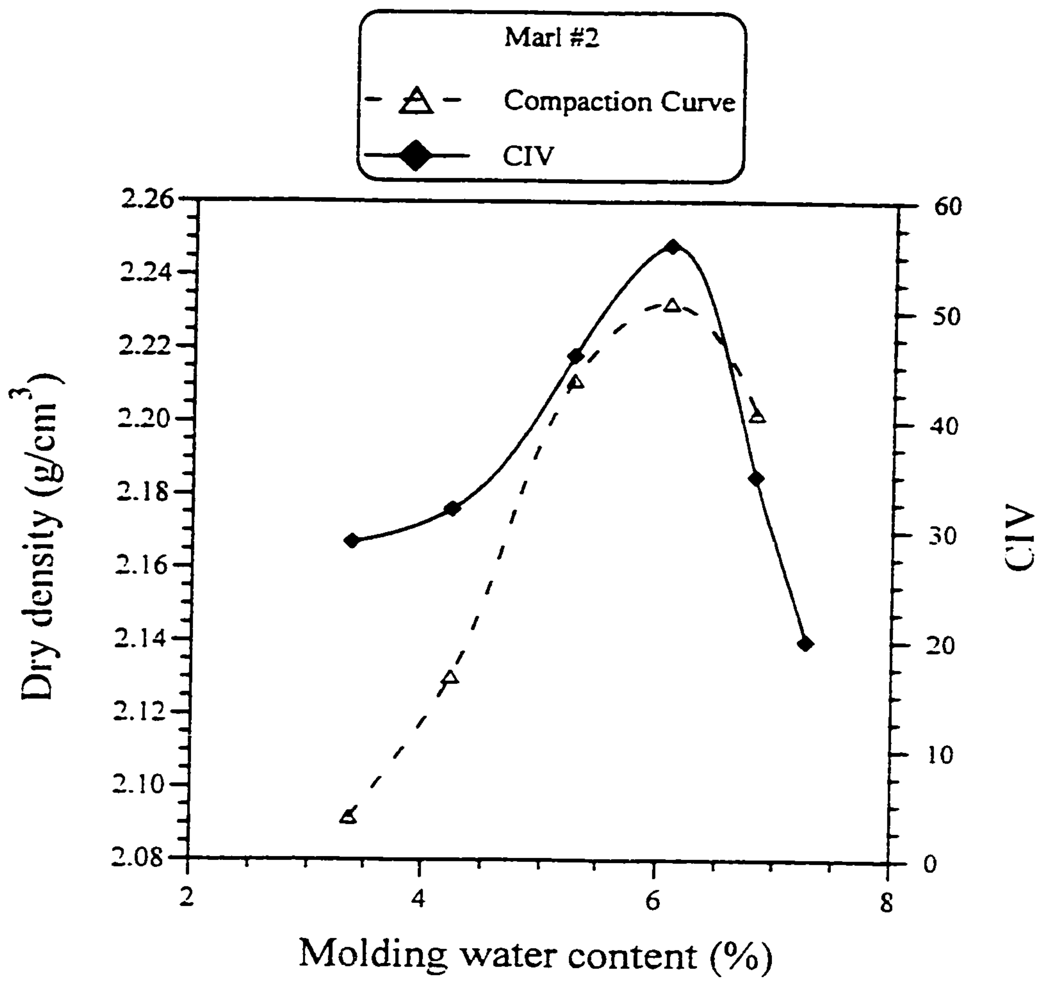


Figure 4.86: CIV-moisture-density relationships of sample prepared in the large mold using scalp and replace method for the medium gradation for marl #2

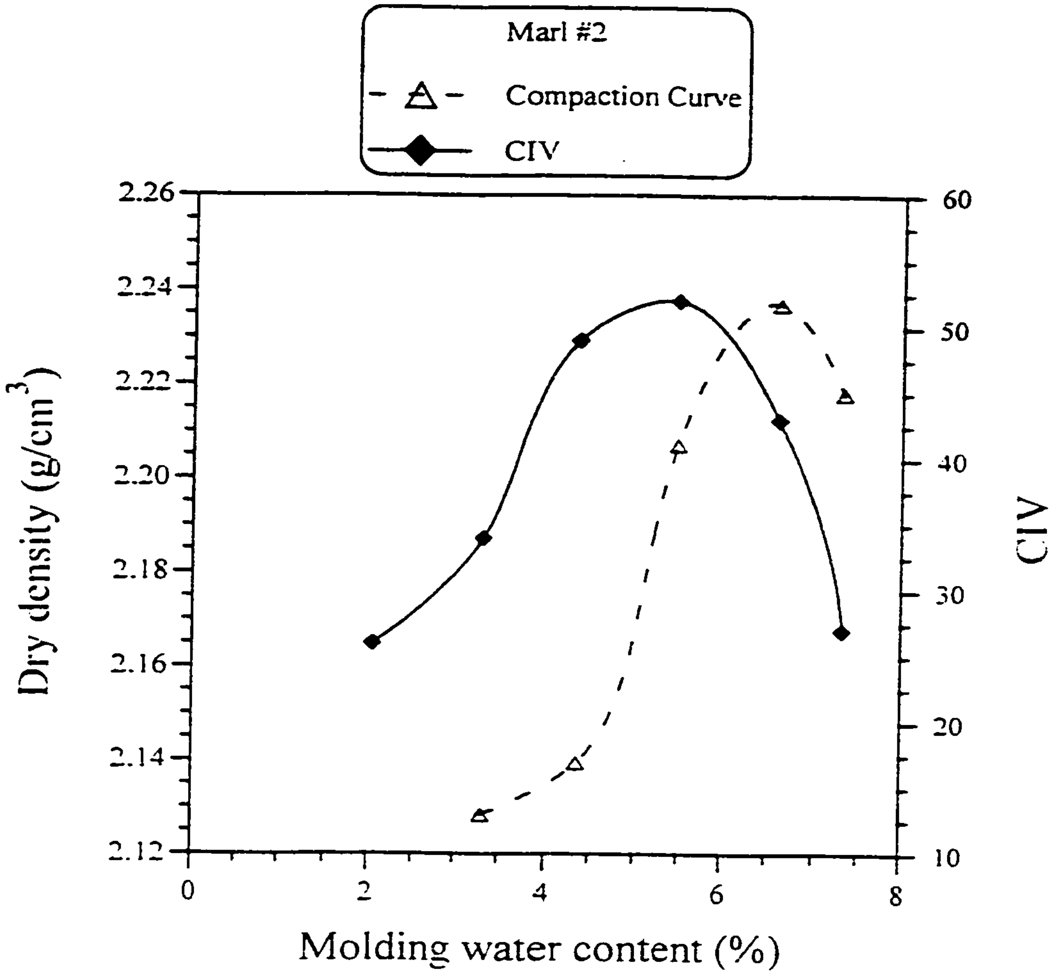


Figure 4.87: CIV-moisture-density relationships of sample prepared in the large mold using the entire material for the medium gradation for marl #2

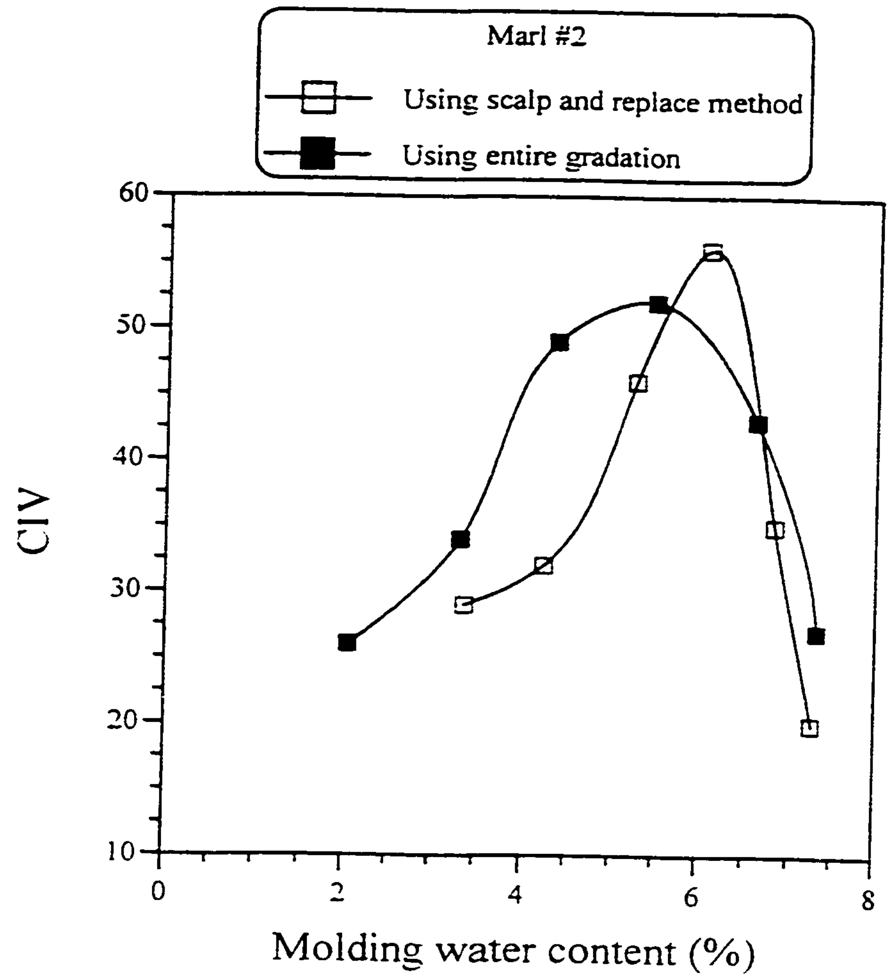


Figure 4.88: CIV-moisture relationships of sample prepared in the large mold using scalp and replace method and entire material for the medium gradation for marl #2

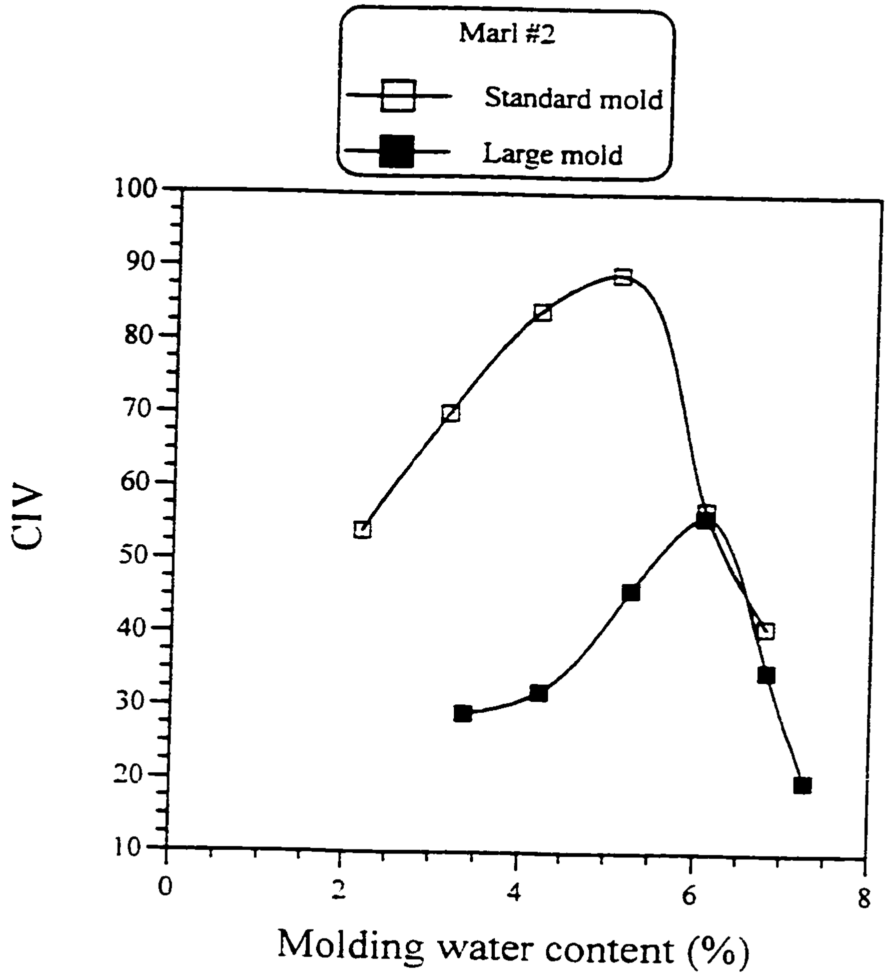


Figure 4.89: CIV-moisture relationships of the material prepared using scalp and replace method using standard and large molds for marl #2

In addition to the CIV tests at the center of the mold, tests were performed at points located between the center and the edge of the mold. The CIV values for different locations are almost the same, as shown in Figs. 4.90 to 4.93. This indicates that only slight effect of mold confinement and wall restraints on the CIV values was observed. This is not in agreement with the CBR values since the CBR tests produces large deformation and plastic flow of material and thus the confinement of the walls will increase the resistance to deformation and the resulting CBR is expected to be higher.

4.5 Comparison between the commonly used Correction Methods

Compaction control for construction necessitates that the fieldwork should meet a certain percentage of the maximum dry density as obtained by a standard test procedures such as ASTM D698 or AASHTO T99. For gravelly soils or soils with oversize particles, the value of this reference dry density is usually accounted for using certain correction procedures after conducting the laboratory test on the finer fraction only or material passing the ¾ in. (19 mm) sieve. Certain correction methods were proposed by different standards, but the adequacy of these correction methods for use with different soils, must be investigated prior to their implementation. In this research program, four different correction methods were selected in order to study their applicability for calcareous sediments. The correction methods used in this testing program were:

(1) The scalp and replace method (ASTM 1557, 1991).

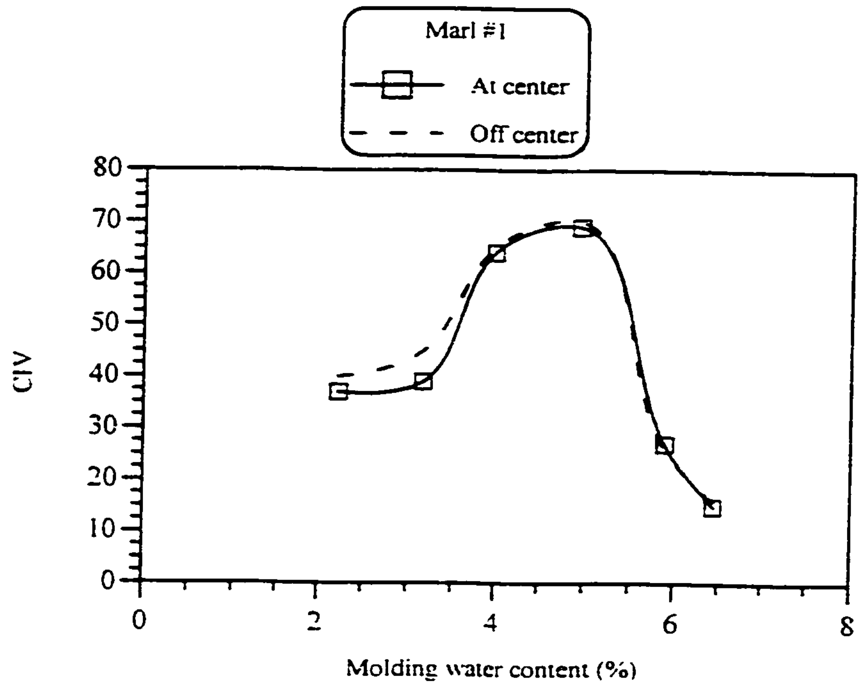


Figure 4.90: CIV-moisture relationships at center and off center points for material prepared by scalp and replace method using the large mold for marl #1

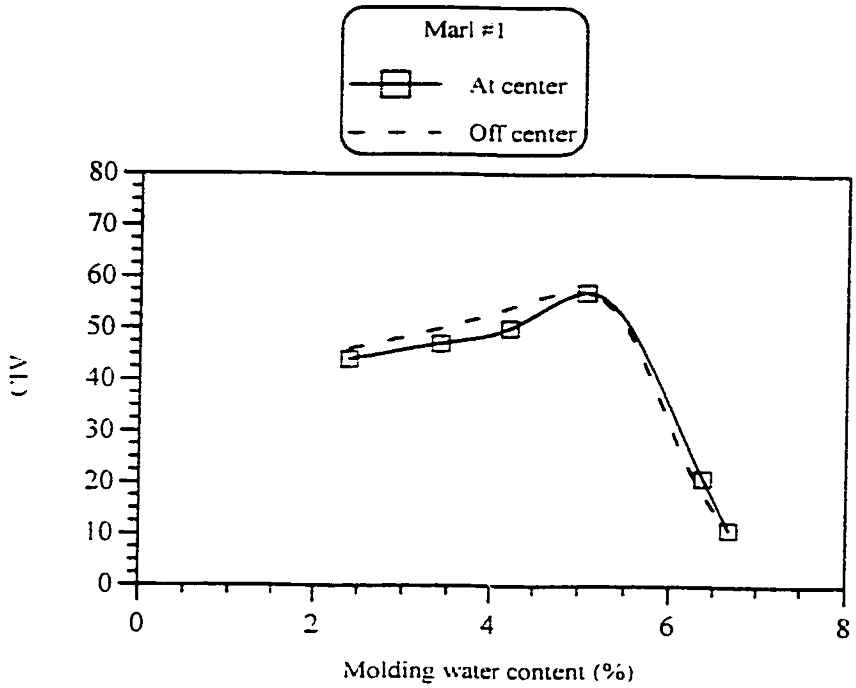


Figure 4.91: CIV-moisture relationships at center and off center points for material prepared in the large mold using the entire gradation for marl #1

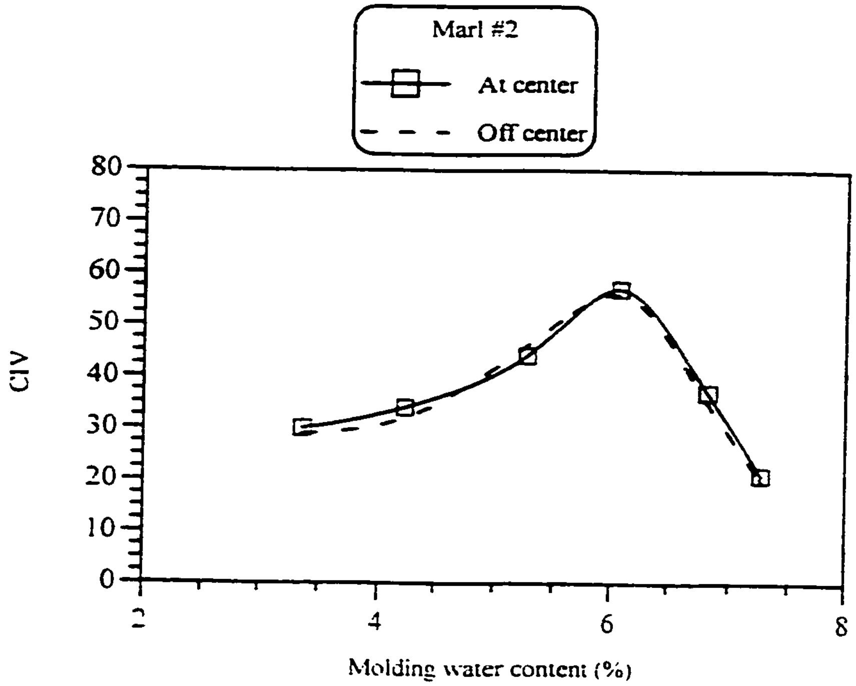


Figure 4.92: CIV-moisture relationships at center and off center points for material prepared by scalp and replace method using the large mold for marl #2

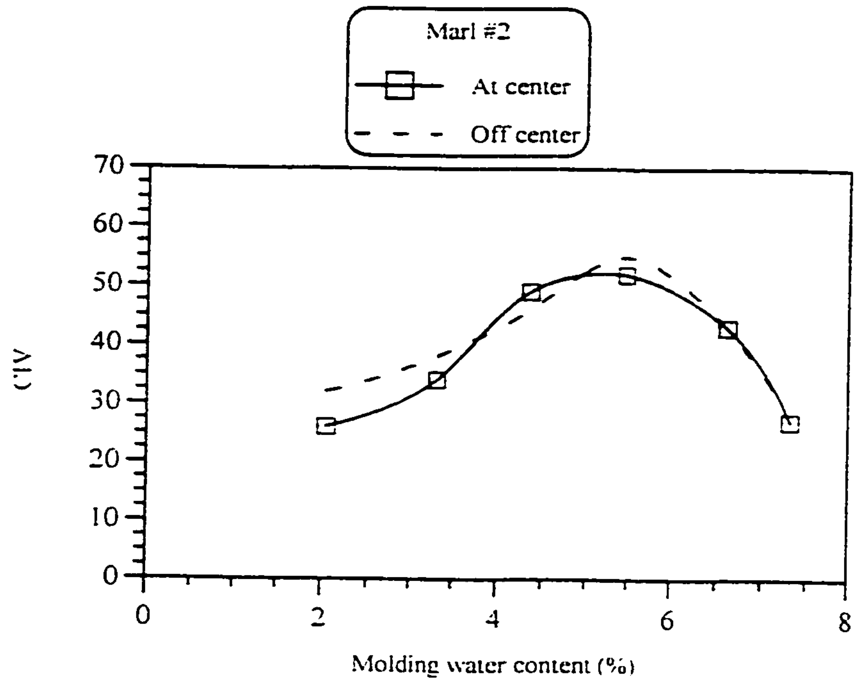


Figure 4.93: CIV-moisture relationships at center and off center points for material prepared in the large mold using the entire gradation for marl #2

(2) ASTM D4718 method.

(3) AASHTO T224-method 1(using an empirical equation).

(4) AASHTO T224-method 2.

In order to apply the correction equations on the selected marls, samples were prepared using the elimination method, whereby the oversize particles (material retained on the ¾ in. sieve) were excluded. The material was reconstituted to the medium gradation.

4.5.1 Correction of the Dry Densities using Scalp and Replace Method

Scalp and replace method was performed for the two marls utilizing the medium gradation. In this method, the material retained on the ¾ in. (19 mm) sieve is excluded from the sample and then replaced with the same mass of material passing the ¾ in (19 mm) sieve and retained on the No. 4 sieve. The moisture density relationships obtained using the elimination method accompanied with the corrected curves using scalp and replace method in addition to the compaction curve obtained using the large mold for the entire gradation, are shown, for both marls, in Figs. 4.94 and 4.95. As shown in the figures, the scalp and replace method gave good approximation for moisture density relationships, for the entire material. The maximum differences between the densities were 0.04% and 0.9% for Marl #1 and Marl #2, respectively.

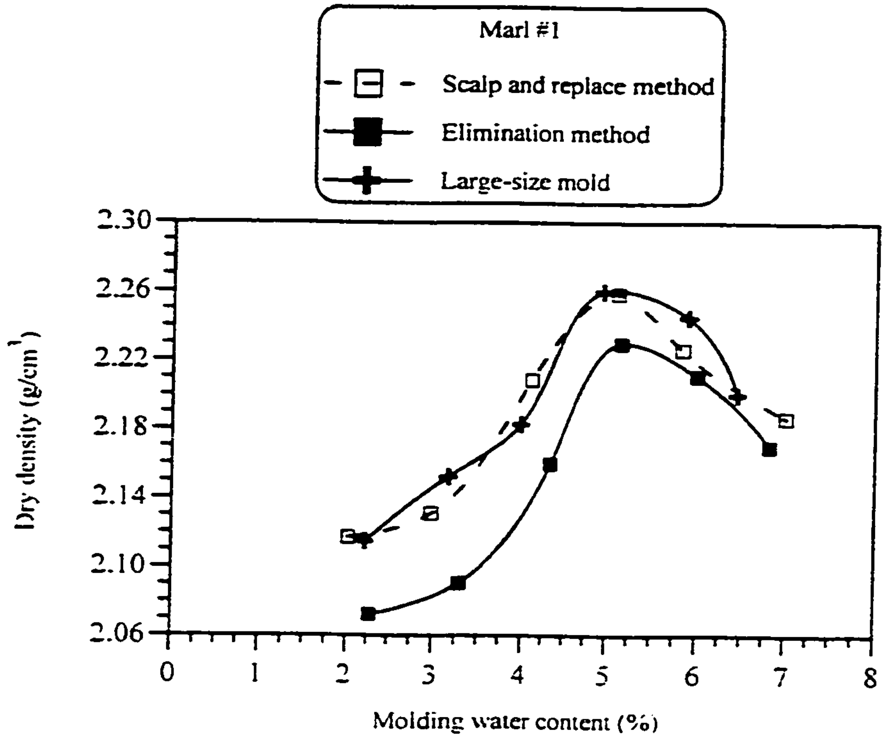


Figure 4.94: The moisture-density relationships obtained using the scalp and replace method, elimination method and large mold for marl #1

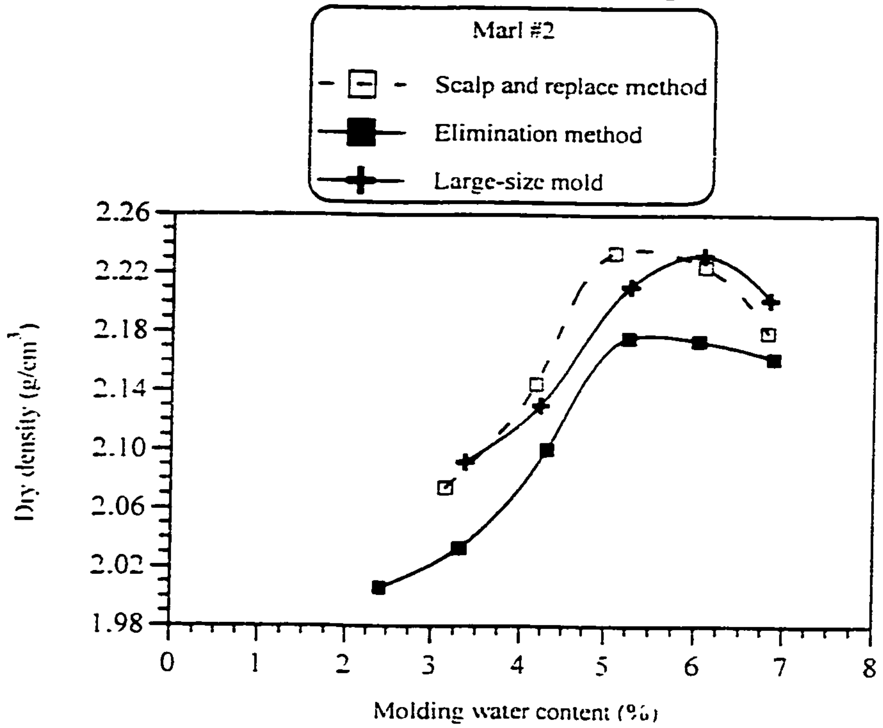


Figure 4.95: The moisture-density relationships obtained using the scalp and replace method, elimination method and large mold for marl #2

4.5.2 Correction of the Dry Densities using ASTM Equation

The American Society for Testing and Materials (ASTM) suggested the following equation to correct for discarding the oversize particles from the test specimen:

$$D = \frac{\gamma_w}{\frac{P_c}{G_m} + \frac{\gamma_w (1 - P_c)}{D_f}} \qquad (4.1)$$

where,

D = Dry density of total soil

D_f = Dry density of the fine material (material passing the ¾ in. (19 mm) sieve)

P_c = Percent of oversize particles by weight (decimal)

G_m = Bulk specific gravity of coarse particles

γ_w = Unit weight of water

Using this equation to correct for the dry densities obtained using the elimination method, new curves, for both marls, were obtained as shown in Figs. 4.96 and 4.97, in addition to the compaction curves obtained using the large mold for the entire gradation. A general increase of dry density values is observed when using the correction equation as shown in the figures. The maximum differences between the dry density values obtained using the correction equation and the large mold, were 1.5 and 1.12% for Marl #1 and Marl #2, respectively. Hence, the ASTM correction equation gave a reasonable approximation for the dry density values of the entire material.

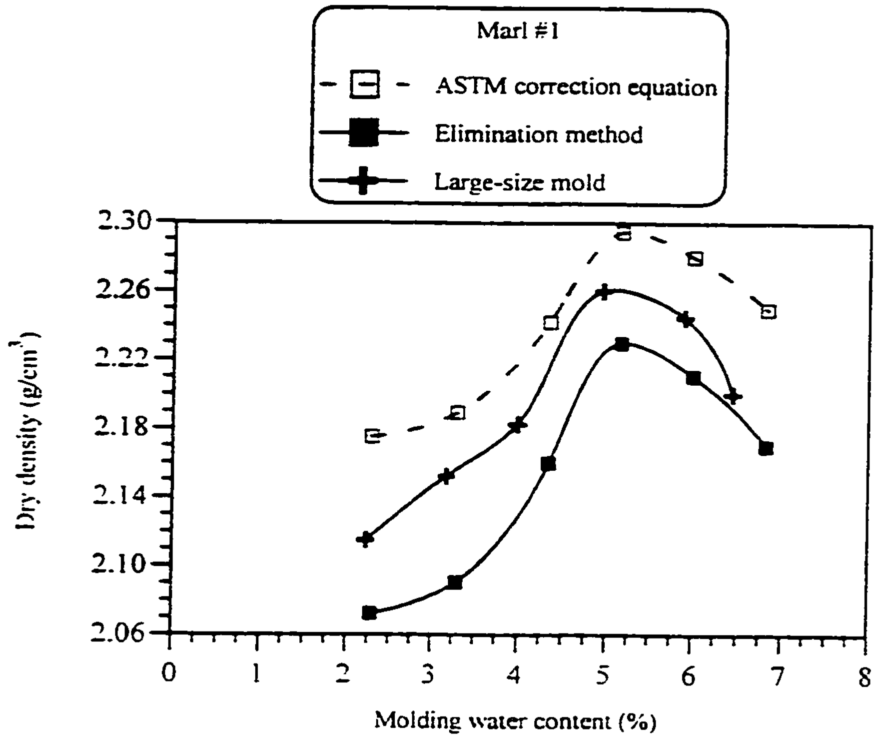


Figure 4.96: The moisture-density relationships obtained using the ASTM correction equation, elimination method and large mold for marl #1

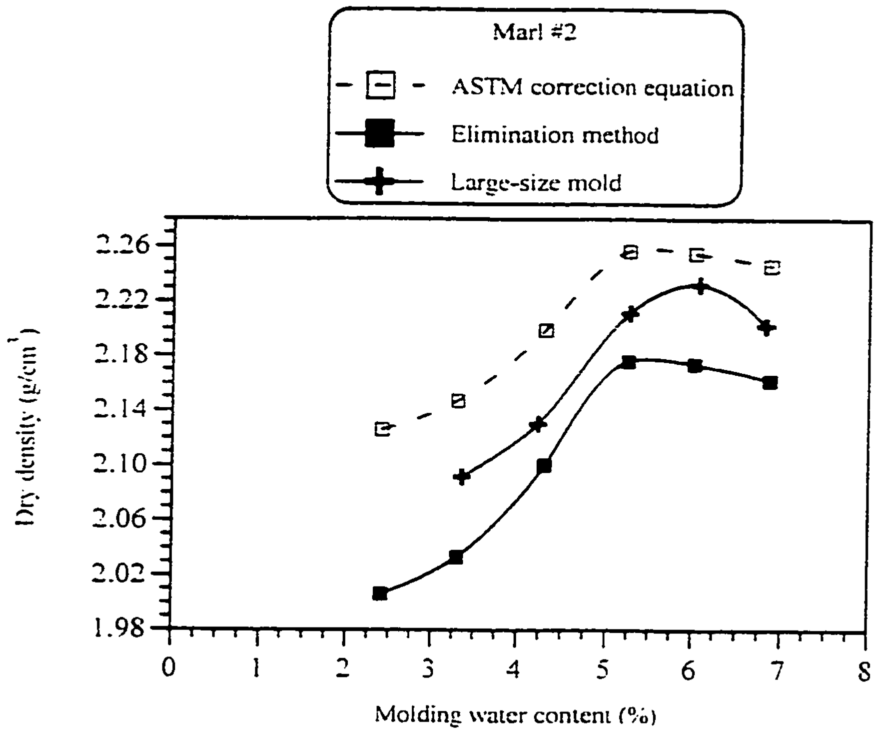


Figure 4.97: The moisture-density relationships obtained using the ASTM correction equation, elimination method and large mold for marl #2

4.5.3 Correction of the Dry Densities using the AASHTO Equations

The American Association of State Highway and Transportation Officials (AASHTO) suggests the following empirical equations to correct for the absence of the oversize particles in the tested specimen:

$$D = (1-P_c)D_f \div 0.9\, P_c\, (\gamma_w)G_m \qquad (4.2)$$

For better predicted values AASHTO suggests the following equation:

$$D = \frac{\gamma_w}{\dfrac{P_c}{G_m} + \dfrac{\gamma_w(1-P_c)}{r_a\, D_f}} \qquad (4.3)$$

where,

D = Dry density of the total soil

D_f = Dry density of the fine material (material passing the ¾ in. (19 mm) sieve)

P_c = Percent rock by weight (decimal)

G_m = Bulk specific gravity of rock

r_a = Correction factor in AASHTO equation to account for interference of large aggregate

γ_w = Unit weight of water

The compaction curves, which were produced using these equations, compared to the uncorrected curves as well as the compaction curves obtained from the large mold for the entire gradation, are shown in Figs. 4.98 to 4.101, for both marls. For AASTO-1 equation,

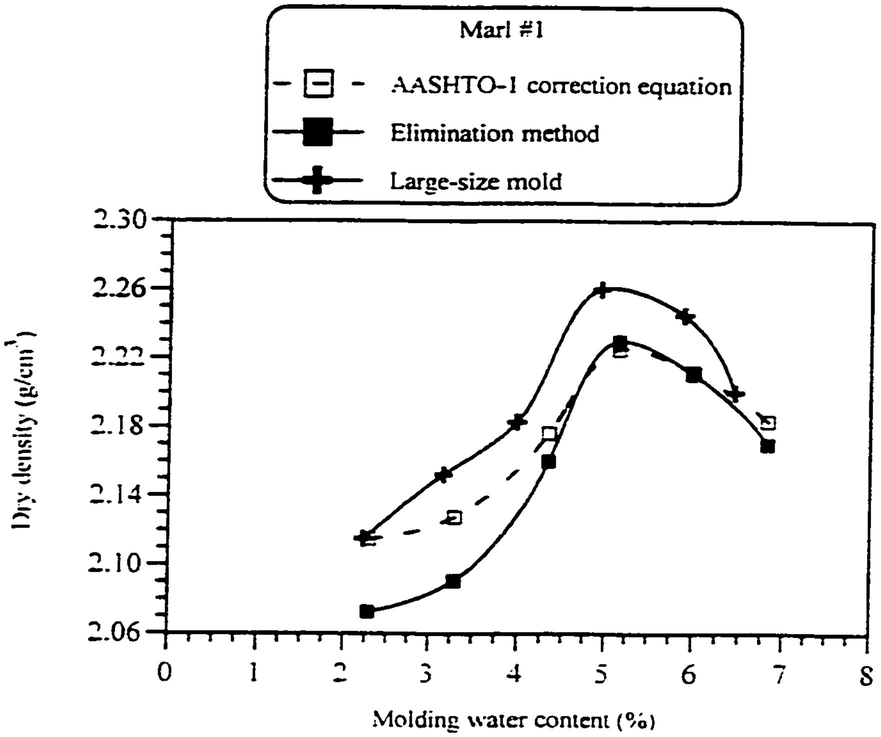


Figure 4.98: The moisture-density relationships obtained using the AASHTO-1 correction equation, elimination method and large mold for marl #1

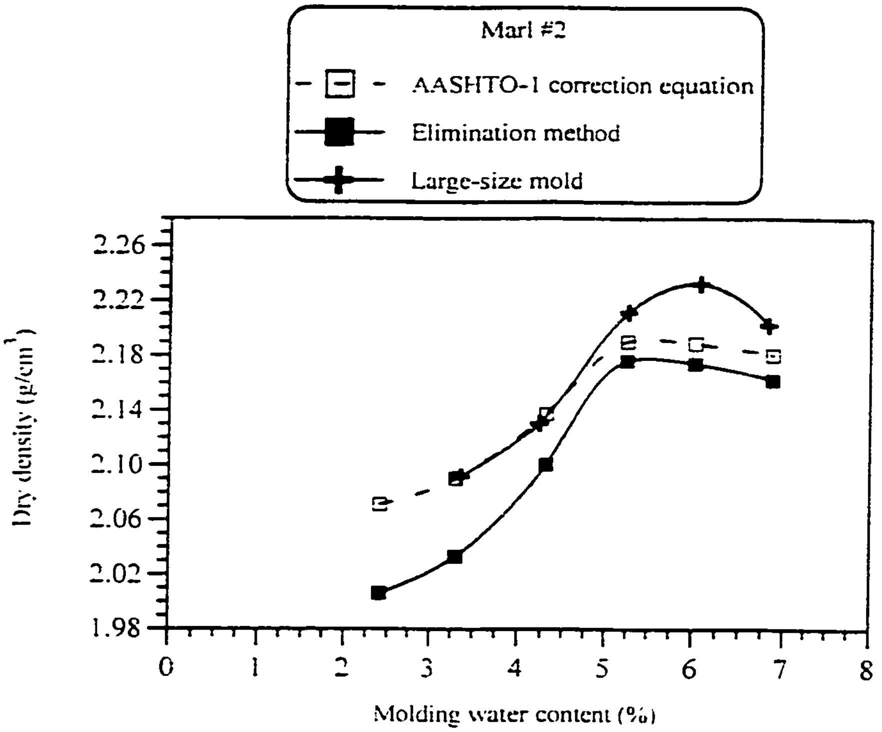


Figure 4.99: The moisture-density relationships obtained using the AASHTO-1 correction equation, elimination method and large mold for marl #2

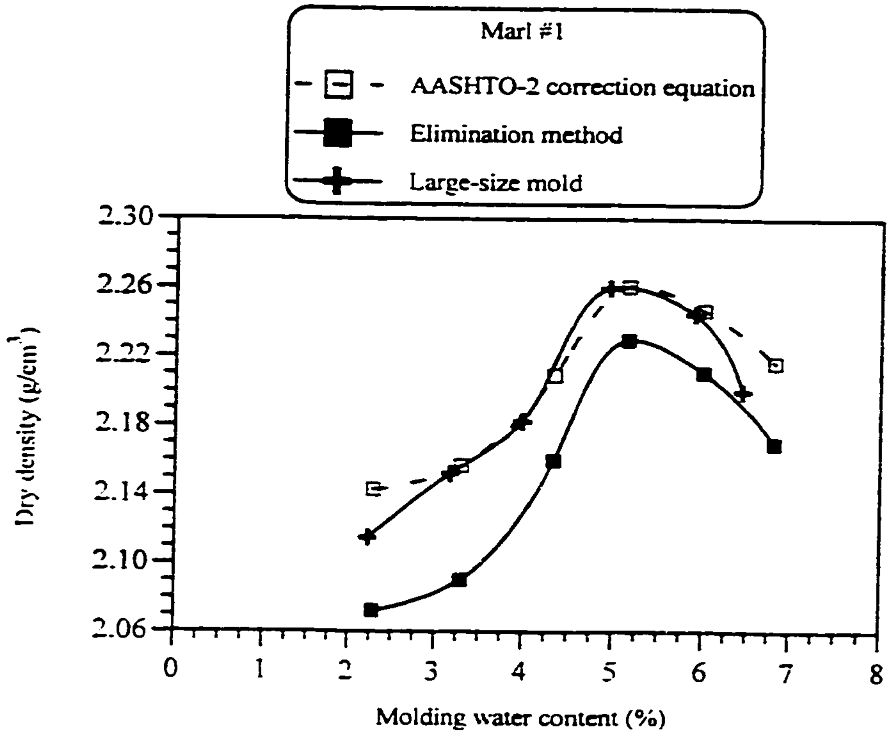


Figure 4.100: The moisture-density relationships obtained using the AASHTO-2 correction equation, elimination method and large mold for marl #1

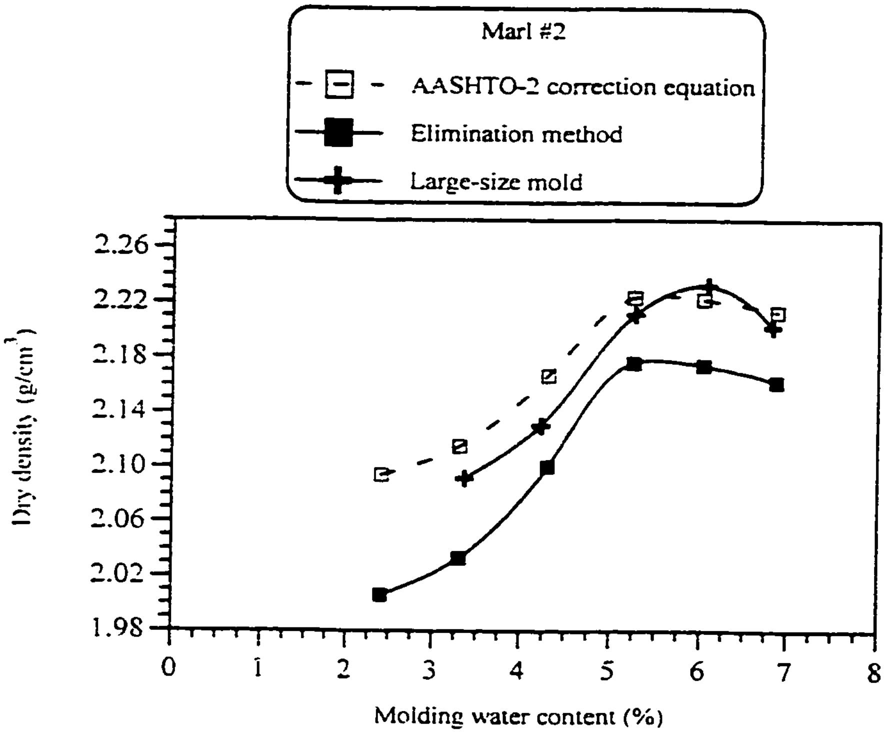


Figure 4.101: The moisture-density relationships obtained using the AASHTO-2 correction equation, elimination method and large mold for marl #2

the maximum differences between the dry density values obtained using the equation and the large mold, were 1.54 and 1.88% for Marl #1 and Marl #2, respectively. While for AASHTO-2 equation, the maximum differences were 0.04 and 0.9% for Marl #1 and Marl #2, respectively. Hence AASHTO-2 equation also gave a good approximation for the entire material moisture-density relationships.

The corrected curves, using the four methods, for both marls are compared together, as shown in Figs. 4.102 and 4.103 for the two marls. In addition, the moisture density relationships obtained using the entire material, compacted in the large size mold are also plotted. As shown in the figures, for both marls the ASTM correction equation gave the highest dry density values, while the AASHTO-1 equation gave the lowest dry densities values. The curves obtained using the scalp and replace method, AASHTO-2 equation and the large size mold, are very close to each other. Hence the ASTM equation is believed to overestimate the values for dry density while the AASHTO-1 equation underestimates the values. The scalp and replace method gave an adequate dry density values compared with the results obtained from the large size mold. Furthermore AASHTO-2 equation shows a good approximation for the dry density values when compared with that obtained from the large size mold.

By comparing the differences between the maximum dry densities obtained when using the correction methods and the entire material compacted in the large mold. It is found that the maximum difference obtained from all methods is less than 2% of the maximum dry density obtained from the large size mold. Hence the used correction

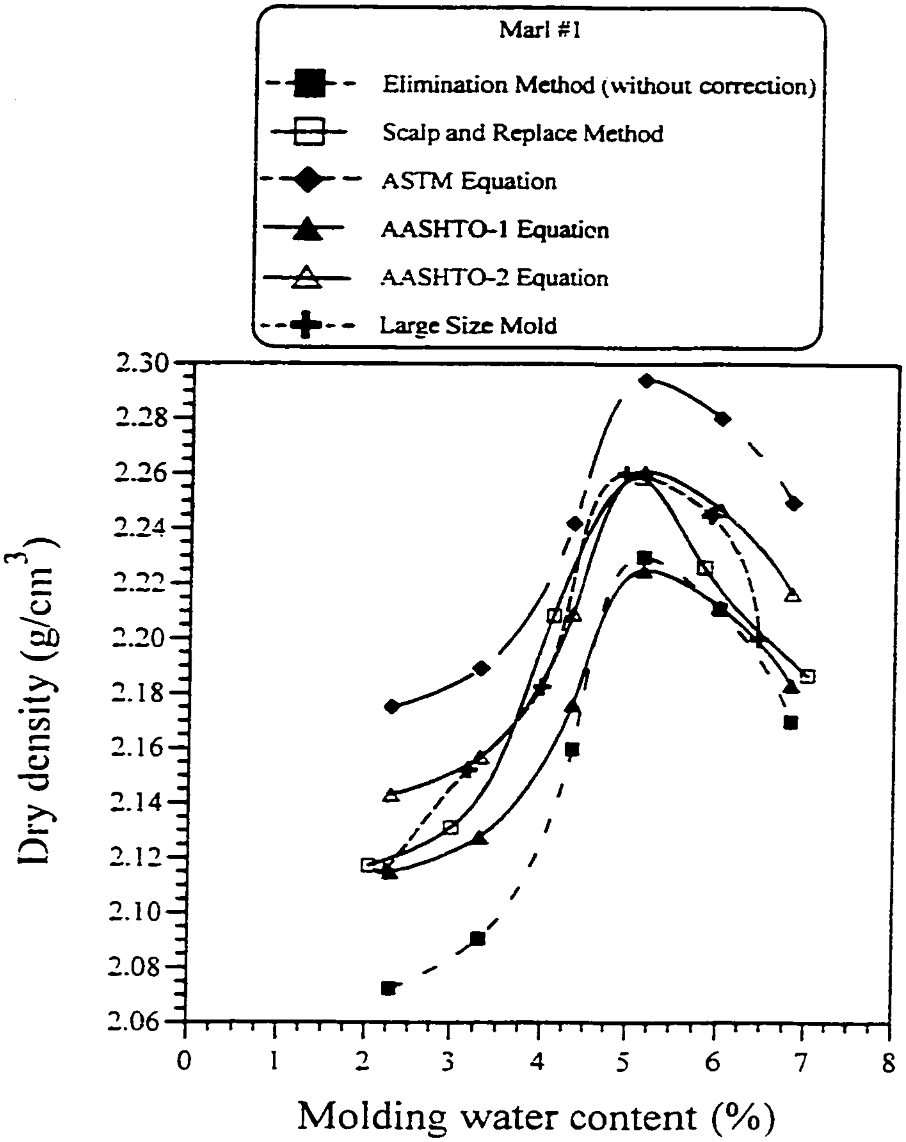


Figure 4.102: The moisture-density relationships obtained using the different correction methods and the elimination method for marl #1

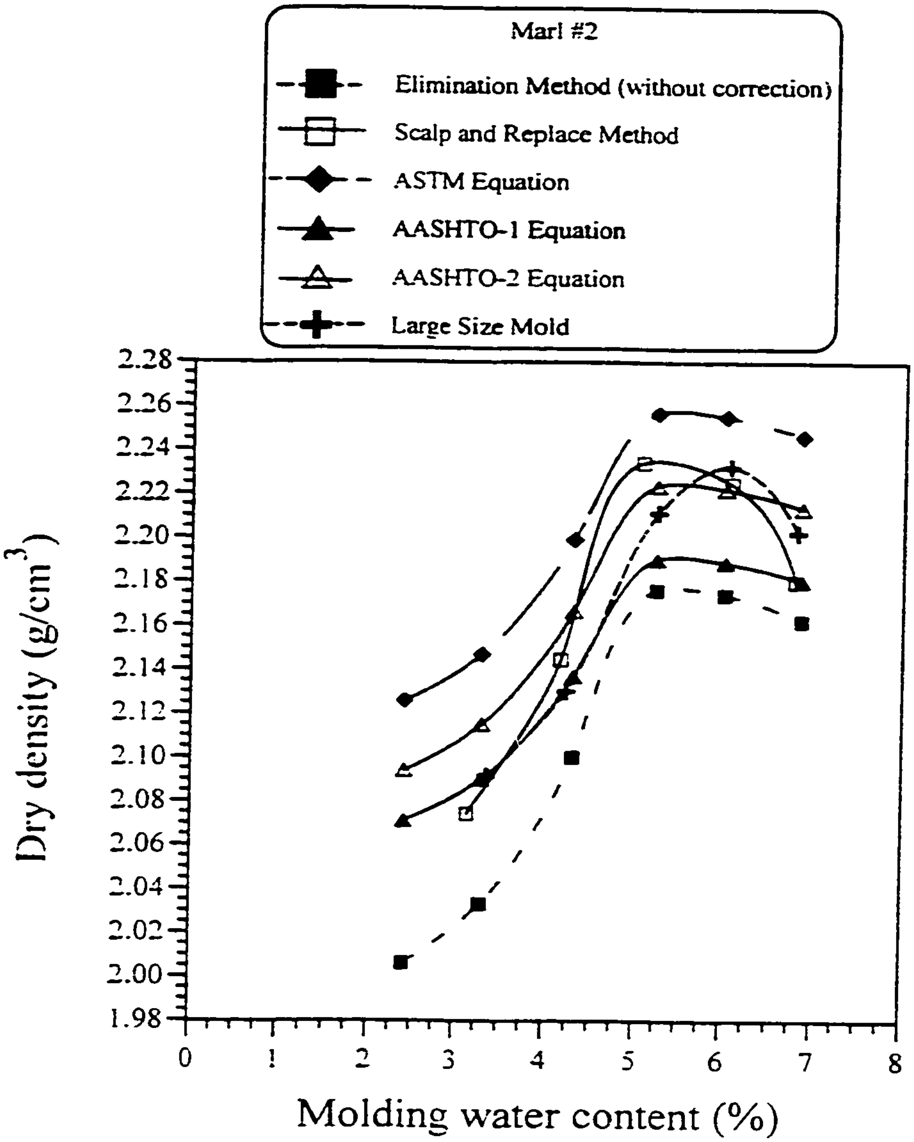


Figure 4.103: The moisture-density relationships obtained using the different correction methods and the elimination method for marl #2

methods are applicable for calcareous soils, however ASTM equation and AASHTO-2 equation gave overestimated and underestimated density values, respectively.

4.6 The effects of Scalp and Replace Method on the CBR and CIV values

Scalp and replace method is the commonly used oversize correction method. However, this method was not adopted in the recent editions of the ASTM standard (ASTM, 2000). In this research program, a comparison was done between the CBR and CIV values for material prepared using the elimination method (ASTM D1557, method C), the scalp and replace method using the standard and large molds and the entire gradation prepared in the large mold. The moisture density relationships obtained using the elimination method indicate lower values of dry densities compared to scalp and replace method using the medium gradation. In addition, a decrease in the unsoaked CBR and the CIV values was observed upon the use of the elimination method compared to those obtained when using the scalp and replace method as shown in Figs 4.104 to 4.107, for both marls. This decrease in dry densities, CBR and CIV values is attributed to elimination of the large particles without making any substitution for them.

As shown in the figures, when comparing the CBR and CIV values obtained using these preparation methods with the values obtained using the large mold, for different preparation methods, it is clear that the large mold gave the lowest CBR and CIV values as a result of the minimization of the boundary conditions in the large mold. The maximum CBR and CIV values obtained using the two methods and the large mold are

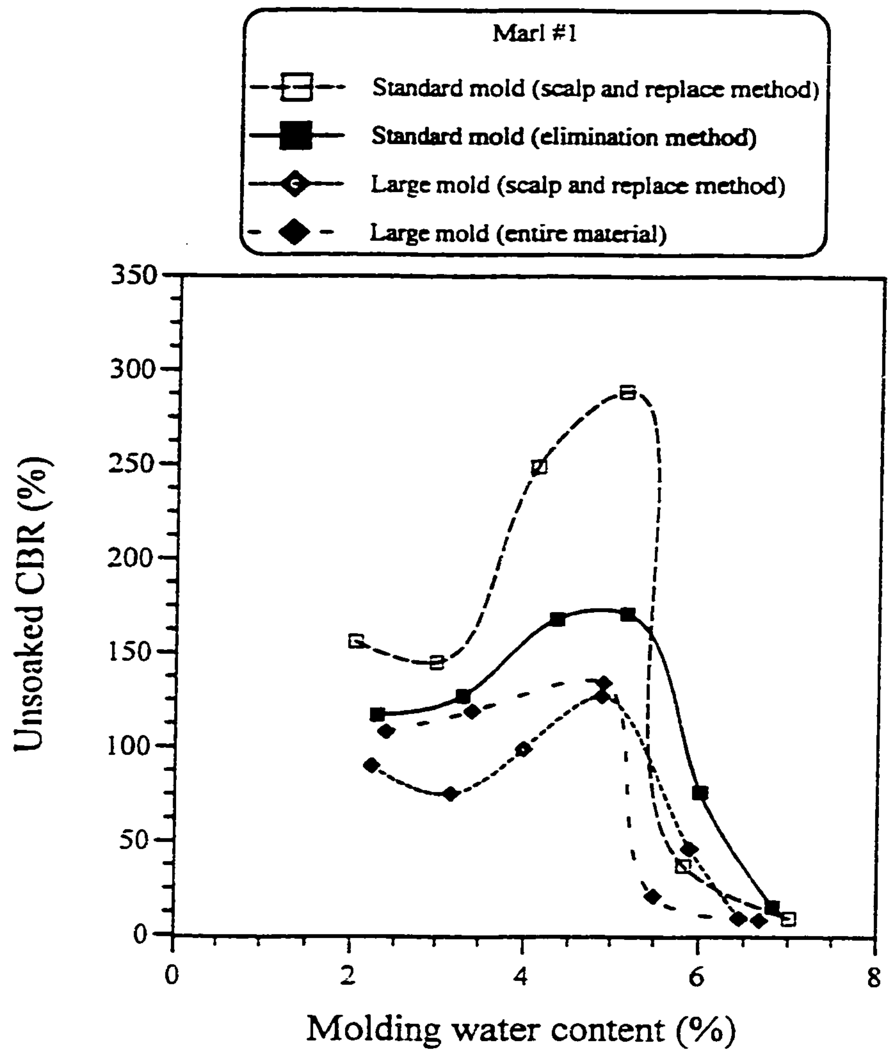


Figure 4.104: The CBR-moisture relationships obtained using standard mold (scalp and replace and elimination methods) and large mold (scalped and replaced and entire materials) for marl #1

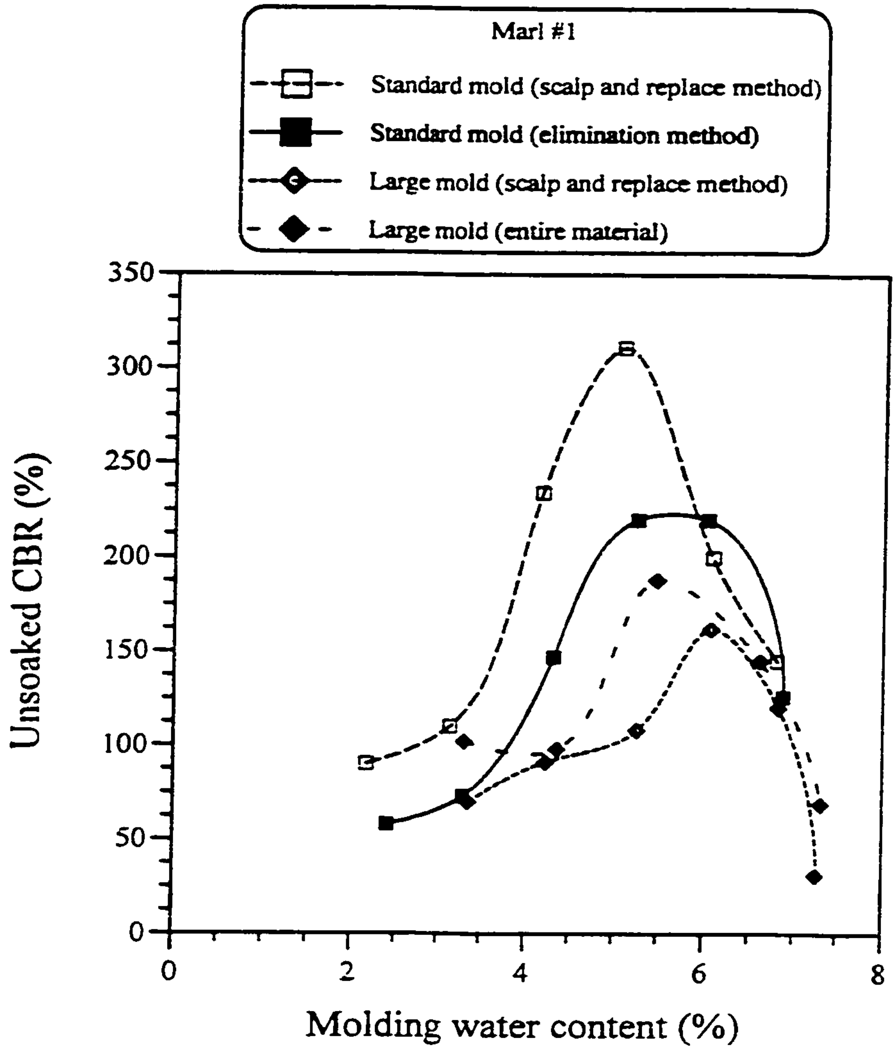


Figure 4.105: The CBR-moisture relationships obtained using standard mold (scalp and replace and elimination methods) and large mold (scalped and replaced and entire materials) for marl #2

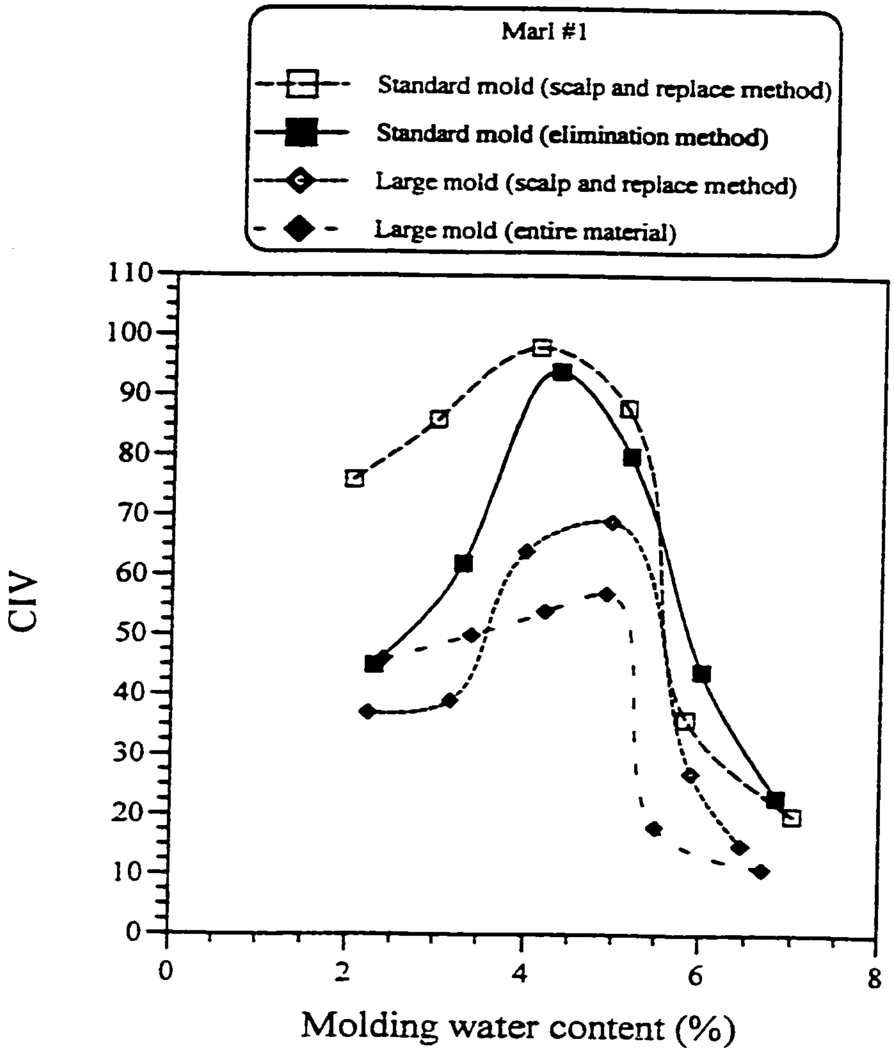


Figure 4.106: The CIV-moisture relationships obtained using standard mold (scalp and replace and elimination methods) and large mold (scalped and replaced and entire materials) for marl #1

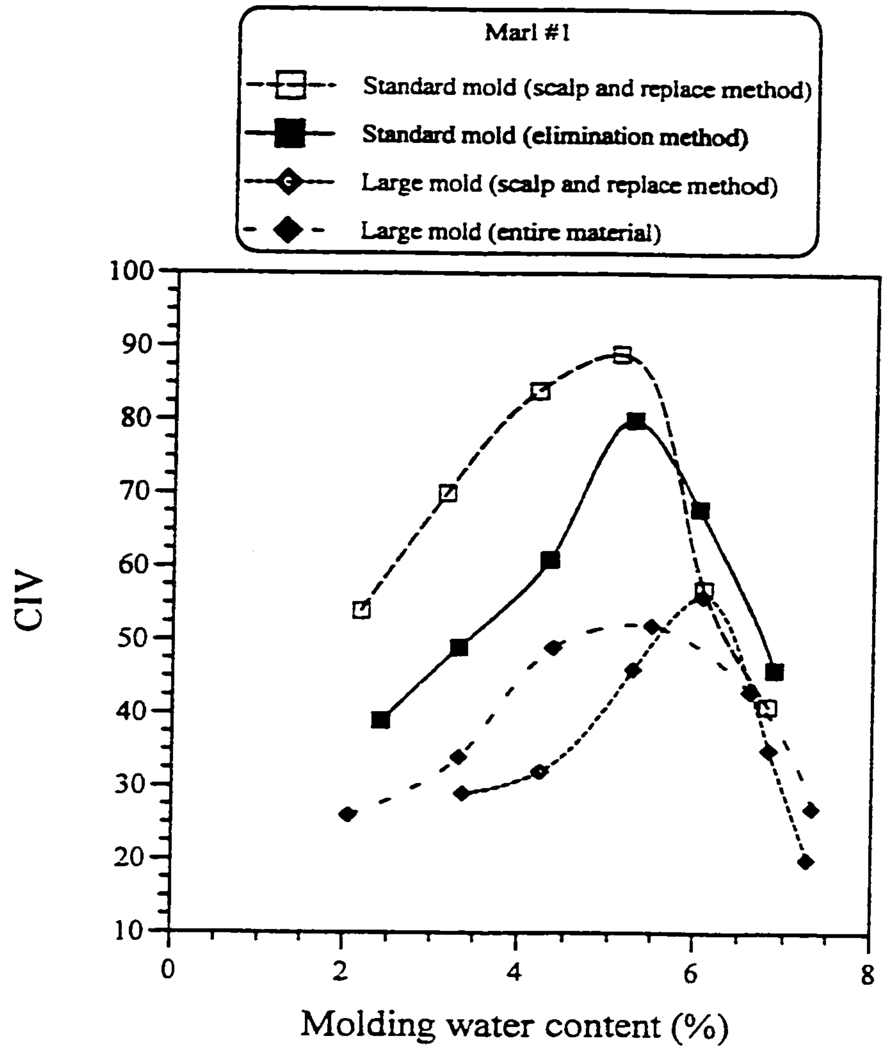


Figure 4.107: The CIV-moisture relationships obtained using standard mold (scalp and replace and elimination methods) and large mold (scalped and replaced and entire materials) for marl #2

shown in Table 4.6, for both marls. As shown in the table, the effect of replacing the oversize particles with finer ones did not have the same impact on the obtained CBR and CIV values. Decreases of 41 and 28% in the maximum CBR values were observed for samples prepared with replacement for Marl #1 and Marl #2, respectively when the same material was tested without replacement (using the elimination method). However only reductions of 4 and 10% were observed in the maximum CIV values, for materials prepared with replacement, for Marl #1 and Marl #2, respectively when the same material was tested without replacement. It is also observed that the maximum CIV values obtained for each mold size did not show remarkable variations with the change of the preparation method using the same mold size.

Table 4.6: The CBR_{max} and CIV_{max} values for the two marls for the medium gradation using scalp and replace and elimination methods for the standard mold and using scalp and replace method and entire gradation for the large mold

Preparation Method	Marl #1		Marl #2	
	CBR_{max} (%)	CIV_{max}	CBR_{max} (%)	CIV_{max}
Scalp and Replace Method	289	98	311	89
Elimination Method	171	94	224	80
Large Mold (scalp and replace Method)	127	69	162	56
Large Mold (Entire Gradation)	134	57	188	52

4.7 Statistical Analysis

Experimental work is usually performed by conducting a test or series of tests in which changes are made to the input variables of a certain process or system so that the reasons for changes in the output may be identified. In such research program, the experiment results can be described using a statistical model.

Using linear statistical model, the effect of marl type, soil gradation and moisture content, on the CBR, UCS, and CIV values was investigated for the conventional tests. In addition, the effect of marl type, preparation method and moisture content value on the dry density, CBR and CIV values was also studied for the large-size setup. Furthermore the contribution of marl type, mold size and moisture content value on dry density and CBR values was studied. Despite knowing that the moisture content value has high level of significance on these variables, its effect was studied to assess the reliability of the used methodology. The model used is a three-factor model with only single replicate. Assuming no interaction between the different factors, the three-factor analysis of variance model will be,

$$y_{ijk} = \mu + \tau_i + \beta_j + \nu_k + \varepsilon_{ijk} \qquad \begin{cases} i = 1,2 \\ j = 1,2,3 \\ k = 1,\ldots,6 \end{cases}$$

where,

μ = the overall mean effect

τ_i = the effect of the i-th marl type

$\beta_j =$ the effect of the j-th parameter (gradation, preparation method…etc)

$\nu_k =$ the effect of the k-th moisture content value

$\epsilon_{ijk} =$ the random computed error

The linear statistical model used is found to be appropriate for data with more than one factor and only one observation per cell (Montgomery, 1997).

4.7.1 Hypothesis Testing

A statistical hypothesis is a statement about the parameters of a probability distribution. For example, if we have two values, the values may or may not be equal. This may be stated formally as:

H_0: $\mu_1 = \mu_2$

H_1: $\mu_1 \neq \mu_2$

Where μ_1 and μ_2 are the means of the response variables (CBR, UCS, CIV or γ_{dry}) for Marl #1 and Marl #2 respectively. The statement H_0:$\mu_1 = \mu_2$ is called the null hypothesis and H_1:$\mu_1 \neq \mu_2$ is called the alternative hypothesis. To test a hypothesis a procedure was devised by taking a random sample, computing the appropriate test statistic, and then rejecting or failing to reject the null hypothesis H_0 (Montgomery, 1997).

P-value approach has been adopted widely in practice. It is defined as the smallest level of significance that would lead to rejection of the null hypothesis H_0. The analysis of variance (ANOVA) were conducted using the software STATISTICA. The data and ANOVA tables are shown in Tables 4.7 to 4.9. The significance of each parameter was determined at a confidence level of 95%.

For the CBR tests, only the gradation of the tested marls shows insignificant effect on the obtained CBR values. Hence, changing soil gradation within the gradation limits has no impact on the produced CBR values. This substantiates the efficiency of the gradation limits provided by Dammam Municipality. While for the UCS test, all parameters show a remarkable significance on the obtained results. Hence the UCS values are found to be more sensitive to the change of marl type and gradation than the CBR test. This is attributed to the significant effect of the amounts of fine and their plasticity on the UCS values. For the Clegg Impact test, it is clear that changing marl type and soil gradation did not alter the produced CIV values significantly.

For the large size mold, the data and the ANOVA tables for the γ_{dry}, CBR and CIV are shown in Tables 4.10 to 4.12. The parameters studied were the marl type, preparation method (scalp and replace and entire material) and moisture content. For the dry density, the preparation method shows no significance on the obtained results, which substantiate the efficiency of the scalp and replace method as an oversize correction method. For the CBR results, all parameters show high level of significance on the obtained results. It is clear that utilizing all particle sizes has a remarkable impact on the obtained CBR values compared to the values obtained using the scalp and replace method. Furthermore, marl type and preparation method show no valuable contribution on the produced CIV results.

Table: 4.7-a. The CBR data for the two tested marls for the small mold

	Marl #1																	
	Fine Limit Gradation						Medium Gradation						Coarse Limit Gradation					
m.c%	2	3	4	5	6	7	2	3	4	5	6	7	2	3	4	5	6	7
CBR%	119	146	218	188	50	13	156	144	238	288	32	9	134	167	195	261	50	18

	Marl #2																	
	Fine Limit Gradation						Medium Gradation						Coarse Limit Gradation					
m.c%	2	3	4	5	6	7	2	3	4	5	6	7	2	3	4	5	6	7
CBR%	107	118	154	320	244	163	88	107	207	308	209	136	89	102	163	238	207	111

Table: 4.7.b. ANOVA Table for the CBR test for the small mold

	Effect (F/R)	df Effect	MS Effect	df Error	MS Error	F	P-value
{1} Marl Type	Fixed	1	11556.3	27	3066.96	3.76798	**0.06275**
{2} Soil Gradation	Fixed	2	732.194	27	3066.96	0.23874	**0.78927**
{3} Moisture Content	Fixed	5	27957.8	27	3066.96	9.11582	**3.6E-05**

Table: 4.8-a. The UCS data for the two tested marls for the small mold

	Marl #1																	
	Fine Limit Gradation						Medium Gradation						Coarse Limit Gradation					
m.c%	2	3	4	5	6	7	2	3	4	5	6	7	2	3	4	5	6	7
UCS gm/cm^3	388	450	388	269	107	38	625	688	650	400	126	63	676	850	787	525	126	65

	Marl #2																	
	Fine Limit Gradation						Medium Gradation						Coarse Limit Gradation					
m.c%	2	3	4	5	6	7	2	3	4	5	6	7	2	3	4	5	6	7
UCS gm/cm^3	131	194	238	231	177	52	181	338	344	319	250	106	231	338	350	338	300	163

Table: 4.8.b. ANOVA Table for the UCS test for the small mold

	Effect (F/R)	df Effect	MS Effect	df Error	MS Error	F	P-value
{1} Marl Type	Fixed	1	240100	27	17172.7	13.9815	0.00088
{2} Soil Gradation	Fixed	2	94750.1	27	17172.7	5.51748	0.00979
{3} Moisture Content	Fixed	5	148433	27	17172.7	8.64354	5.5E-05

Table: 4.9-a. The CIV data for the two tested marls for the small mold

	Marl #1																	
	Fine Limit Gradation						Medium Gradation						Coarse Limit Gradation					
m.c%	2	3	4	5	6	7	2	3	4	5	6	7	2	3	4	5	6	7
CIV	62	75	78	63	36	22	76	86	98	90	33	20	60	80	88	92	46	24

	Marl #2																	
	Fine Limit Gradation						Medium Gradation						Coarse Limit Gradation					
m.c%	2	3	4	5	6	7	2	3	4	5	6	7	2	3	4	5	6	7
CIV	43	55	90	92	78	49	51	68	83	89	58	38	49	57	67	90	82	63

Table: 4.9.b. ANOVA Table for the CIV test for the small mold

	Effect (F/R)	df Effect	MS Effect	df Error	MS Error	F	P-value
{1} Marl Type	Fixed	1	148.028	27	225.184	0.65736	**0.42458**
{2} Soil Gradation	Fixed	2	73.5833	27	225.184	0.32677	**0.72406**
{3} Moisture Content	Fixed	5	2191.52	27	225.184	9.73211	**2.1E-05**

Table: 4.10-a. The Dry Density data for the large-size mold

	Marl #1										Marl #2									
	Scalp and Replace Method					Entire Gradation					Scalp and Replace Method					Entire Gradation				
m.c%	3	4	5	6	7	3	4	5	6	7	3	4	5	6	7	3	4	5	6	7
γdry	2.15	2.19	2.26	2.24	2.16	2.13	2.16	2.26	2.26	2.25	2.08	2.12	2.2	2.24	2.19	2.13	2.14	2.18	2.23	2.24

Table: 4.10-b. ANOVA Table for the Dry Density values for the large-size mold

	Effect (F/R)	df Effect	MS Effect	df Error	MS Error	F	p
{1} Marl Type	Fixed	1	0.004805	4	0.000632	7.596838	**0.051046**
{2} Preparation Method	Fixed	1	0.001125	4	0.000632	1.778656	**0.25319**
{3} Moisture Content	Fixed	4	0.010343	4	0.000632	16.35178	**0.009581**

Table: 4.11-a. The CBR data for the large-size mold

	Marl #1										Marl #2									
	Scalp and Replace Method					Entire Gradation					Scalp and Replace Method					Entire Gradation				
m.c%	3	4	5	6	7	3	4	5	6	7	3	4	5	6	7	3	4	5	6	7
CBR%	75	98	128	40	2	115	125	133	18	8	60	85	100	159	100	106	97	137	175	119

Table: 4.11-b. ANOVA Table for CBR values for the large-size mold

	Effect (F/R)	df Effect	MS Effect	df Error	MS Error	F	p
{1} Marl Type	Fixed	1	7840.8	4	112.675	69.58775	**0.001129**
{2} Preparation Method	Fixed	1	1729.8	4	112.675	15.35212	**0.017274**
{3} Moisture Content	Fixed	4	2374.375	4	112.675	21.07278	**0.005972**

Table: 4.12-a. The CIV data for the large-size mold

	Marl #1										Marl #2									
	Scalp and Replace Method					Entire Gradation					Scalp and Replace Method					Entire Gradation				
m.c%	3	4	5	6	7	3	4	5	6	7	3	4	5	6	7	3	4	5	6	7
CIV	38	65	69	24	8	48	53	58	15	11	28	31	40	54	28	32	45	51	50	36

Table: 4.12-b. ANOVA Table for CIV values for the large-size mold

	Effect (F/R)	df Effect	MS Effect	df Error	MS Error	F	p
{1} Marl Type	Fixed	1	1.8	4	44.075	0.040839	0.84971
{2} Preparation Method	Fixed	1	9.8	4	44.075	0.222348	0.661826
{3} Moisture Content	Fixed	4	680.175	4	44.075	15.43222	0.01066

The effect of marl type, mold size and moisture content on the dry density, CBR and CIV values is studied using ANOVA and are shown in Tables 4.13 to 4.15, accompanied with the data. As shown in the tables, changing the mold size from small to large did not show any significance on the obtained dry density; hence, the large mold can be taken as a possible replacement for the small mold for compaction. However, changing the mold size caused a remarkable differences between the produced CBR values due to the variation of the boundary effects between the two mold sizes. For the CIV values it is noticed that changing the size of the mold, from small to large, has significant impact on the obtained CIV values, in addition the contribution of marl type was increased.

4.7.2 A Proposed Correction Method to Correct for the Effect of Mold Confinement on the CBR values

The CBR results obtained from the conventional small mold size are believed to be overestimated values. Hence, to have reliable correlation between the laboratory and field CBR results, the CBR values obtained using the conventional CBR mold need to be corrected.

A linear regression relation was obtained using the test results obtained using the small and the large molds as shown in Fig. 4.108. Hence the CBR values for the large size mold can be obtained using the CBR values obtained from the conventional mold using the following relation

$$CBR_{Large\text{-}Mold} = 0.3832 CBR_{Small\text{-}Mold} + 26.889 \tag{4.4}$$

Table: 4.13-a. The Dry Density data for the small and large molds

	Marl #1										Marl #2									
	Small Mold					Large Mold					Small Mold					Large Mold				
m.c%	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6
γdry	2.11	2.13	2.21	2.26	2.23	2.11	2.15	2.18	2.26	2.24	2.07	2.14	2.23	2.22	2.18	2.09	2.13	2.21	2.23	2.2

Table: 4.13-b. ANOVA Table for the Dry Density values for the small and large molds

	Effect (F/R)	df Effect	MS Effect	df Error	MS Error	F	P-value
{1} Marl Type	Fixed	1	0.00162	4	9.5E-05	17.05263	**0.0145**
{2} Mold Size	Fixed	1	2E-05	4	9.5E-05	0.210526	**0.67018**
{3} Moisture Content	Fixed	4	0.014745	4	9.5E-05	155.2105	**0.000122**

Table: 4.14-a. The CBR data for the small and large molds

	Marl #1										Marl #2									
	Small Mold					Large Mold					Small Mold					Large Mold				
m.c%	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6
CBR%	156	145	249	289	37	90	75	99	127	46	90	110	234	311	200	70	91	108	162	120

Table: 4.14-b. ANOVA Table for CBR values for the small and large molds

	Effect (F/R)	df Effect	MS Effect	df Error	MS Error	F	P-value
{1} Marl Type	Fixed	1	1674.45	4	811.125	2.064355	**0.224137**
{2} Mold Size	Fixed	1	34694.45	4	811.25	42.77325	**0.002825**
{3} Moisture Content	Fixed	4	12050.68	4	811.125	14.85674	**0.01143**

Table: 4.15-a. The CIV data for the small and large molds

	Marl #1										Marl #2									
	Small Mold					Large Mold					Small Mold					Large Mold				
m.c%	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6	2	3	4	5	6
CIV	76	86	98	88	36	37	39	64	69	27	54	70	84	89	57	29	32	46	56	35

Table: 4.15-b. ANOVA Table for CIV values for the small and large molds

	Effect (F/R)	df Effect	MS Effect	df Error	MS Error	F	P-value
{1} Marl Type	Fixed	1	231.2	4	40.325	5.733416	0.074805
{2} Mold Size	Fixed	1	4620.8	4	40.325	114.589	0.000432
{3} Moisture Content	Fixed	4	982.575	4	40.325	24.3664	0.00454

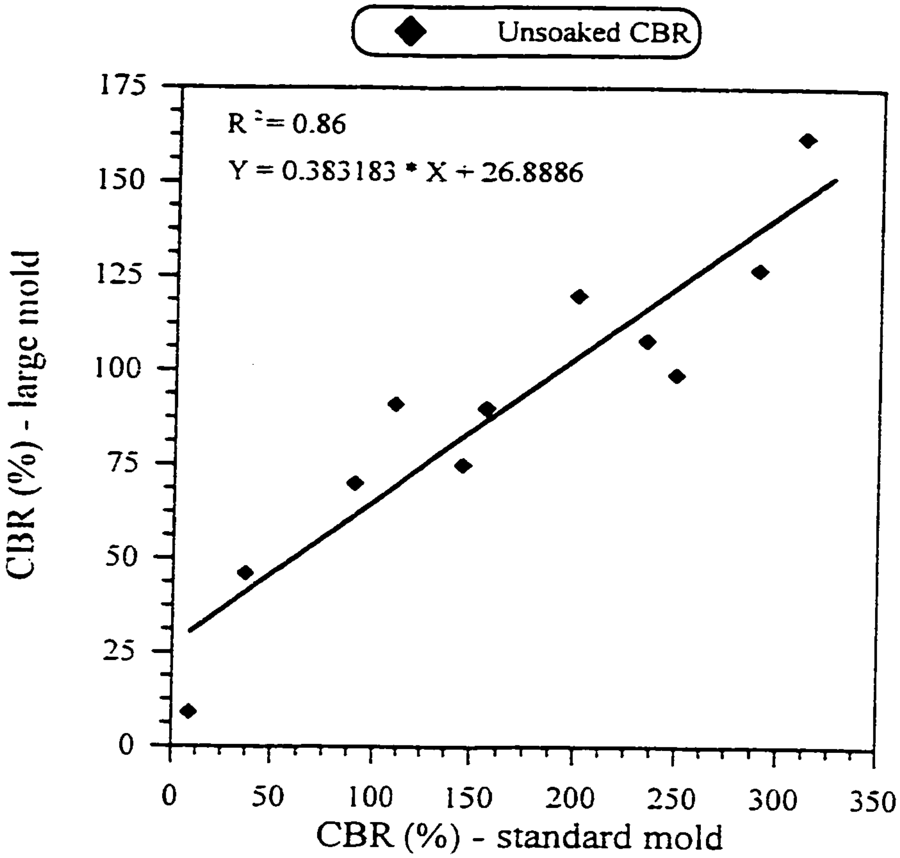


Figure 4.108: The relation between the standard and large molds unsoaked CBR values

The CBR values obtained using the conventional small mold in addition to the experimental and the predicted CBR values for the large size mold are shown in Table 4.16. As shown in the table, the proposed relation gave a reasonable approximation for the large mold CBR values. The experimental versus the predicted CBR values for the large size mold are plotted in Fig. 4.109, which shows that the prediction gave quite close values compared to the actual values despite the limited number of data points. Hence this relation can be used as an initial stage for a reliable correction method to correct for the effect of boundary conditions on the laboratory CBR values.

Table 4.16: CBR values obtained from the small and large molds and the predicted large mold CBR values

CBR (Small Mold)	CBR (Large Mold)	CBR (Predicted Values)
156	90	87
145	75	82
249	99	122
289	127	138
37	46	41
9	9	30
90	70	61
110	91	69
234	108	117
311	162	146
200	120	104

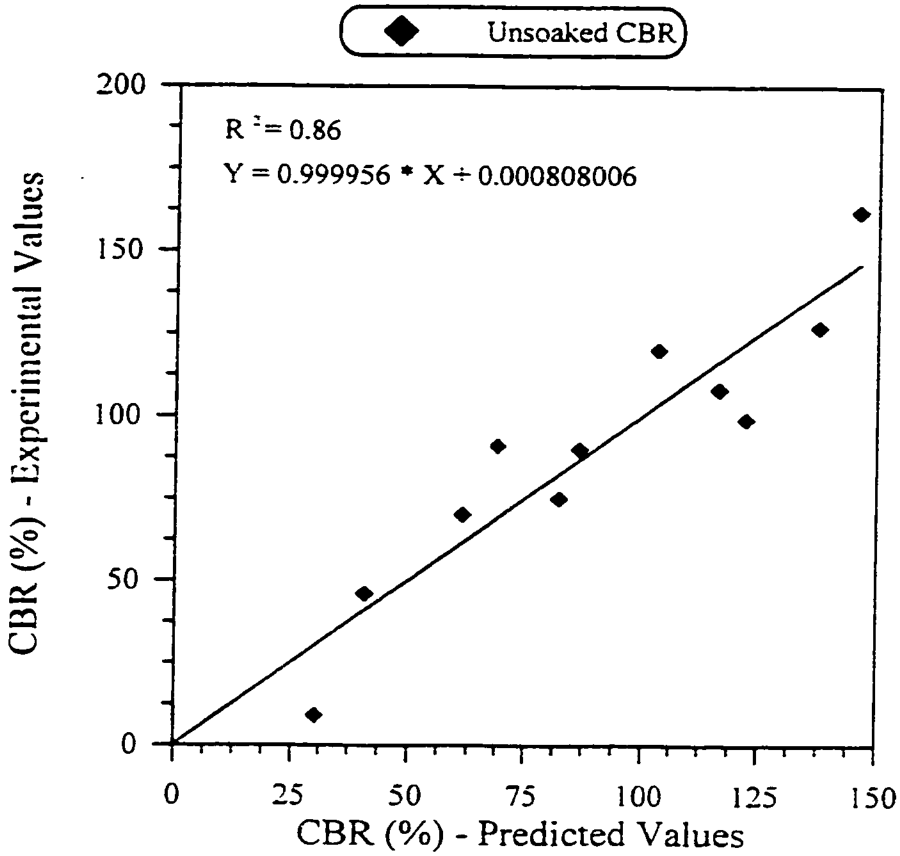


Figure 4.109: The relation between the experimental and the predicted Unsoaked CBR values for the large mold

Chapter 5

Summary, Conclusions and Recommendations

5.1 Summary

As a result of the scarcity of good construction geomaterials, calcareous sediments are extensively used in the Eastern Province of Saudi Arabia. This extensive exploitation of calcareous sediments for constructional purposes, without realization of their upnormal behavior under sever climatic and loading conditions, led to many post constructional problems. Despite the past research efforts, the complicated engineering properties of calcareous soils and the effect of different factors on them are still not properly investigated. In addition, testing of these soils is usually performed without realization of their complicated characteristics, which may lead to inaccurate results. Hence, the possibility of developing new testing techniques or, at least, modifying the existing ones, needs to be investigated. In addition, there is a lack of published information about the adequacy of the standard oversize correction methods for use for marl soils and their effects on the produced strength.

Determination of the effect of gradation and the commonly used testing procedures on the assessment of the load carrying capacity of calcareous sediments was the primary objectives of this study. Two marl samples were collected from two borrow areas in eastern Saudi Arabia. The basic characteristics of the marl samples were obtained following the ASTM testing procedures. Small and large size testing setups were used to study the compactibility and the load carrying capacity of the selected marls. using different gradations.

Based on the results obtained from this experimental program, it was found that the test type and the preparation method have significant effects on the results. In addition, the gradation and the maximum particle size included in the sample were found to have remarkable effects on the strength, especially for some test types. Furthermore, it was found that the preparation method is highly affecting the obtained strength values.

5.2 Conclusions

Based on the results revealed from the experimental research program, the following conclusions can be drawn:

1. The assessment of the load carrying capacity of marl soils using one testing procedure may give misleading results especially when the used testing methodology requires some modifications on the soil gradation. Hence,

different testing procedures must be performed in order to have better engineering judgment.

2. The maximum particle size, which is included in the specimens, has a great significance on the strength, especially when using conventional testing procedures, such as CBR and UCS tests.
3. The strength of the tested marls is extremely sensitive to the molding moisture content. There was remarkable loss of the bearing capacity as a result of increasing the moisture content, especially for samples with plastic fines.
4. The maximum dry density and the optimum moisture content values were found to be independent of soil gradation.
5. High maximum dry density value is not a sufficient condition for a practical strength for marl soils.
6. The mold confinement was found to have a significant effect on the CBR values. More than 100% increase can occur on the CBR values as a result of mold confinement.
7. The AASHTO 2nd equation, for oversize correction, was found to give the best prediction for the maximum dry density of the entire material while AASHTO 1st equation failed to predict the maximum dry density of the total material.
8. The ASTM equation for oversize correction gave higher predicted maximum dry density values for the entire material compared to the entire material compacted in the large mold. Hence contractors may face some difficulties to

achieve the required density if the specifications were given according to the ASTM equation without realizing this phenomenon.

9. Scalp and replace method was found to be an adequate oversize correction method. However it gave slightly lower maximum dry density and optimum moisture content values compared to those obtained for the entire material.
10. Material prepared using scalp and replace method resulted in a higher CBR values compared to those obtained using the elimination method, which excludes the oversize particles without replacement.
11. The CIV values were found to have a strong correlation with the CBR values and a moderate correlation with the UCS values. However the CBR-UCS correlation was found to be weak for the tested specimens.

5.3 Recommendations for Further Study

1. There is a need to develop a large size setup that is fully automated in order to test soils, which commonly include oversize particles (particles larger that ¾ in.). The new setup that was used throughout this program can be the base of the intended setup.
2. More marl soils must be tested to increase the reliability of the obtained correlations. In addition, the use of marl samples with different characteristics will enrich any further study.

3. The effect of soil plasticity and grain crushing on the load carrying capacity of marls need to be investigated further. This can be studied using marl samples with different plasticity and aggregate stiffness.
4. Comparison program between laboratory and field-testing should be executed whereby relationships between results obtained using small molds, large molds and field data are compared. Specific recommendations can then be stated after such study.
5. The contribution of sample height and confinement on the CBR and CIV should be studied. Such study may initiate a correction method, which can be applied for the CBR and the CIV tests.

Nomenclature

C	Fraction by weight (dry basis) of the portion of the total soil passing either No. 4 or ¾ in. (19 mm) sieves, expressed as decimal.
CBR	California Bearing Ratio
CIV	Clegg Impact Value
D	Maximum dry density of the total soil (pcf)
D_f	Maximum dry density of the fine material (pcf)
DCP	The dynamic cone penetration test result value
DPA	The Dynamic Probing (type A) test result value
e	The void ratio of the total material
e_o	The initial void ratio
F	The fraction of oversize particles by weight
F_{opt}	Optimum water content factor
G_g	The specific gravity of gravel
G_m	Bulk specific gravity of the gravel
G_s	The specific gravity of soil binder
I_c	Density interference coefficient
O	Fraction by weight (dry basis) of the portion of the oversize particles in the total soil expressed as decimal.
P	The ratio between the weight of gravel to the weight of total material
P_c	Percent rock (material retained on the No. 4 or the ¾ in. (19 mm) sieve by weight (decimal)
P_F	Percent finer fraction by weight
P_G	Percent gravel
R^2	Coefficient of determination
r_a	Correction factor in AASHTO equation to account for interference of large aggregates
r_u	Correction factor in USBR equation to account for interference of large aggregates
S	The Vane Shear test result value
SPT	The Standard Penetration test result value
UCS	Unconfined compressive strength
W_{fopt}	Optimum water content for finer material
w_{opt}	Optimum water content
W_t	Calculated maximum dry density.
W_o	Density of the oversize material as given by its ASTM bulk specific gravity multiplied by the density of water.
W_c	Measured maximum dry density of the portion of the total soil passing either No. 4 or ¾ in. (19 mm) sieve.
γ_{adj}	The adjusted total maximum dry density (*pcf*)

γ_{fmax}	Maximum dry unit weight of the finer fraction
γ_w	The density of water.
γ_l	The laboratory maximum dry density of the soil matrix (*pcf*)
ϕ	Angle of internal friction
μ	The overall mean effect
τ_i	The effect of the *i*-th marl type
β_j	The effect of the *j*-th parameter (gradation, preparation method…etc)
ν_k	The effect of the *k*-th moisture content value
ϵ_{ijk}	The random computed error
σ'	The effective stress
σ	The total stress
u.	The pore water pressure

References

AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II, July 1982, 13^{th} Edition, Published by *AASHTO*, Washington.

AASHTO Guide for Design of Pavement Structures, 1986, Published by *AASHTO*, Washington.

Ahmed, H. R. (1995), "Characterization and Stabilization of Eastern Saudi Marls, *M.S. Thesis, Department of Civil Engineering, KFUPM*, Dhahran, Saudi Arabia.

Aiban, S.A. (1995), "Strength and compressibility of Abqaiq marl, Saudi Arabia", *Engineering Geology*, Vol. 39, pp. 203-215.

Aiban, S.A. and Al-Abdul Wahhab H, "Utilization of Steel Slag Aggregate in Road Construction", *technical report submitted to Heckett Multi Services-Saudi Arabia Ltd*, March 1997.

Aiban, S.A., Al-Amoudi, O.S.B., Al-Abdul Wahhab, H.I., and Ahmed, H.R. (1999), "Characteristics and chemical stabilization of eastern Saudi calcareous sediments", *Conference on Engineering for Calcareous Sediments*. Balkema, Rotterdam, pp. 101-111.

Akili, W, (1980), "Some properties of remolded carbonate soils". *Proc. 10^{th} Int. Conf. Soil Mechanics and Foundation Engineering*, Stockholm, Vol. 4, pp. 537-542.

Al-Khafaji, A.W. and Andersland, O.B. ((1992), "Geotechnical Engineering and Soil Testing", *Saunders College Publishing*.

Alsanad, H and Albader, B. (1990), "Laboratory Study on Leaching of Calcareous Soil from Kuwait", *Journal of Geotechnical Engineering*, Vol. 116, No. 12, pp. 1797-1809.

"American Heritage Dictionary", 1991, 2nd edition, *Houghton Mifflin Company*

"Annual Book of ASTM Standards", 1991, Section 4, Vol. 04.08 Soil and Rock.

"Annual Book of ASTM Standards", 2000, Section 4, Vol. 04.08 Soil and Rock.

Asi, I.M., Al-Abdul Wahhab, H.I. and Al-Amoudi, O.S.B. (1992), "An Investigation on Clegg Impact Hammer in Saudi Arabia", *Proceedings. The Jordanian Conference on Civil Engineering*, Amman, Vol. 1, pp. 55-56.

Black, W.P.M. (1961), "The Calculation of Laboratory and In-Situ Values of California Bearing Ratio From Bearing Capacity Data".*Geotechnique*, Vol. 11, No. 1, pp. 14-21.

Carter, J. P, Airey, D. W and Fahey, M. (1999), "A review of Laboratory Testing of Calcareous Soils", *Proceedings of the 2nd International Conference on Engineering for Calcareous Sediments*,Vol. 2 February, Manama, Bahrain, pp. 401-431.

Cook, A. W. (1999), "The Soil to Rock Boundary for Weak Calcareous Rocks", *Proceedings of the 2nd International Conference on Engineering for Calcareous Sediments*, Vol. 2, Manama, Bahrain, pp. 131-137.

Datta, M., Gulhati, S.K. and Rao, G.V. (1982), "Engineering Behavior of Carbonate Soils of India and Some Observations on the Classification of Such Soils", *ASTM STP 777*, American Society for Testing and Materials, Philadelphia, pp. 113-140.

Day, R. W. (1989), "Relative Compaction of Fill having Oversize Particles", *Journal of Geotechnical Engineering*, Vol. 115, No. 10 October, pp. 1487-1491.

Donaghe, R.T. and Torrey, V.H. (1994), "A compaction test method for soil-rock mixtures in which equipment size effects are minimized," *Geotechnical Testing Journal*, Vol. 17, No. 3, pp. 363-370.

Donaghe, R.T. and Torrey, V.H. (1994), "Proposed New Standard Test Method for Laboratory Compaction Testing of Soil-Rock Mixtures Using Standard Effort," *Geotechnical Testing Journal*, Vol. 17, No. 3, pp. 387-392.

Fookes, P.G. and Hingginbottom, I.E, 1975. The classification and description of near-shore carbonate sediments for engineering purposes. *Geotechnique*, 2: 406-411.

Fragaszy, R. J, Su, W and Siddigi, F. H. (1990), "Effects of Oversize Particles on the Density of Clean Granular Soils", *Geotechnical Testing Journal*, Vol. 13, No. 2, pp. 106-114.

Garga, V.K, Madureira, C.J. (1985), "Compaction Characteristics of River Terrace Gravel", *Journal of Geotechnical Engineering*, Vol. 111, No. 8, pp 987-1007.

Hardin, B. O. (1985), "Crushing of Soil Particles", *Journal of Geotechnical Engineering*, Vol. 111, pp. 1177-1193.

Head, K.H.(1982), "Manual of soil laboratory testing," 1st edition, Published by *Pentech Press Limited*, Estover Road, Plymouth, Devon.

Horta, J.C, (1988). "Characterization of calcrete and gypcrete as pavement materials". *Proc. Int. Conf. Road and Road Transport Problems*, ICORT-88, New Delhi, India, pp. 107-123.

Houston, S.L. and Walsh, K.D. (1993), "Comparison of rock correction methods for compaction of clayey soils," *Journal of Geotechnical Engineering*, Vol. 119, No. 4, pp. 763-778.

Hsu, T.S. and Saxena, S.K. (1991), "A general formula for determining density of compacted soils with oversize particles", *Soils and Foundations*, Vol. 31, No. 3, *Japanese Society of Soil Mechanics and Foundation Engineering*, pp. 91-96.

Ismail, N. F and Ahmed, H, N. (1990), "Bearing Capacity of Footings on Calcareous Sands", *Soils and Foundations*, Vol. 30, No. 3, 81-90, September, *Japanese Society of Soil Mechanics and Foundation Engineering*, pp. 81-90.

Johnson, D.H. (1987). "Gulf coastal region and its hinterland, General geology". In S.S. Al-Sayyari and J.G.Zotl (Editors), Quaternary Period in Saudi Arabia. *Spinger, Berlin*, pp. 45-50.

Johnson, S, Audibert, J. M. A and Stevens, R. F. (1999), "PCPT-based Method for Selecting upgraded Pile Foundation Design Parameters", *Proceedings of the 2nd International Conference on Engineering for Calcareous Sediments*, Vol. 2, Manama, Bahrain, pp. 473-480.

Khan, Z., Al-Amoudi, O. S. B, Asi, I. M. and Al-Abdul Wahhab, H. I. (1995), "Field and Laboratory Assessment of the Clegg Hammer-CBR Correlation", *Proceedings, 4th Saudi Engineering Conference*, Jeddah, Saudi Arabia, Vol. II, pp. 339-345.

Livneh, M. (1989), "Validation of Correlations Between a Number of Penetration Test and Insitu California Bearing Ratio Tests", *Transportation Research Board*, No. 1219, National Research Council, pp. 56-67.

McLeod, N. W. (1970), "Suggested Method for Correcting Maximum Density and Optimum Moisture Content of Compacted Soils for Oversize Particles", *ASTM Special Technical Publication 479*, Special Procedures for Testing Soil and Rock for Engineering Purposes, 5th edition, pp. 111-113.

Mikhail, M. Y., Seeds, S. B., Alavi, S. H., and Ott. W. C. (1999), "Evaluation of Laboratory and Backcalculated Resilient Moduli from WesTrack Experiment", *Transportation Research Board*, No. 1687, National Research Council, pp. 55-65.

Nataatmadja, A, "CBR Test of a Lateritic Material Aspects and Implications", *Proceedings of Civil Engineering Department*, University of Rookee, 1988, pp.176-185.

Osman, S. A. (1995), "Searching for a Fundamental Characteristics of Unbound Materials Using CBR Testing", *MSc. Thesis, IHE.* Delft, Netherlands.

Proctor, R.R. (1933), "Design and construction of rolled-earth dam," Article II, "Description of Field and Laboratory Methods," *Engineering News Record*, Vol. III, No. 9, pp. 286-289.

Qahwash, A.A. (1989), "Geotechnical Properties of Fine-Grained Calcareous Sediments for Engineering Purposes", *Engineering Geology*, Vol. 26, pp. 161-169.

Ramadan, E. H. (1999), "Prediction of Collapse Potential of Soils in Western Desert of Egypt", *Proceedings of the 2nd International Conference on Engineering for Calcareous Sediments*, Vol. 1, Manama, Bahrain, pp. 139-148.

Roger, J, (1985). Explanatory notes to the industrial mineral resources map of Ad Dammam, Kingdom of Saudi Arabia. *Ministry of Petroleum and Mineral Resources*, Geoscience Map GM-111.

Southgate, H.F. and Mahboub, K.C. (1994), "Proposed uniform scale for stiffness of unbound pavement materials for pavement design," *Journal of Transportation Engineering*, Vol. 120, No. 6, pp. 940-952.

Terrel, R.L., Epps, J.A., Barenberg, E.J., Mitchell, J.K. and Thompson, M.R. (1984), Soil Stabilization in Pavement Structures A Users Manual, Vol. 2, Mixture Designation, *Prepared for Federal Highway Administration Department of Transportation*, Washington.

Torrey, V.H. and Donaghe, R.R. (1994), "Compaction control of earth-rock mixtures: a new approach", *Geotechnical Testing Journal*, Vol. 17, No. 3, pp. 371-386.

Wagner, A.A. (1970), "Suggested Method of Test for Moisture-Density Relations of Gravelly Soils", *ASTM Special Technical Publication 479*, Special Procedures for Testing Soil and Rock for Engineering Purposes, 5th edition, pp. 108-110.

Walsh, K.D., Houston, S.L. and Wilson, G.P. (1994), "Rock correction issues in compaction specifications for high gravel content soil," *Transportation Research Record,* 1462, pp. 3-9.

Wilson, J.L. (1975), "Carbonate Facies in Geologic History", *Springer- Verlag publishing.*

Winter, M. G, Holmgeirsdttir, Th and Suhardi. (1998), "The effect of large particles on acceptability for earthworks compaction", *Quarterly Journal of Engineering Geology*, 31, pp. 247-268.

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