INFLUENCE OF GRADING ON THE STRENGTH OF WATER-BOUND MACADAM

A Project Report
by
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Dedicated to

My

"BELOVED PARENTS"

ABSTRACT

The research work was undertaken to study the influence of grading on the strength of water-bound macadam (WBM). The study includes the selection of a suitable aggregate grading for water-bound macadam construction from the considerations of strength and stability. Attempts are also made to study the properties of brick-aggregate used for WBM construction.

The whole research work was carried out in three stages. In the first stage, compaction type was selected from degradation of brick aggregate due to different compactive efforts (i.e. 2.5 kg, 4.5 kg and vibrating hammer). Use of screenings was selected in the second stage from bearing capacity of the mix for two conditions — one by mixing screenings with coarse aggregate and another by providing screenings on the top surface of coarse aggregate. Finally, aggregate grading was selected from strength and stability of the aggregate mix.

Four gradations were initially selected from the recommended specified gradations. Modified Bearing Ratio (MBR) tests were then performed after compacting by a vibrating hammer at optimum moisture contents as CBR test is not applicable to most of the gradings used.

The study reveals that "Picked Jhama" brick-aggregate-sand-soil mixes are good for use in WBM construction, from the standpoint of strength and stability. WBM mixes with picked jhama brick aggregates give satisfactory results when they are constructed using dense grading compacting by vibrating hammer at optimum moisture content.

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DECLARATION

I hereby declare that the project work presented herewith has been performed by me and that this work has not been submitted for any other degree previously.

Gmkarim

(GAZI MD. REZAUL KARIM)

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ABBREVIATIONS

AASHO - American Association for State Highway Officials
AASHTO - American Association for State Highway and Transportation Officials
ACT - Aggregate Crushing Test
ACV - Aggregate Crushing Value
AIV - Aggregate Impact Value
ANSI - American National Standard Institute
ASTM - American Society for Testing and Materials
B.C. - Before Christ
BRRL - Bangladesh Road Research Laboratory
BS - British Standard
BSI - British Standards Institution
CBR - California Bearing Ratio
CDH - California Division of Highways
DOE - Department of Environment
Fig. - Figure
IRC - Indian Roads Congress
IS - Indian Standard
LAA - Los Angeles Abrasion
Max. - Maximum
MBR - Modified Bearing Ratio
NO. - Number
OMC - Optimum Moisture Content
PI - Plasticity Index
PSI - Pavement Servicibility Index
TFV - Ten Percent Fines Value
TRRL - Transport and Road Research Laboratory
U.K. - United Kingdom
U.S. - United States
USA - United States of America
WBM - Water-Bound Macadam

UNITS AND SYMBOLS

UNITS:

cm - Centimetre
gm - Gram
in. - Inch
in/min. - Inch per minute
kg - Kilogram
kg/m^3 - Kilogram per cubic metre
kN - Kilonewton
kW - Kilowatt
lbs/cft- Pounds per cubic foot
m - Metre
mm - Millimetre
mm/min. - Millimetre per minute
mm^2 - Square millimetre
pci - Pound per cubic inch
psi - Pound per square inch
sq.in. - Square inch

SYMBOLS:

C - Cohesion
C_u - Coefficient of uniformity
°C - Degree centigrade
D_2 - Displacement
D_{50} - Diameter of particles finer than 50%
K - Modulus of subgrade reaction
P - Pressure
P_h - Horizontal pressure
P_v - Vertical pressure
R - Stabilometer resistance
S - Shearing resistance
Ø - Angle of internal friction
℄ - Centre-line
Δ - Deflection
" - Inch
μ - Micron
% - Percent
σ - Vertical stress

CHAPTER–ONE

INTRODUCTION

1.1 GENERAL

The term "macadam" originally referred to a road surface or base in which crushed or "broken" stone was mechanically "keyed" or locked by rolling and cemented together by the application of stone screenings and water. Later, when "bituminous macadam" roads were built by using a bituminous material as the binder, the original type of road came to be known as a "water-bound macadam".

Macadam roads originally developed in France and England and are named after John Louden McAdam, a famous Scottish road builder and engineer. Modern broken stone roads were first introduced in France by Tresaguet about 1765 and by Telford in England about 1805. Both these engineers used large pieces of broken stones in the lower course and placed a layer of finer crushed stones as a wearing surface. Telford advocated the use of large flat pieces of stones, which were hand placed to form the base. Telford bases employing flat pieces of ledge stone 7.5 cm to 20 cm (3 in. to 8 in.) in thickness and from 10 cm to 40 cm (4 in. to 16 in.) in width and length and placed by hand are still sometimes employed in subgrades having low supporting power. These bases are however expensive particularly when manual labour is not cheap. McAdam however, insisted upon the use of smaller stones, about 40 mm ($1^1/_2$ in.) maximum size, for the entire thickness of the pavement. The invention of the stone crusher in 1858 by Eli Blake and also the invention of steam roller in France in about 1860 greatly advanced macadam construction. However, horse-drawn rollers were employed for many more years. The technique of macadam construction improved rapidly over the years, and by the end of the nineteenth century many miles of this type of surface had been built and were in service as rural roads and city streets in the United States. With the advent of the motor vehicle, the basic macadam or broken-stone surface become less suited to the demands of traffic, and this type of pavement assumed less importance in highway construction. Roads of the original macadam type were (and are) basically unsuited for use as wearing surfaces carrying large numbers of rapidly moving pneumatic tyred vehicles as they became quite rough and dusty under this type of traffic.

The combination of tightly keyed coarse aggregate with the bond produced by stone chips and dust creates a base course equally as good as other untreated bases. Thus, in areas where cheap aggregates were not available in stream beds or other gravel deposits so that quarrying and crushing of ledge rock was required, macadam bases compacted favourably. Today, where crushing of ledge rock is required to produce aggregate, macadam bases may be less costly than "graded" granular ones, since crushing to produce the intermediate sizes is expensive.

After World War II, macadam bases fell into disfavour, and today they have been dropped from the specifications of many highway agencies. In certain instances, and particularly on low-traffic roads, some developing countries like India and Bangladesh and some agencies have gone back to them since they are equally as good and less costly than bitumen-or cement-treated bases. In India, the macadam construction is still very popular and is in extensive use both in rigid as well as in flexible pavements. In case of rigid pavements a granular layer serves as a levelling course or as a working platform before laying the concrete slab. In case of flexible pavements the granular layers are used to serve as the main load dispersion layers and are generally a two component system consisting of soling stone (which is currently tending to be replaced by either a mechanically stabilized soil layer or lime-soil-flyash mixtures) and water-bound macadam layer (mainly as base courses). Almost all the rural and minor district roads in India have macadam construction as base and surfacing layers. In Bangladesh, almost all flexible pavement highways, macadam construction is used as a base or sub-base.

1.2 WATER-BOUND MACADAM (WBM)

Macadam construction can be classified into the following four categories:

i) Dry bound macadam

ii) Cement bound macadam

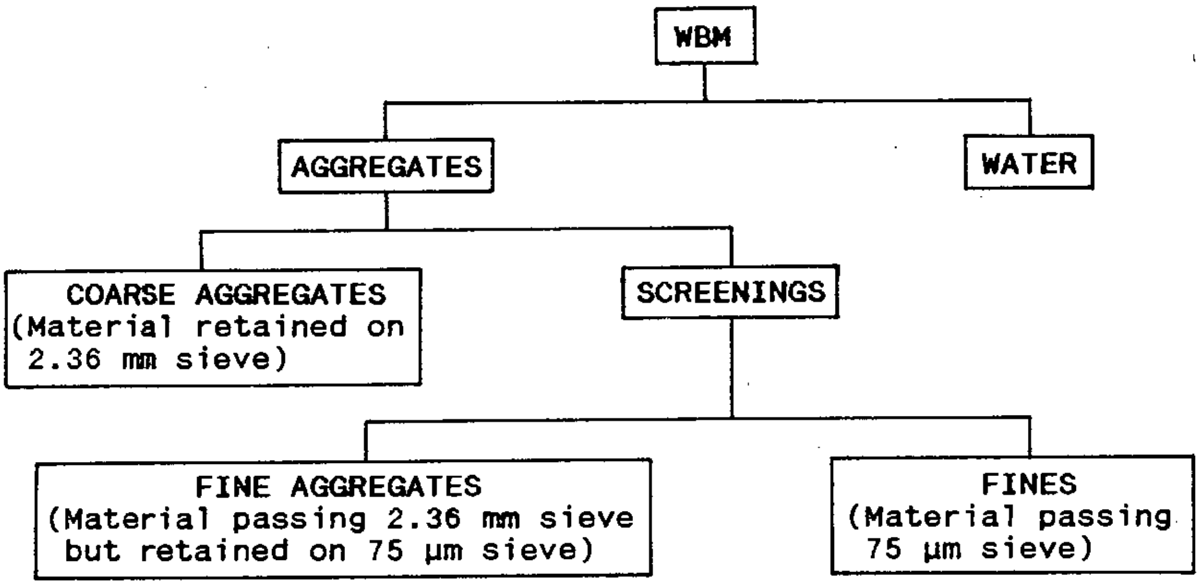
iii) Water-bound macadam (WBM)

iv) Bituminous macadam

Water-bound macadam or wet-mix macadam is a layer composed of broken stone (or crushed gravel or slag) fragments that are bound together by stone dust (screenings) and water is applied during construction, in connection with consolidation of the layer by a heavy roller or a vibratory compactor.

Water-bound macadam is constructed in thickness ranging from about 8 cm to 30 cm (3 in. to 12 in.) depending upon the purpose for which they are intended. Their principal use is as bases for flexible pavements, although their use as wearing course is permitted in some cases. Invariably water-bound macadam layers are covered with some sort of bituminous wearing surfacing soon after their construction. One course construction is permitted upto 12.5 cm to 15 cm (5 in. to 6 in.) compacted thickness, 22.5 cm (9 in.) by two course construction and greater thickness in three lifts. Each compacted layer is generally expected to compress from 75 to 80 percent of loose thickness.

Ingredients of water-bound macadam can be shown by diagram as below:



In water-bound macadam mixes, coarse aggregates give stability through interlocking of the aggregate particles which contribute to the frictional resistance to displacement. They provide the framework to distribute or "spread" the stresses created by wheel loads acting on the wearing surface so that the stresses transmitted to the subgrade will not be sufficiently great to result in excessive deformation or displacement of that foundation layer.

Through interlocking of the particles the fine aggregates give some additional stability to the mix and at the same time fine aggregates reduce the voids in the coarse aggregates.

The function of fines in mixes is to fill the excess voids in mineral aggregates. While water acts as a lubricant during compaction and serves to keep the framework of aggregates in position by developing cohesion.

1.2.1 Merits and Demerits of WBM

There are some merits as well as demerits of WBM construction used for base or sub-bases of flexible pavements which are given below:

Merits:

i) Economically and technically sound in areas of the world where labour is less costly and labour-intensive method of construction are desirable.

ii) Moderate initial cost.

iii) Less skidding tendency to vehicles.

iv) Skilled labour is not required.

v) Easy tracts to vehicles in good condition.

Demerits:

i) Noisy with steel-tyred wheels if it is used as surface course.

ii) Muddy during rainy season and dusty in dry weather.

iii) Costly to maintain under heavy traffic.

iv) Pot holes are easily formed.

v) Unsuitable for modern highspeed or heavy steel-tyred traffic as they remove the binder and reduce the surface to a pile of loose rocks.

1.2.2 Construction of WBM in Bangladesh

Bangladesh is an under developed populous country where transportation system is not fully developed. Except some urban highways and a few rural roads most of the roads of this country are earth roads. The development of transportation system is primarily required for development of a country by industrialization. Because the raw materials required by an industry is carried by transportation system and mostly by highways as rail roads are not well developed in this country. Again the improvement of highway transport system in Bangladesh is also to mean the improvement of the earth roads. Since Bangladesh is a poor country and as no binder is required for WBM construction; so it is appropriate to use WBM construction for economic road construction in this country. It is recommended that WBM is suitable for low volume traffic (1). So low volume rural roads where vehicles less than 1500 tons per day whose weights are 3 to 5 tons can be improved by using WBM. WBM may also be used for bases and sub-bases for flexible pavement construction which is also recommended by the road construction authority of Bangladesh.

1.2.3 Use of Brick Aggregates for Road Construction in Bangladesh

In recent years the use of bricks for road construction has increased unexpectedly in many countries of the world. Specially in developing countries of south-east Asia, crushed bricks are being used in many civil engineering works including roads. High crushing strength, low absorptive natural aggregate such as gravels and boulders are not available in Bangladesh except in small quantities in Northern Sylhet and Dinajpur areas. It is expensive to carry these natural aggregates from their sources to job sites if the haulage length is high. Again with the increased and continued consumption, those naturally occurring aggregate sources are being depleted. Due to insurmountable deficiency of conventional natural aggregates in Bangladesh, crushed bricks are used for the construction of base and sub-base course of flexible pavements. Good quality brick aggregate in unbound condition has found to be satisfactory, from strength consideration, provided they are compacted in a dense grading applying appropriate compacting energy (2).

Some research works were carried on the use of brick aggregates for road construction in Bangladesh. Hoque (3) investigated behaviour of brick-aggregate asphaltic concrete for road pavement and suggested that "Picked Jhama Bricks" are suitable for the surface courses of asphaltic concrete pavements. Zakaria (4) also carried out investigations on behaviour of brick aggregates in base and sub-base courses and suggested that gas burnt picked jhama bricks and jhama bricks are the most suitable type of bricks for base and sub-base construction in Bangladesh. Besides, Sobhan (5) showed that "Picked Jhama" bricks are also suitable for the bituminous macadam for road construction in Bangladesh.

Since Bangladesh is a poor country where labour is cheap and due to shortage of natural aggregates the roads in this country are to be constructed with brick aggregates. Good bases constructed with dense graded high quality brick aggregates is one of the economic solutions of road construction in this country (6).

1.3 IMPORTANCE OF GRADING IN MACADAM ROAD CONSTRUCTION

Grading of aggregates means grain size distribution of various sizes of aggregate or batching by sieving. Grading plays an important role in water-bound macadam. Grading of aggregate is necessary for economic construction to obtain a densest mixture of aggregates and stability of the construction.

Stability of macadam construction depends upon particle-size-distribution i.e. gradation, particle shape, relative density, internal friction and cohesion. A granular material designed for maximum stability should posses high internal friction to resist deformation under load. Internal friction and subsequent shearing resistance depend to a large extent upon density, particle shape and gradation. Of these factors, gradation of aggregates, particularly the proportion of fines to the coarse fraction, is considered to be the most important (4). For maximum stability macadam construction should have fines just sufficient to fill all the voids between aggregates. The density will decrease and the macadam is practically impervious and frost susceptible. Thus the stability of macadam construction is dependent upon gradation of aggregates. Since aggregate gradation played an important role in macadam construction; so it is necessary to determine a suitable gradation of aggregates which gives higher strength.

1.4 OBJECTIVES OF THE STUDY

The objectives of this study are as follows:

i) To determine the physical properties of brick aggregates used in the WBM mixes.

ii) To select the appropriate compaction method for lower degradation of brick aggregates.

iii) To find the effect on CBR value due to placing of screenings.

iv) To investigate the effect of gradation on the strength of WBM mixes in terms of MBR/CBR values.

v) To select a gradation which is suitable for WBM construction in Bangladesh.

CHAPTER-TWO
LITERATURE REVIEW

2.1 GENERAL

Limited literatures are available on the use of brick aggregates in WBM (base and sub-base) construction. Stone aggregates and stone-sand or stone-soil mixtures have been used for the construction of WBM (base or sub-base) courses for high type pavements and surface courses for low type pavements. Literatures, specifications, recommendations are available regarding the characteristics of natural aggregates and soil-aggregate mixtures in WBM (base and sub-base) courses. Sometimes the aggregates are stabilized by some admixtures to have a firm base or sub-base to provide the support for a relatively thin wearing surface. Literatures are also available for such type of construction. The present research work has been undertaken to determine the characteristics of brick aggregate-soil mixture in WBM (base and sub-base) constructions. Limited literature available for unstabilized stone aggregate and/or gravel-soil mixtures are discussed in the following articles.

2.2 GRADING SPECIFICATIONS FOR WBM

WBM is normally used for base or sub-base construction of flexible pavement. The WBM used in this investigation is a mixture of aggregate and non-plastic soil. The stability, and density are developed in the WBM course by using proper gradation, appropriate amount of fine material——silt and clay, moisture content and of course by proper compaction. When aggregates are available near road side, soil from the road bed can be mixed with them and compacted. In most cases, this type of construction is economical. The research work has been concerned with the evaluation of this type of mixtures. Almost in all developing countries and also in some developed countries this type of construction is in use for rural roads. In the following articles the recommendations and practices regarding this WBM constructions are briefly discussed.

2.2.1 Practices in U.S.A. and AASHTO Recommendations

In some states of USA low type roads are constructed with WBM mixtures as the surface course. In intermediate type of roads and in some high type rural roads WBM mixtures are used for base and sub-base constructions. The American Society for Testing and Materials (ASTM) and the American Association for State Highway and Transportation Officials (AASHTO) have first come to specify materials for this type of constructions. The materials for WBM base courses are specified by ASTM (7) under designation ASTM D694-71 and WBM surface course is specified by AASHTO (8) under designation AASHTO M77-64. The gradings are shown in Table 2.1 and the requirements are as follows.

Table 2.1 Grading Requirements for Aggregates for use in WBM Base Courses (ASTM D694-71) and Surface Course (AASHTO M77-64)

	Size No.	Size Range	Sieve Designation	Percent by weight passing the sieve
Coarse Aggregate	1	3 1/2" to 1 1/2"	4" (100 mm)	100
			3 1/2" (90 mm)	90-100
			2 1/2" (63 mm)	25-60
			1 1/2" (37.5 mm)	0-15
			3/4" (19 mm)	0-5
	2	2 1/2" to 1 1/2"	3" (75 mm)	100
			2 1/2" (63 mm)	90-100
			2" (50 mm)	35-70
			1 1/2" (37.5 mm)	0-15
			3/4" (19 mm)	0-5
	3	2" to 1"	2 1/2" (63 mm)	100
			2" (50 mm)	90-100
			1 1/2" (37.5 mm)	35-70
			1" (25 mm)	0-15
			1/2" (12.5mm)	0-5
Screenings		NO.4 to 0	3/8" (9.5 mm)	100
			NO.4 (4.75 mm)	85-100
			NO.100 (0.150 mm)	10-30

General Requirements:

a) Coarse aggregate shall consist of hard, durable particles or fragments of stone, gravel or slag.

b) Coarse aggregate shall have percent wear less than 50% for base courses and less than 40% for surface course.

c) Coarse aggregate shall have a unit weight greater than 65 lbs/cft (1041 kg/m^3) and percent of weight loss in Na_2SO_4 soundness test (five cycles) shall not be greater than 20% for base courses and 12% for surface course.

d) Screenings (NO.4 to 0 size) shall have a liquid limit not greater than 30 and a plasticity index not greater than 6 ; shall be well graded, free from dirt and of suitable binding quality.

2.2.2 Practices in U.K. and BSI Recommendations

The Department of Environment (DOE) Specification for Road and Bridge Works (9), U.K. suggests two unbound granular sub-base materials to conform to the gradings specified in Table 2.2. Type-1 aggregates comprise crushed rock, crushed concrete, crushed slag or well-burnt non-plastic shales. These materials will remain stable over a much wider range of moisture contents than Type-2 aggregates which include well graded natural sands, gravels, and rock or slag fines, and are therefore to be preferred in Bangladesh, where site conditions are likely to be wet during construction. For Type-2 sub-base, a minimum CBR of 30 percent on aggregate is required.

Table 2.2 Grading Requirements for Unbound Granular Sub-base Materials

B.S. sieve size	Percent by weight passing	
	Type-1	Type-2
75 mm	100	100
37.5 mm	85-100	85-100
10 mm	40-70	45-100
5 mm	25-45	25-85
600 μm	8-22	8-45
75 μm	0-10	0-10

Additional Requirements:

i) Both type of materials should be well-graded within the limits given.

ii) The material passing the 425 μm sieve should be non-plastic for Type-1 material and have a plasticity index less than 6 for Type-2.

iii) The moisture content as laid and compacted should, for Type-2 material, be within the range + 1 to -2 percent of the optimum moisture content determined in accordance with the vibrating hammer compaction test of B.S. 1377. Type-1 is laid and compacted at the moisture content supplied.

Table 2.3 Grading Requirements for Wet-mix Macadam Road Bases

B.S. sieve size	Percent by weight passing
50 mm	100
37.5 mm	95-100
20 mm	60-80
10 mm	40-60
5 mm	25-40
2.36 mm	15-30
600 µm	8-22
75 µm	0-8

The DOE Specification for Road and Bridge Works (9) suggests wet-mix macadam roadbase where crushed rock or slag aggregate is used to conform to the grading specified in Table 2.3.

2.2.3 Practices and Recommendation by Bangladesh Road Research Laboratory

Bangladesh Road Research Laboratory, BRRL (10) recommends to use broken brick and brick/sand mixtures for sub-base construction although gravels may be available in some areas. For sub-base work the materials should conform to one of the grading requirements shown in Table 2.4.

Table 2.4 Grading Requirements for Sub-base Materials

Sieve Designation	Percent by weight passing sieve		
	Grading A	Grading B	Grading C
50 mm	100	-	-
38 mm	85-100	-	-
20 mm	55-95	100	-
10 mm	35-75	70-100	100
5 mm	25-60	45-85	75-100
2.4 mm	15-50	30-70	50-80
0.600 mm	7-35	10-45	30-55
0.300 mm	-	7-30	20-40
0.075 mm	3-15	4-20	10-25

General Requirements:

a) The plasticity index of the sub-base material shall not be greater than 9% and the liquid limit shall not exceed 25%.

b) The aggregate crushing value (ACV) of the sub-base material shall not be greater than 38% and the Ten Percent Fines Value (TFV) shall not be less than 75 kN.

c) The soaked CBR of the subgrade material compacted to 95% of the maximum dry density heavy (or vibrating hammer) shall not be less than 25%.

BRRL (10) also recommends, WBM for construction of bases of flexible pavements where normally crushed stone aggregates, broken brick aggregates or a mixture of both is used. Base materials should conform to one of the gradings A or B shown in Table 2.5 depending on the maximum size of the aggregate present.

Table 2.5 Grading Requirements for Base Materials

Sieve Designation	Percent by weight passing sieve	
	Grading A	Grading B
50 mm	100	-
38 mm	90-100	-
20 mm	60-90	100
10 mm	40-70	80-100
5 mm	30-55	50-80
2.4 mm	20-45	35-65
0.600 mm	10-30	15-40
0.300 mm	7-25	10-30
0.075 mm	5-15	5-15

General Requirements:

a) The plasticity Index of the base material shall not be greater than 5% and the liquid limit shall not exceed 20%.

b) Aggregate crushing value (ACV) and the Ten Percent Fines Value (TFV) of the base material shall be in accordance with Table 2.5A.

c) The soaked CBR of the base material compacted to 98% of the maximum dry density heavy (or vibrating hammer) compaction shall be in accordance with Table 2.5A.

Table 2.5A Strength Requirements for BRRL Base Materials

Base* Type	CBR Not less than	ACV Not more than	Ten percent fines Not less than
I	80%	30%	150 kN
II	50%	35%	100 kN

* To minimise the construction cost the base is divided into two layers --base Type I is the top layer while base Type II is the bottom layer.

2.2.4 Practices in India and IRC Recommendations

Standard specification and code of practice (11) for water-bound macadam roads are covered by IRC:19-1972. This standard was originally published in 1966. Later, it was approved for publication as the finalised standard by the IRC Council in their 79th meeting held at Gandhinagar in the 25th November,1972.

According to IRC specification (11), water-bound macadam shall consist of clean crushed coarse aggregates mechanically interlocked by rolling, and voids thereof filled with screenings and binding material with the assistance of water, laid on a prepared subgrade, sub-base, base or existing pavement as the case may be. Water-bound macadam may be used as a sub-base, base course or surfacing course and in each case, it shall be constructed with the specifications given below.

General Requirements for Coarse Aggregates:

Coarse aggregates shall be either crushed or broken stone, crushed slag, overburnt brick metal or naturally occurring aggregates such as kankar or laterite. The coarse aggregates shall conform to one of the gradings shown in Table 2.6. The aggregates shall conform to the physical requirements set forth in Table 2.7. The use of grading 1 shall be restricted to sub-base courses only.

Crushed or broken stone shall be hard, durable and generally free from flat, elongated, soft and disintegrated particles. It shall also not have excess of dirt or other objectionable matter. Crushed slag shall be manufactured from air-cooled blast furnace slag. It shall be of angular shape, reasonably uniform in quality and density, and generally free from any thin, elongated and soft pieces, dirt or other objectionable matter. Crushed slag shall not weigh less than 1120 kg/m^3 and the percentage of glassy material in it shall not be in excess of 20. Water absorption of slag shall not exceed 10 percent. Brick metal shall be made out of overburnt bricks or brick bats and be free from dust and other foreign matter. Kankar shall be tough having a blue almost opalescent fracture. It shall not contain any clay in the cavities between nodules. Laterite shall be hard, compact, heavy and of dark colour.

Table 2.6 Size and Grading Requirements of Coarse Aggregates for WBM (IRC:19-1972)

Grading No.	Size Range	Sieve Designation	Percent by weight passing the sieve
1	90 mm to 40 mm	100 mm	100
		80 mm	65-85
		63 mm	25-60
		40 mm	0-15
		20 mm	0-5
2	63 mm to 40 mm	80 mm	100
		63 mm	90-100
		50 mm	35-75
		40 mm	0-15
		20 mm	0-5
3	50 mm to 20 mm	63 mm	100
		50 mm	95-100
		40 mm	35-70
		20 mm	0-10
		10 mm	0-5

Table 2.7 Physical Requirements of Coarse Aggregates for WBM

Serial No.	Type of Construction	Test	Test Method	Requirement
1	Sub-base	Los Angeles Abrasion value* or Aggregate Impact Value*	IS:2386 (Part-IV) IS:2386	Max. 60% Max. 50%
2	Base course with bituminous surfacing	a) Los Angeles Abrasion value* or Aggregate Impact Value* b) Flakiness Index***	IS:2386 (Part-IV) IS:2386 (Part-IV) or IS:5640** IS:2386 (Part-I)	Max. 50% Max. 40% Max. 15%
3	Surfacing course	a) Los Angeles Abrasion value* or Aggregate Impact Value* b) Flakiness Index***	IS:2386 (Part-IV) IS:2386 (Part-IV) or IS:5640** IS:2386 (Part-I)	Max. 40% Max. 30% Max. 15%

Notes: * Aggregates may satisfy the requirements of either the Los Angeles test or Aggregate Impact Value Test.

** Aggregates like brick metal, kankar and laterite which get softened in presence of water, should invariably be tested for impact value under wet conditions in accordance with IS:5640.

*** The requirement of Flakiness Index shall be enforced only in the case of crushed/broken stone and crushed slag.

General Requirements for Screenings:

IRC has further specified that screenings to fill voids in the coarse aggregates should generally be of the same material as the coarse aggregates. However, from economic considerations, predominantly non-plastic material such as kankar nodules, moorum or gravel (other than rounded river-borne material), if used for this purpose, should have liquid limit and plasticity index below 20 and 6 respectively and the fraction passing 75 micron sieve should not exceed 10 percent. As far as possible, the screenings should conform to the gradings given in Table 2.8. Screenings of Type A should be used with coarse aggregates of grading 1 in Table 2.6. With coarse aggregates of grading 2, either Type A or Type B screenings may be used; but with coarse aggregates of grading 3, only Type B screenings should be used. The use of

Table 2.8 Grading Requirements of Screenings for WBM

Grading/ Classification	Size of Screenings	Sieve Designation	Percent by weight passing the sieve
A	12.5 mm	12.5 mm	100
		10.0 mm	90-100
		4.75 mm	10-30
		150 micron	0-8
B	10.0 mm	10.0 mm	100
		4.75 mm	85-100
		150 micron	10-30

screenings may be omitted in case of soft aggregates such as brick metal, kankar and laterite.

Requirements for Binding Materials:

Binding material to prevent ravelling of WBM shall consist of a fine grained material possessing PI value of 4-9 when the WBM is to be used as a surfacing course, and upto 6 when the WBM is being adopted as a sub-base/base course with bituminous surfacing. If limestone formations are available nearly, limestone dust or kankar nodules may be usefully employed for this purpose.

Application of binding material may not be necessary where the screenings consist of crushable type material like moorum or gravel. However, for WBM used as a surfacing course, where the PI of crushable type screenings is less than 4, application of a small amount of binding material having PI of 4-9 would be required at the top. The quantity of screenings could be reduced slightly on this account.

Quantities of Material Required for WBM:

According to IRC specification (11), the approximate percent of screenings required for WBM sub-base, base or surface course constructions are given in Table 2.9 (which is converted from Table-4 and Table-5 of IRC specification IRC:19-1972).

Table 2.9 Approximate Percent of Screenings Required for WBM Sub-base, Base and Surface Course (Converted from Table 4 and Table 5 of IRC specification IRC:19-1972)

Type of course	Compacted thickness	Coarse aggregate grading	Type of screenings	Percent of screenings required to the total aggregate mix	
				Stone screenings	Crushable type screenings
Sub-base	100 mm	Grading-1	Type A	22% to 27%	24% to 28%
Base	75 mm	Grading-2	Type A	14% to 18%	24% to 28%
			Type B	22% to 27%	
		Grading-3	Type B	20% to 25%	
Surfacing	75 mm	Grading-2	Type A	12% to 16%	24% to 28%
			Type B	18% to 22%	
		Grading-3	Type B	17% to 21%	

The quantity of Binding Material will depend on the type of screenings and function of WBM. Generally, the quantity required for 75 mm compacted thickness will be 0.06-0.09 $m^3/10\ m^2$ in the case of WBM sub-base/base course and 0.10-0.15 $m^3/10\ m^2$ when the WBM is to function as a surface course. For 100 mm thickness, the quantity needed respectively will be 0.08-0.10 $m^3/10\ m^2$ and 0.12-0.16 $m^3/10\ m^2$.

2.3 METHOD OF EVALUATION

The base and sub-base courses are evaluated in the field and/or in the laboratory by the following methods:

i) Plate Bearing Test - (F*)

ii) Triaxial Compression Test - (L**)

iii) Stabilometer Test - (L)

iv) California Bearing Ratio Test - (F and L)

i) Plate Bearing Test

The Plate Bearing test developed by N.W. McLeod (12) of Canada Transportation Department expanded the results of his extensive field study relating to it to pavement design methods

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* F indicates field test.

** L indicates laboratory test.

based on the theory of elasticity. The test can be used to measure the strength at any elevation in an asphalt pavement structure: surface of the subgrade, top of sub-base, top of the base course, or surface of the finished pavement.

The schematic arrangement of equipment used in performing the plate bearing test is shown in Fig.2.1. It consists of a loading frame which has a hydraulic jack, a proving ring and a reaction beam. The dial gauges which rest on a separate datum frame are used for the measurement of the settlement of the plate. A thin bed of sand is commonly used to insure an even bearing between the plate and the underlying material. Load is applied to the plate by means of a large capacity hydraulic jack equipped with pressure gages.

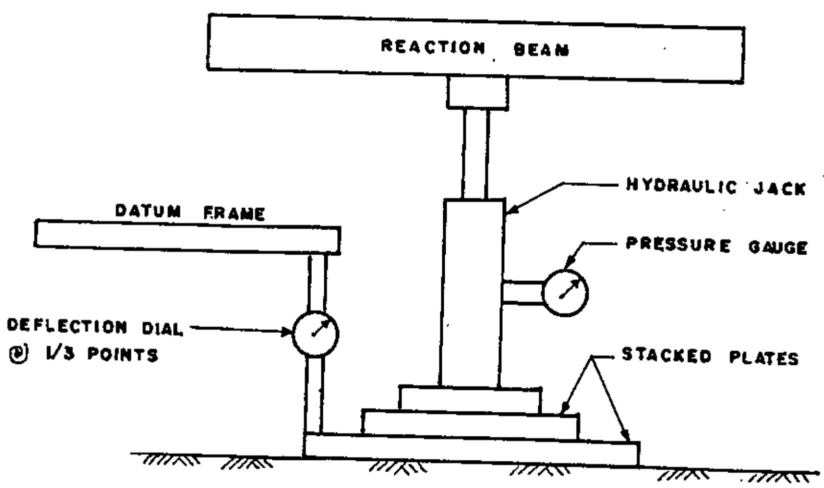

FIG. 2.1 EQUIPMENT USED IN PERFORMING PLATE BEARING TESTS.

Deflection of circular rigid plates are measured deflection dials under vertical loads applied at some standard rate. The modulus of the layer on which the plates are placed is calculated by means of the following equation,

$$K = \frac{P}{\Delta} \quad \text{........................(2.1)}$$

where, K = modulus of subgrade reaction (pci)

P = unit load on the plate (psi)

Δ = deflection of the plate (inches)

Tests are generally made directly on the unconfined base course and the field value is adjusted to the most unfavourable base condition that can be expected (4). This can be accomplished by the additional data obtained by loading the samples in confined conditions.

ii) Triaxial Compression Test

The triaxial compression test was developed to determine the shear stress of a soil sample under lateral pressure. The test is suitable for only fine grained soil or sand where specimens are loaded vertically under constant lateral pressure to have a shear failure. The stability of the soil is determined by the equation,

$$S = C + \sigma \tan\phi \quad \text{..................(2.2)}$$

where, S = shearing resistance developed

C = cohesion

σ = vertical stress applied

ϕ = angle of internal friction

Values of cohesion and internal friction, can be determined from the plots of the test results.

Based on the principles of triaxial test, McDowel (4) developed Texas triaxial cell to evaluate base course materials. The apparatus is suitable for performing a large number of test economically. Shear stress is measured at different normal stresses and the results are plotted to classify different types of materials.

iii) Stabilometer Test

California Division of Highways developed a semi-theoritical method of flexible pavement design based on two properties of materials— cohesion and friction. These properties of treated or untreated base,sub-base or subgrade materials are determined by tests in the Hveem stabilometer developed by F.N. Hveem and R.M. Coumany (4) of California Division of Highways (CDH) which measures the horizontal pressure developed in a short cylindrical sample loaded vertically on its ends. This device was conceived to measure the stability of both field and laboratory samples of bituminous pavement and treated base courses. Loads are applied vertically; the resulting horizontally developed stresses and the vertical stresses are measured. The vertical and horizontal pressure are utilized in the following equation for calculating stabilometer resistance values.

$$R = 100 - \frac{100}{(2.5/D_2)(P_v/P_h - 1) + 1} \quad ....(2.3)$$

where, R = resistance value

P_v = applied vertical pressure (160 psi)

P_h = transmitted horizontal pressure at P_v=160 psi

D_2 = displacement of stabilometer fluid necessary to increase horizontal pressure from 5 to 100 psi, measured in revolutions of a calibrated pump handle.

Cohesion is measured by means of the cohesiometer, an apparatus capable of breaking small beams of base course materials. The base course materials are then ranked on the basis of stabilometer resistance values and cohesiometer values.

iv) California Bearing Ratio Test

The California Bearing Ratio test abbreviated as CBR is the most widely used method of evaluation of subgrade, sub-base and base course materials. The method was first developed by the

California Division of Highways and then adopted and modified by U.S. Corps of Engineers, in 1961. The American Association for State Highway Officials (AASHO) accepted this test in 1963 with Designation T193-63 for determining the bearing values of subgrade soils and some sub-base and base course materials containing only a small amount of material retained on the 20 mm (3/4 in.) sieve.

CBR test is a penetration test wherein, a standardized plunger, having an end area of 1935 mm² (3 sq.in.) is caused to penetrate the sample at a standard rate of 1.25 mm/min. (0.05 in/min.). A load-penetration curve is then drawn. CBR values are then computed from this load-penetration plot at penetrations 2.54 mm (0.1 in.) and 5.0 mm (0.2 in.) and the greater value is used for design of pavement where,

$$CBR = \frac{\text{Measured Force}}{\text{Standard Force}} \times 100\% \quad .....(2.4)$$

Standard Forces are 13.24 kN and 19.96 kN for penetrations 2.54 mm and 5.0 mm respectively.The test is to be carried on sample of sub-base or base material compacted to the moisture and density condition which site investigation and considerations of the construction methods and plant to be used, indicate to be appropriate. For design purpose the CBR value of the base and sub-base course at worst condition is required which can be obtained by testing the sample, after being saturated. Due to swelling of the specimen, the top surface may be loose to some extent. Therefore the stress-strain curve obtained from the penetration test sometimes will be concave upward which requires correction by moving it to the right. By CBR value it means corrected value when this correction has been applied to the curve. A surcharge weight 4.5 kg (10 lbs) is usually applied on the sample during soaking and testing of the sample to simulate the weight of pavement and to prevent heaving up around the plunger during the test.

Laboratory CBR values obtained on samples compacted in confining steel moulds, having an internal diameter of 152 mm (6 in.), are obviously to be checked in the field after construction. This can be done by comparing the densities in the laboratory and in the field or directly penetrating the course in the field. Field CBR test is basically the same as the laboratory test but a correlation is to be established to correct the field values for saturation. The laboratory CBR values are expected to be slightly higher than those obtained in the field because of the confining action of the mould in the laboratory. Precautions must be taken during field tests to ensure intimate contact of the plunger with an undisturbed surface of the material tested.

2.4 LIMITATIONS OF CBR TEST

CBR test is the most widely used imperical penetration test which can be carried out on most types of soil ranging from heavy clay to material of medium gravel size. The test was originally devised to provide a rational method of design for flexible pavements (such as macadam or asphalt), but it can also be applied to the design of rigid (concrete) pavement and granular base courses. Tests data are applicable to the design of airfield runways and taxiways as well as roads. CBR test can be used for evaluation of subgrade as well as sub-base and base course materials, and the results obtained enable maximum utilization to be made of low cost materials where better quality material is not available.

CBR test has many limitations primarily because of its arbitrary nature. The mould size, the plunger size and the maximum aggregate size for the test limited the application of the test for many granular mixes. Standard procedure must be followed throughout the test. Deviation from rate of loading, size of penetration plunger, and size of compacted sample may invalidate the test result. CBR value is a dimensionless number which is regarded as an index property, the application of which is restricted to pavement construction. The worldwide accepted CBR method of testing is largely criticised for its confining boundary effects during testing of granular base or sub-base aggregates. The CBR design approach is unsatisfactory when new situations such as heavier axle loadings, shortage of conventional high quality aggregates, untypical environmental conditions, changes in compaction techniques, etc. are met, which are at variance with the past experiences upon which specifications are based.

CBR test is not applicable for granular mixes having significant proportion of aggregates over 20 mm size where replacement is necessary for aggregates larger than 20 mm size as this replacement causes change in grading (shown in Fig. 4.2 and Fig. 4.3) which ultimately change the strength of the mixture. For some aggregates and gradings, this replacement of large sized particles may change the whole behaviour of the material in respect of stability, degradation and load distribution. Hence complications arise during the correct assessment of the materials as CBR test is not exactly applicable to these aggregate mixes. To avoid the worst of the problems associated with the inherent confining effect of CBR mould and its plunger size, Lees and Bindra (13) proposed a Modified Bearing Ratio (MBR) test for large sized base/sub-base aggregates.

2.5 MODIFIED BEARING RATIO (MBR) TEST

The Modified Bearing Ratio test abbreviated as MBR test was first proposed by Lees and Bindra (13) for larger sized (upto 40 mm) base/sub-base aggregate. It is a modified form of California Bearing Ratio (CBR) test. Like CBR test, the apparatus of MBR test is also simple consisting of a mould, a plunger and surcharge weights. The inside dimension of the mould is 400 mm diameter by 200 mm height and plunger diameter is 100 mm. The surcharge weight* in MBR test is equivalent to that provided in the standard CBR test. In MBR test, aggregates are also compacted using corresponding moisture content applying appropriate compacting energy**. The procedure of loading in MBR test is almost similar to that of CBR test. MBR test apparatus are shown in Fig. 2.2 and Fig. B-7 and test procedures are shown in Appendix-B from Figs. B-8 to B-10 . Like CBR test a constant rate of penetration of 1.25 mm/min. is applied to obtain a load penetration plot and then MBR value is derived from this plot for a penetration of 2.54 mm.

Zakaria (14) carried out laboratory investigations to compare CBR and MBR values for different granular mixes which are shown in Table 2.10. In those investigations, two types of aggregate such as crushed bricks and crushed quartzite gravel were used and three aggregate gradings ranging from single size to dense were employed.

Table 2.10 shows that the MBR and CBR values vary widely with different initial gradings for both types of aggregate. It is also seen that for dense initial grading, the bearing ratio

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* Surcharge weight in MBR test is determined from equivalent surface area.

** Compaction energy in MBR test is determined from equivalent volume.

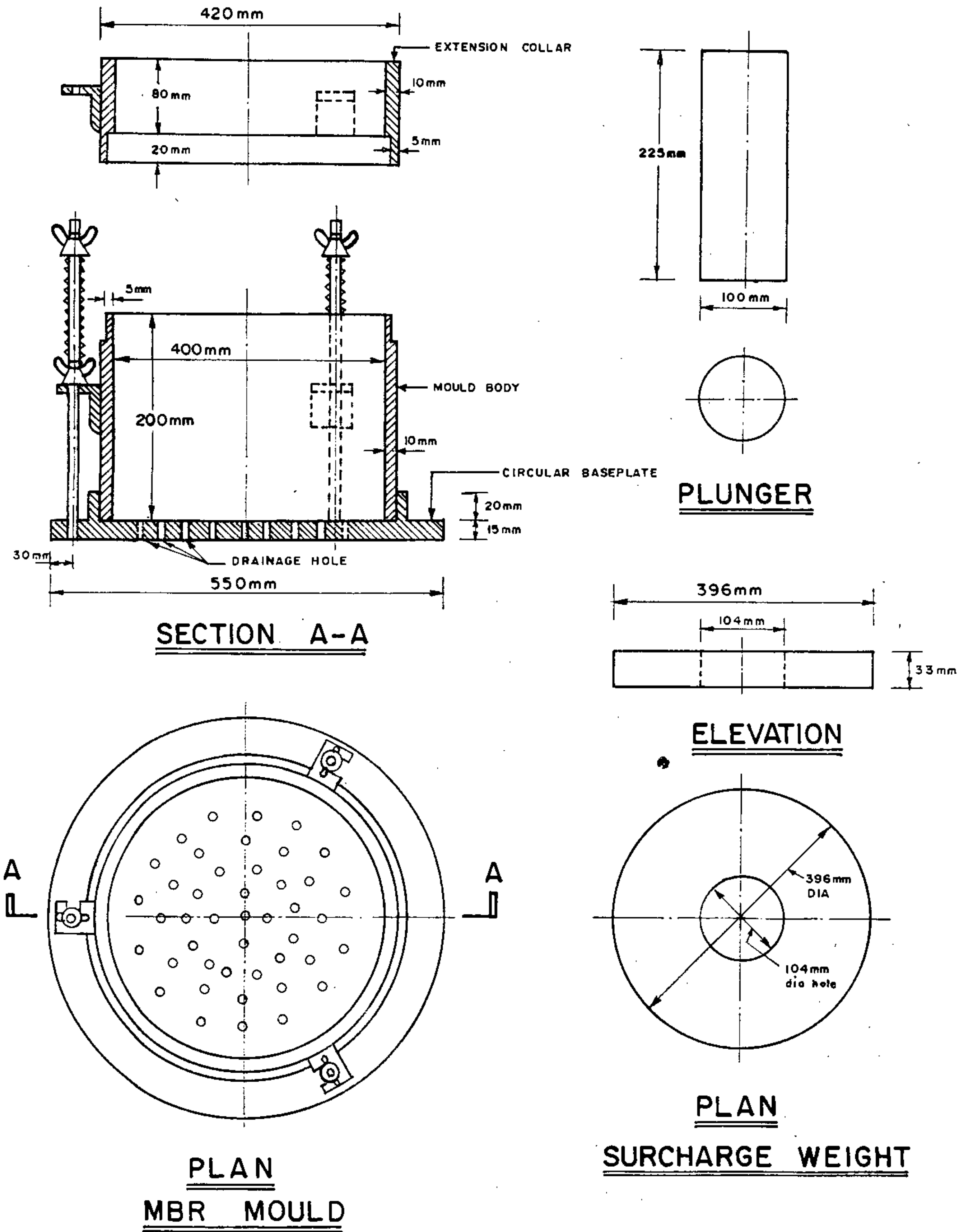


FIG. 2.2. MODIFIED BEARING RATIO TEST APPARATUS.

Table 2.10 Grading and Load Bearing Characteristics of Experimental Aggregates

Grading type	Coef. Cu	Max. size mm	Percent passing 75 µm	Maximum dry density kg/m³		CBR value at omc. for BS vib. comp.		MBR value at omc. for BS vib. comp.	
				Brick	Q'zite	Brick	Q'zite	Brick	Q'zite
Single	1.2	14	0	1450	1835	80	75	26	25
Open	12	50	2	1660	2260	130	105	44	30
Dense	187	40	10	1715	2370	185	190	60	62

values for brick and quartzite aggregate do not differ significantly but for the gradings open and single size the values for brick aggregate are markedly higher than those for quartzite aggregate. This difference according to Zakaria (14) was attributed to the high compactability of the relatively soft brick aggregate.

2.6 RESEARCHES ON GRANULAR AGGREGATE MIXES FOR HIGHWAY CONSTRUCTION

In the early days, roads were constructed from stone blocks or flat pieces of ledge rock. The first hard surfaces were constructed in Mesopotamia soon after the discovery of the wheel about 3500 B.C. A stone surface road was constructed as early as 1500 B.C. on the island of Crete in the Mediterranian sea. But actual development of the roads started in the 18th century in France and England when crushed aggregates were used for road construction. As highway is closely related to human civilization so a lot of research works were carried in connection with highway construction and many highway research organisations were formed like Transport and Road Research Laboratory (TRRL) in U.K., American Association for State Highway and Transportation Officials (AASHTO) in U.S.A., Indian Roads Congress (IRC) in India and Bangladesh Road Research Laboratory (BRRL) in Bangladesh. Both laboratory and field studies were carried on the selection of suitable type of construction, suitable type of aggregate, gradation of aggregate and test procedures for evaluating the properties of aggregate. Some laboratory and field studies are discussed in the following articles.

2.6.1 Laboratory Studies

Load supporting capacity indicated by the stability and density of the aggregate soil mixture is the principal requirement of a WBM base or sub-base course. Different organisations carried investigations to find a suitable gradation to have dense and stable mixtures. These works in the form of recommendations were discussed in Article 2.2.

The proportion of the fines (material passing NO.200 sieve) to coarse fraction is also an important factor to have a dense and stable mixture. Deklotz's (15) works discussed by Yoder (16) on "Effect of varying the quantity of fines of highway aggregates on their stability" reveals that there is an optimum amount of fines for a particular aggregate-soil mixture at which the mixture will have its maximum density and stability for a particular compactive effort. According to Yoder (16) results of laboratory studies on a well graded stone aggregate show that the maximum density occurs when the mixes contain 8 to 10 percent fines passing a NO.200 (75 µm) sieve.

Yoder (16) also has shown the effect of compactive effort on density and stability of a stone aggregate mixture for a particular gradation. He concluded that the more the compactive effort, the more will be the density and stability. But the response for the increase is more pronounced in the case of stability rather than density.

Laboratory studies also were carried on to compare the density and stability values between round shaped naturally occurring gravel mixtures and crushed stone mixtures. These tests and field experiences have shown that crushed particles have, in general, more stability than round grained material primarily due to added grain interlock (4).

Tests to find the effect of the physical properties of binder soil on the stability of the mixture were done by Makdisi-Ilyas, Faiz (17). They found that the effect of plasticity is detrimental to the strength with higher amount of material passing NO.200 (75 µm) sieve. This happens due to the reason that plasticity is dependent upon the amount of material passing NO.30 (600 µm) sieve is increased. The required amount of material passing NO.30 (600 µm) sieve for maximum density is 15.30 percent and when the binder content exceeds this value, the plasticity becomes important.

A laboratory degradation study was carried on both brick and quartzite aggregates due to different types of degrading energies by Zakaria (18). In this study, aggregates were subjected to (i) a vibratory compaction for 3 minutes by a 750 kW Kango hammer in a standard CBR mould (British standard (BS) vibratory compaction), (ii) a static load of 400 kN applied at a rate of 40 kN per minute through a plunger of 152 mm diameter in a 154 mm diameter mould (BS Aggregate Crushing Test (ACT) mould) and (iii) impact and abrasion in the Los Angeles machine for 500 revolutions using 6 steel shots. The overall degradations due to these degrading energies were finally found from the "D_{50} ratio" and "sum of difference of percent finer" values. It was seen that degradation was small for both types of aggregate when BS vibratory compaction hammer was used. He claimed that as most of the aggregate degradation is occurred during initial compaction in the layer in the field, vibratory compaction in laboratory mould can provide estimates of compaction degradation provided the same method (vibratory) of compaction is adopted in the field. This study supports the appropriate selection of vibrating hammer for sample compaction.

A laboratory study was carried by Lees and Bindra (13) on granular mixes having significant proportion of aggregates over 20 mm size and established a correlation between MBR and CBR. They proposed that the correlation between MBR and CBR at 2.5 mm and 5.0 mm penetration for a wide range of moisture content and grading is

$$MBR = 0.393\ CBR - 8.43 \quad \ldots\ldots\ldots\ldots\ldots (2.5)$$

Later, Zakaria and Less (19) carried out further laboratory investigation and they showed a linear correlation between CBR and MBR at 2.54 mm penetration where

$$MBR = 0.40\ CBR - 8.5 \quad \ldots\ldots\ldots\ldots\ldots (2.6)$$

This above relationship (2.6) is further modified for a wide range of moisture content and grading and a correlation between MBR and CBR is obtained with a coefficient of determination of 98.1 percent for 34 degrees of freedom, where

$$MBR = 0.19 + 0.33\ CBR \quad \ldots\ldots\ldots\ldots\ldots (2.7)$$

A comparison of CBR and MBR values at optimum moisture content for different grading for BS vibratory compaction was shown in Table 2.10. Using these values alongwith other CBR and corresponding MBR values a simple linear relationship between CBR

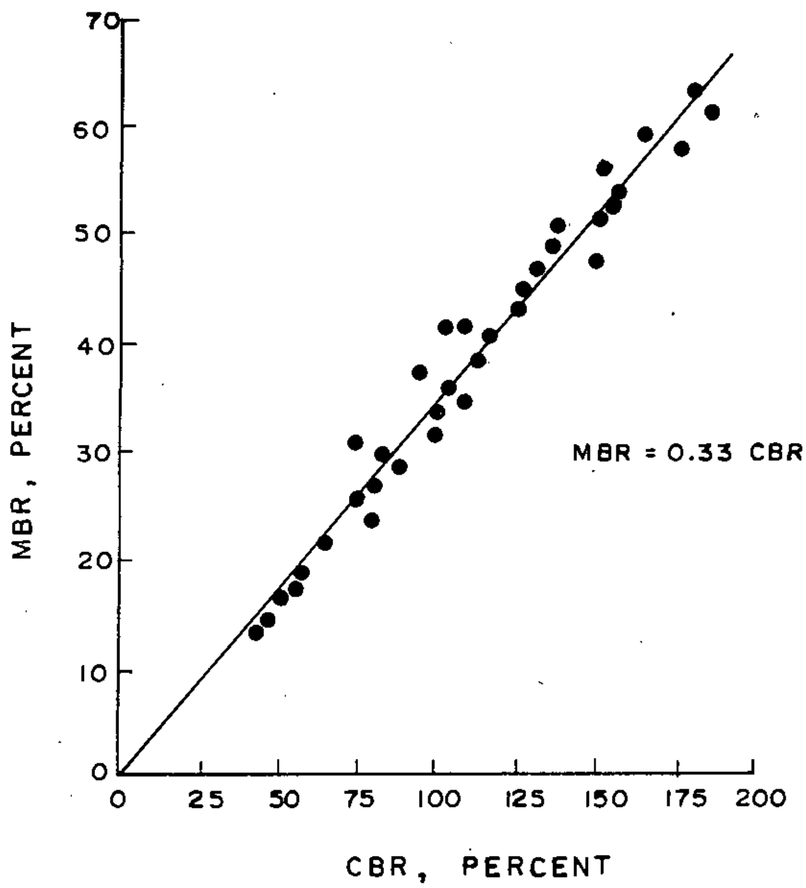


FIG. 2.3 RELATION BETWEEN MBR AND CBR FOR A WIDE RANGE OF INITIAL GRADING, AGGREGATE TYPE AND MOISTURE CONTENT (14)

and MBR was developed by Zakaria (14) for a wide range of initial grading,aggregate type and moisture contents with a correlation coefficient of 0.95 which is shown in Fig.2.3, where

$$MBR = 0.33\ CBR \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.8)$$

2.6.2 Field Studies

There has been a vast increase in highway traffic after the second world war and many test roads were constructed to observe the performance of the roads and to develop design methods. In 1949, a research project was set up by Highway Research Board in Maryland, U.S.A, to determine the relative effects of various axle loads and configurations on distress of pavements (4). This test was intended for observation of the behaviour of concrete pavement constructed on a improved granular sub-base material and no findings regarding base course was obtained.

The AASHO road test was done prior to AASHTO road test near Malad, Idåho, USA. For flexible pavement the test recommends a total thickness of pavement and base for different axle loads.

The most intensive and extensive research on road was a co-operative project sponsored by AASHO at Ottowa, Illinois, USA. In the principal flexible pavement test section the base course was constructed by a well-graded crushed lime stone and the sub-base by a uniformly graded sand-gravel mixture. Other test sections were constructed with bases by a well graded uncrushed gravel, a bituminous plant mixture and a cement treated aggregate. On the basis of the performance records of the various test sections a new concept to evaluate the pavement infrastructure was introduced which is PSI - Pavement Servicibility Index. The important conclusion made by them about bases is —— "the performance of the treated gravel base is definitely superior to that of the untreated crushed stone base".

The first British full scale field experiment began in 1949 on a section of the A1 trunk road in Yorkshire U.K. They used three types of base material—— tar macadam base;dry bound stone base; hand pitched base. From observations over a period of ten years they found that the open textured tar macadam bases performed much better than either the dry bound stone bases or the hand pitched bases. In addition the hand-pitched base showed increased surface deformation and found worse than dry bound stone base and is no longer recommended.

A further experiment, also on the A1 road at Alconbary Hill in Huntingdonshire, U.K was initiated in 1957 to compare the performances of five different base materials laid to various thickness. The base materials were cement stabilized sand, wet mix slag, lean concrete, tar macadam, and hot rolled asphalt. Experimental sections, each 2000 ft (610 metre) long were constructed on a sand sub-base of varying thicknesses. Results after six years show that the sections with rolled asphalt bases have performed the best whilst those with sand-cement base are clearly the worst. All these field tests for base courses were comparative studies between different types of bases, treated or untreated but no test was done with one specific base by varying its components, materials etc.

Field experiments were done by Drake (20) of Kentucky Department of Highways on the performances of flexible base courses designed primarily to improve the riding qualities of high type bituminous pavements., Four inch (10 cm) dense graded lime stone base was constructed with 1 inch (25 mm) downgraded; 5 to 15 percent passing NO.200 (75 µm) sieve, well graded material over 4 inches (10 cm) water-bound macadam sub-base course. Observations and measurements showed that the combination of the two courses could be built satisfactorily, the dense graded aggregate produced a high base density and the possibilities for finishing to uniform section were much better with the dense graded aggregate than with the macadam.

Field experiments were also done by Saxena, Rajagopal and Justo (21) in India on the construction of WBM overlays on bituminous pavements. First alternate diagonal furrows were cut on the bituminous surface of the existing pavement by removing the bituminous layer to the full depth and then WBM overlays were constructed over bituminous pavement. WBM overlays were constructed at six different localities in Karnataka, Gujarat, Maharashtra and Rajasthan in India. They showed that of the six test sections, the performance of three sections were satisfactory. Based on these experiments, they proposed that construction of WBM overlays on bituminous pavements is economical and suitable for the roads in low rainfall regions or at locations where the surface and sub-surface drainage system are very effective.

Cedergren (22) made field CBR tests on granular sub-base and bases. He criticised the general idea that the CBR values on large macadam type construction will be higher than with smaller aggregates. In testing coarse grained materials he found almost invariably that CBR values are not too high but too low.

Literature review reveals that only a limited research was done on WBM construction. In all these research aggregates were conventional nature aggregate (i.e. boulder and gravel). No research work was carried out on WBM using brick aggregate. The present investigation on WBM using brick aggregate was undertaken due to extensive use of brick aggregates for road construction in Bangladesh.

CHAPTER-THREE

MATERIALS

3.1 GENERAL

Water-bound macadam mixture is normally composed of coarse aggregates and screenings. Coarse aggregates may be either crushed or broken stones, crushed slags, or over-burnt brick aggregates. Screenings are normally stone chips. The use of screenings may be omitted in case of soft aggregates such as brick metal, kankar and laterite. The coarse aggregates and screenings should conform the required grading specifications as the design of a pavement is valid only if the materials of construction meet the specification. Various alternate materials are available but the pavement should be constructed with the materials which are economically cheap and give sufficient stability and durability. WBM constructions are mainly used as base or sub-base of flexible pavement which is the main structural component to distribute the superimposed load to the subgrade. If there is some failure in the sub-base or base course due to the weaknesses in the material, deterioration after being progressive, chuckholes may be formed. This type of distress gradually leads to the eventual failure of the road. The properties of these materials individually or the mixture as a whole are of such critical importance to pavement life that examples of pavement failure traceable to improper material selection and use are numerous. Therefore, special emphasis must be given for the selection of materials for construction of base and sub-base courses.

3.2 MATERIALS USED FOR THE RESEARCH

The materials used in the WBM mixture (to be used in base or sub-base construction) consists of crushed picked jhama brick, medium to fine sand and silt and clay fractions of soil for road construction in Bangladesh. These three components of the mixture were selected to meet the desired gradation. Main portions of the coarse and fine aggregates in the mixture were obtained by crushing picked jhama bricks which were collected from UNITED ENTERPRISE, NARAYANGANJ. Highly over burnt and porous bricks were rejected. Very small amount of materials between 600 µm (NO.30) sieve to 75 µm (NO.200) sieve were obtained during crushing of these bricks due to higher hardness. To meet the grading requirement between 600 µm sieve to 75 µm sieve some medium to fine Sylhet sand was blended whose fineness modulus was 2.23. Non-plastic soil from flood control dam near Hazaribagh was used as fines passing 75 µm (NO.200) sieve having a specific gravity 2.78 and unit weight of 1135 kg/m^3 (dense condition).

3.2.1 Coarse Aggregates Used for the Study

Coarse aggregate for WBM construction has been defined as that portion of the mixture which is retained on 2.36 mm (NO.8) sieve according to the Asphalt Institute. Coarse aggregates were obtained by crushing the picked jhama bricks and crushing was done manually and brought to the sizes of 50 mm (2 in.) or less. The aggregates were then sieved using U.S. standard sieves and separated out in different fractions. They were then washed and dried and combined in appropriate proportions of designed gradations.

3.2.2 Screenings (Fine Aggregates and Fines) Used for the Study

Screenings for WBM normally composed of fine aggregates and fines.

Fine Aggregate:

Fine aggregate is that portion of the aggregate material in the mixture which passes 2.36 mm (NO.8) sieve and is retained on a 75 µm (NO.200) sieve according to the Asphalt Institute. After crushing picked jhama bricks it is seen that all of the fine aggregates required for a well graded mixture are not obtained. Therefore some sand is to be blended with the crushed aggregate to meet the grading specification. For the gradations considered in this work medium to fine sand having fineness modulus between 2 to 2.5 is required to be mixed. The fine aggregate in the mixture is therefore a mixture of crushed brick and sand.

Fines:

Fines are that portion of the mixture which passes 75 µm (NO.200) sieve. From crushing of over burnt picked jhama brick a negligible amount of fines are obtained. The major portion of the fines is obtained by mixing dry powdered soil which is the cheapest among the available materials. Non-plastic soil from adjacent site can be mixed with aggregate blend to have the desired percentage of fines in the mixture.

3.3 TEST PROCEDURES FOR DETERMINING PHYSICAL PROPERTIES OF AGGREGATES

Coarse Aggregate: Tests required to determine the physical properties of coarse aggregates are -

i) loose and dense unit weight (C29-T19),
ii) specific gravity (C127-T85),
iii) absorption of water (C127-T85),
iv) percent wear by Los Angeles Abrasion (C131-T96).

These tests were performed according to the procedures specified by ASTM and AASHTO standards (1984). Aggregate impact value (AIV), aggregate crushing value (ACV) and ten percent fines value (TFV) were determined according to the procedures specified by BS812:1975, Part-3. Physical properties of coarse aggregates and strength characteristics of experimental aggregates are shown in Table 3.1 and Table 3.2 respectively.

Table 3.1 Physical Properties of Coarse Aggregate

Name of the test		ASTM Designation	Test Results
Unit weight (kg/m^3)	Loose	C29-78	1089
	Dense		1185
Specific gravity	Apparent	C127-84	2.33
	Bulk		1.79
Absorption (percent)		C127-84	13

The minimum unit weight of aggregate required for WBM base courses according to ASTM specification D694-71 and for WBM surface course according to AASHTO specification M77-64 is 65 lbs/cft (1041 kg/m^3).Again according to IRC specification IRC:19-1972,the minimum unit weight of aggregate shall not be less than

1120 kg/m³ for WBM construction. Table 3.1 shows that the loose and dense unit weight of picked jhama brick aggregates used in the study are 1089 kg/m³ and 1185 kg/m³ respectively which are much higher than the above values. Therefore, the selection of the experimental aggregates is appropriate.

Table 3.2 Strength Characteristics of Experimental Aggregate

Properties	Test Results
Los Angeles Abrasion (Grade-B), LAA (Percent)	38
Aggregate impact value, AIV (Percent)	31
Aggregate crushing value, ACV (Percent)	38
Ten percent fines value, TFV (kN)	80

Table 3.2 shows that average percent of wear by Los Angeles abrasion, aggregate impact, aggregate crushing and ten percent fines values of picked jhama brick aggregate are 38%, 31%, 38% and 80 kN respectively. The strength characteristics of coarse aggregate required for WBM construction according to the specifications given by various highway agencies were discussed in Article 2.2. So it is seen that all the aggregates are within the limit of ASTM:D694-71, AASHTO: M77-64, IRC:19-1972 and BRRL (Sub-base) specification.

Fine Aggregate:

Tests were performed on fine aggregates to determine the loose and dense unit weight, specific gravity and water absorption. Tests procedures specified by ASTM (1986) specifications C29 and C128 were followed to determine the above properties. The fine aggregates were mixture of finer portion of crushed brick and sand. About eighty to ninety percent sand were used in the mixture. Physical properties of fine aggregates are shown in Table 3.3.

Table 3.3 Physical Properties of Fine Aggregates

Name of Test		ASTM Designation	Test Results
Unit weight (kg/m^3)	Loose	C29-78	1339
	Dense		1466
Specific gravity	Apparent	C128-84	2.7
	Bulk		2.4
Absorption (percent)		C128-84	4.45

Fines:

The specific gravity of the natural fines was determined according to ASTM standard D854-58 (Reapproved in 1972) and loose and dense unit weight of fines were determined according to ASTM standard C29-71. The values are tabulated in Table 3.4. A hydrometer analysis of fines was also done to determine the grain size distribution and the results are tabulated in Appendix-A.

Table 3.4 Physical Properties of Fines (Soil)

Name of the test		ASTM Designation	Test Results
Specific gravity		D854-58	2.78
Unit weight (kg/m^3)	Loose	C29-71	912
	Dense		1135

3.4 DISCUSSIONS ON AGGREGATE PROPERTIES

Some research works were carried on the use of brick aggregates for road construction in Bangladesh, which were discussed in Article 1.2.3. Physical and strength properties of brick aggregate used by some researcher alongwith the experimental aggregates are shown in Table 3.5.

Table 3.5 Comparison of Physical and Strength Characteristics of Brick Aggregates Used by Some Researcher

Test properties		Name of researcher and aggregate type			Experimental picked jhama brick aggregate
		Hoque(3)	Zakaria(4)	Sobhan(5)	
		Coal burnt picked jhama brick aggregate	Gas burnt picked jhama brick aggregate	Gas burnt picked jhama brick aggregate	
Dense unit weight kg/m³	Coarse aggregate	1078 (67.3 lbs/cft)	1213 (75.72 lbs/cft)	1210	1185
	Fine aggregate	1132 (70.7 lbs/cft)	1340 (83.65 lbs/cft)	1270	1466
Apparent specific gravity	Coarse aggregate	2.34	2.49	2.53	2.33
	Fine aggregate	2.64	2.62	2.42	2.70
Absorption percent	Coarse aggregate	14.0	7.29	8.95	13.0
	Fine aggregate	7.76	4.23	7.07	4.45
LAA(Grade-B),percent		36.45	32.0	34.7	38.0
AIV, percent		-	-	16.0	31.0
ACV, percent		Above 30	-	29.0	38.0
TFV, kN		75 (7.5 tons)	-	90	80

Comparing the aggregate requirement values for WBM by various highway organisations (discussed in Article 2.2) and the values in Table 3.5, it can be said that picked jhama brick aggregate is suitable for WBM (base or sub-base) construction. From the above comparison it is also seen that the experimental bricks are similar to coal burnt picked jhama variety.

CHAPTER-FOUR

LABORATORY INVESTIGATION AND TEST RESULTS

Comparing the grading specification requirements (as discussed in Article 2.2) and the properties of the materials (as discussed in Chapter-three), it can be said that picked jhama brick aggregate is suitable for WBM (Base and sub-base) construction from the consideration of strength and toughness. A suitable compaction method was first selected from degradation characteristics of aggregate-sand-soil mixture for a particular gradation using different compactive efforts. In WBM construction, screenings and coarse aggregate may be used either in mixed form or separately depending on the Highway agencies specifications. Screenings are mixed with coarse aggregate according to BSI and BRRL specifications while placed separately on the top surface of coarse aggregate according to ASTM/AASHTO and IRC specifications. So a search was made for selecting the best way of using screenings which was discussed later in Article 4.3. Then a suitable gradation was found out after performing MBR test with the brick aggregates-sand-soil mixture for different gradations described in Article 4.4. Since a small amount of fines obtained during crushing of picked jhama brick, soil was used in WBM mixture. But the mixture will be plastic if the soil is plastic (i.e PI greater than 6) which is not suitable for base coarse construction. Therefore, an investigation is carried to find the plasticity characteristics of soil passing 420 µm (NO.40) sieve.

The laboratory works done for the research and the test results are discussed in the following articles.

4.1 DEGRADATION OF AGGREGATES DUE TO DIFFERENT COMPACTIVE EFFORTS

Unbound aggregates are used in almost all road construction in both developed and developing countries, sometimes in the base course as the main structural component of the pavement and nearly always as a sub-base course and drainage layer. Aggregate degradation has been reported by many investigators to occur, in unbound layers of pavements, during construction and also during service life (West et al 1970, Lees and Kennedy 1975 and Wylde 1976). The phenomenon of degradation has been recognised as one of the important factors affecting the performances of unbound aggregate in base, sub-base courses or in unpaved roads.

Degradation is defined in this research as the breakdown of the particles or reduction in size of particles which occur during the process of compaction. The extent of particle breakdown depends on aggregate strength and many other factors i.e. initial grading of aggregate, degree of load, type of degrading energy or compactive effort, repetitions of load etc.

But only degradation due to compactive efforts for a particular gradation was considered in this research work. Here degradation study was undertaken to select a suitable compaction method. Three types of compaction equipments such as British vibratory compaction hammer, 4.5 kg (10 lbs) and 2.5 kg (5.5 lbs) hammer (shown in Figure B-1 in Appendix-B) were used in this study. First the selected graded aggregate mixture (shown in Table 4.1) was taken in a CBR mould of internal diameter 152 mm (6 in.) and then compaction was done according to BSI method in dry condition. After carefully collecting the degraded aggregate mixtures from the mould sieve analysis were performed in order to obtain the grain size distributions of the mixtures. Grain size distribution curves for initial and degraded aggregates obtained after degradation process are shown in Fig. 4.1 and degraded gradations are contained in Table 4.1.

4.2 APPLICABILITY OF CBR TEST FOR WBM CONSTRUCTIONS

CBR test has many limitations primarily because of its arbitrary nature. Laboratory CBR test is done on compacted materials in a 152 mm (6 in.) diameter mould. The mould size, the plunger size and the maximum aggregate size for the test limited the application of the test for many granular mixes. The shortcomings of CBR test were discussed in Article 2.5. Some highway agencies specify granular mixes having significant proportion of aggregates over 20 mm size and their strength is also specified in terms of CBR. For example, the Bangladesh Road

Table 4.1 Degradation of Dense Graded Brick Aggregates due to Different Compactive Efforts

Sieve size	Percent passing by weight			
	*Initial ℄ grading	Final ℄ grading obtained after compacting by		
		2.5 kg hammer	4.5 kg hammer	Vibra. hammer
20 mm (3/4")	100	100	100	100
10 mm (3/8")	85	89	90	86
5 mm (NO.4)	65	70	72	68
2.4 mm (NO.8)	50	54.5	58	53
0.6 mm (NO.30)	27.5	33	34	31.5
0.3 mm (NO.50)	18.5	24	25.5	23
0.075 mm (NO.200)	12	16	17	15.5

* Initial grading was selected from BRRL specification (sub-base; Type-B).

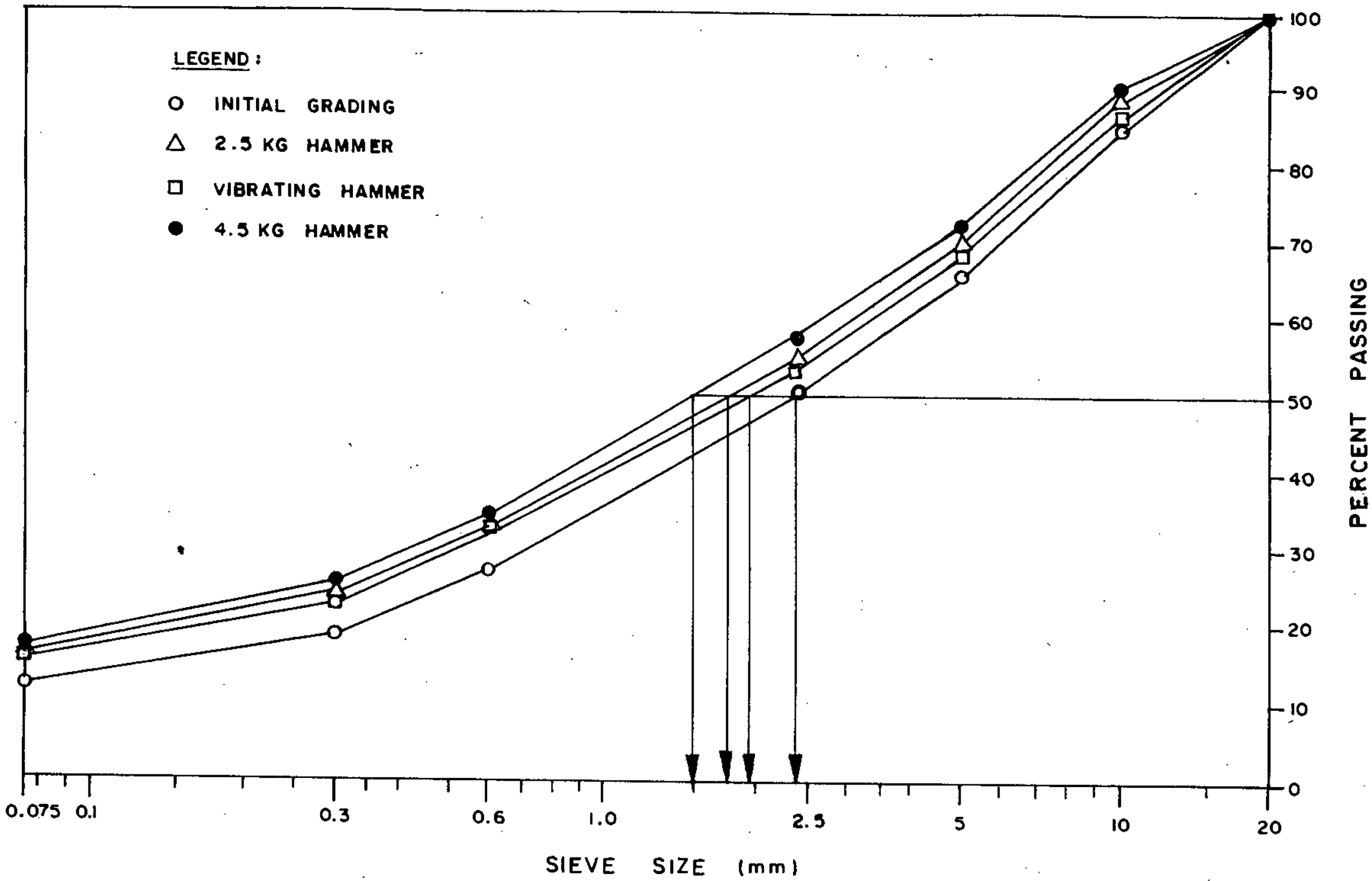


FIG. 4.1 DEGRADATION OF DENSE GRADED BRICK AGGREGATES SUBJECTED TO DIFFERENT COMPACTIVE EFFORTS.

Research Laboratory specified untreated granular mixes with 50 mm maximum size for base and sub-base courses. But the maximum size of aggregates for CBR mould is 20 mm (3/4 in.). According to AASHTO T193-63 and ASTM D1883-73 specifications, material passing the 2 in. sieve and retained on the 3/4 in. sieve shall be replaced with material passing the 3/4 in. sieve and retained on the NO.4 sieve. But according to BS1377:1975 specification (23), if CBR test is to be made on samples containing larger particles than 20 mm and if the percentage of the fraction retained on the 20 mm BS test sieve exceeds 25% of the whole then it should be replaced with a similar fraction of 20 mm to 5 mm of like material otherwise no correction is made. Fig. 4.2 and Fig. 4.3 show the change of grading after replacement of material retained on 20 mm (3/4 in.) BS test sieve for BSI base (i.e X_2) grading and BRRL base Type-A (i.e X_4) grading respectively. For some aggregates and gradings this replacement of large sized particles may change the whole behaviour of the material in respect of stability, degradation and load distribution. With the change of grading, the strength or CBR value of the mixes change which was discussed later in Article 4.5. Hence complications arise during the correct assessment of the materials as CBR test is not exactly applicable to these mixes. The grading specifications for WBM construction recommended by four highway agencies were discussed in Article 2.2 where most of the grading specifications used significant portion of aggregates over 20 mm (3/4 in.) size.

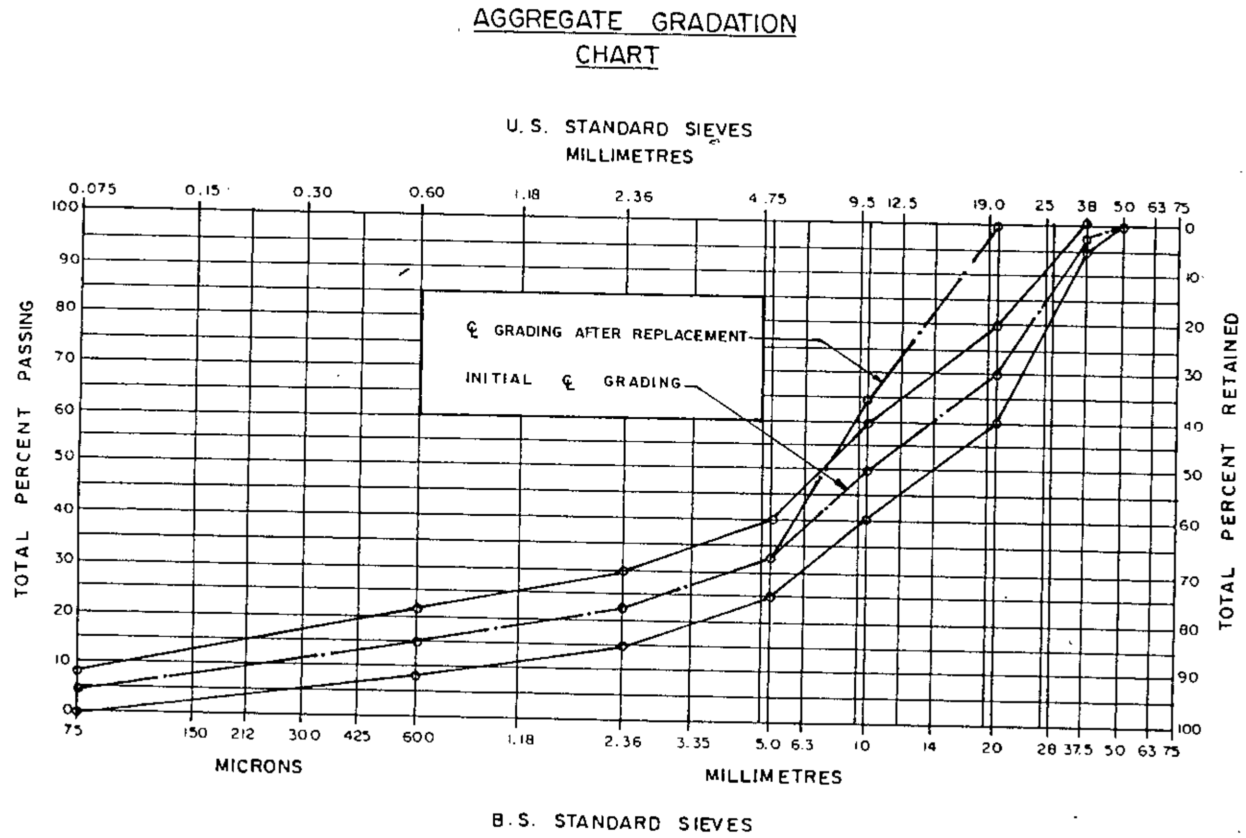


FIG. 4.2 CHANGE OF GRADING (X_2) DUE TO REPLACEMENT FOR CBR TEST.

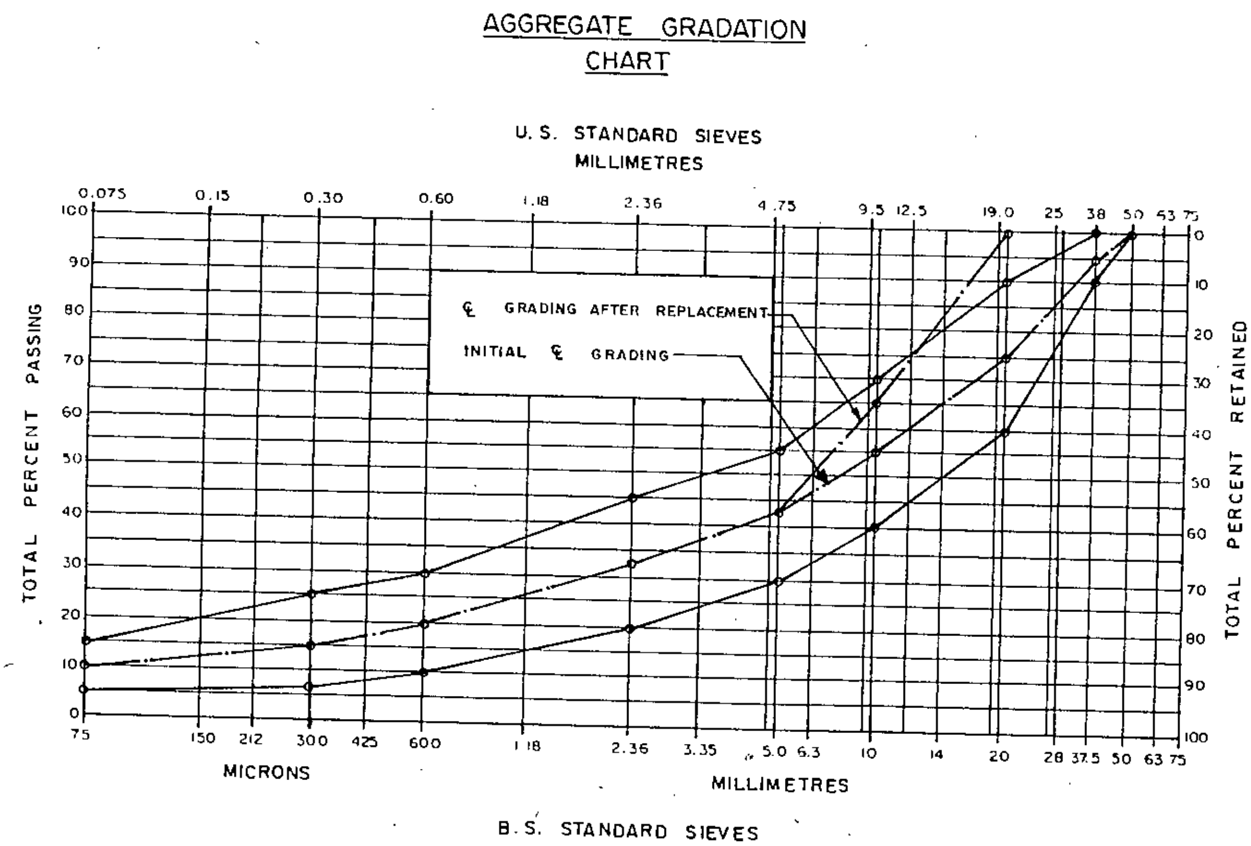


FIG. 4.3 CHANGE OF GRADING (X4) DUE TO REPLACEMENT FOR CBR TEST.

Therefore, it can be said that CBR test is not applicable to most of the gradings used.

4.3 EFFECT OF PLACING OF SCREENINGS ON CBR VALUES

WBM normally composed of coarse aggregate and screenings. Some agencies use screenings by mixing with the coarse aggregate and they specify coarse aggregate and screenings by a single gradation; while others use coarse aggregate and screenings separately and they provide separate grading specification for coarse aggregate and screenings. An investigation is therefore necessary to select the best way of placing of screenings which is one of the objectives of this research. Since BSI and BRRL provide a single grading specification for coarse aggregate and screenings, so their grading specifications were not considered in this study. ASTM/AASHTO and IRC provide separate grading specifications for coarse aggregate and screenings. In ASTM/AASHTO specification, the ratio of coarse aggregate and screenings is not fixed. But in IRC specification, the screenings shall be 24% to 28% of the total aggregate mixture for crushable type aggregate. IRC (Type-3) and AASHTO (Type-3) grading were adopted for this study where screenings were 28% and 25% of the total aggregate mixture respectively. For each type of grading, two types of sample were prepared; one with screenings at the top surface of coarse aggregate and another by mixing coarse aggregate and screenings. CBR test was then performed on each sample (after compacting by a vibrating hammer at optimum moisture content) which was shown in Figs. B-3 to B-6 in Appendix-B and test results were tabulated in Table A-1 and Table A-2 and load-penetration plots were shown in Fig. 4.4(a,b) and Fig. 4.5(a,b) respectively.

4.4 DENSITY-GRADATION RELATIONSHIP

Initially four aggregate gradations were selected from available literature. These gradations are listed in Table 4.2. The grain size distribution bands are shown in Fig. 4.6 to 4.9. The gradations are designated as X_1, X_2, X_3 and X_4. During selection, gradations having larger than 50 mm (2 in.) size were avoided. The gradations where screenings are separately used were not considered as well. Gradations X_1 and X_2 were taken from BSI recommendations for sub-base Type-1 (shown in Table 2.2) and base course (shown in Table 2.3) respectively. Gradations X_3 and X_4 were chosen from BRRL recommendations for sub-base Type-A (shown in Table 2.4) and base Type-A (shown in Table 2.5).

Coarse aggregates, fine aggregates, sand and soil were mixed together in proportions as specified in these gradations and optimum moisture content and maximum dry density were found for these mixtures according to BS vibrating hammer method under the designation BS1377:1975, Test-14. The results of these compaction tests are shown in a concised form in Table 4.3. The moisture

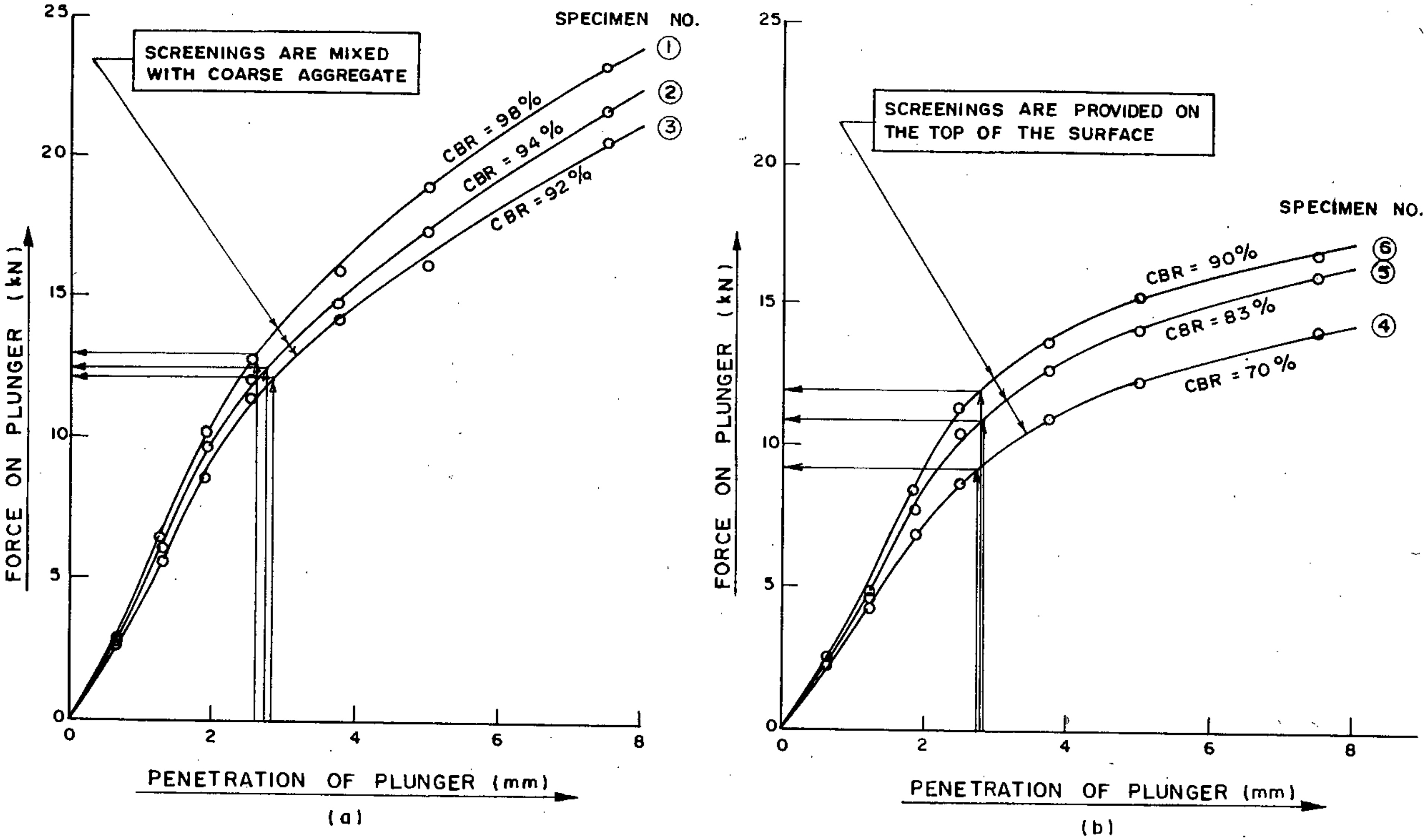


FIG. 4.4 CBR FOR DIFFERENT SPECIMENS (IRC TYPE - 3 GRADATION)

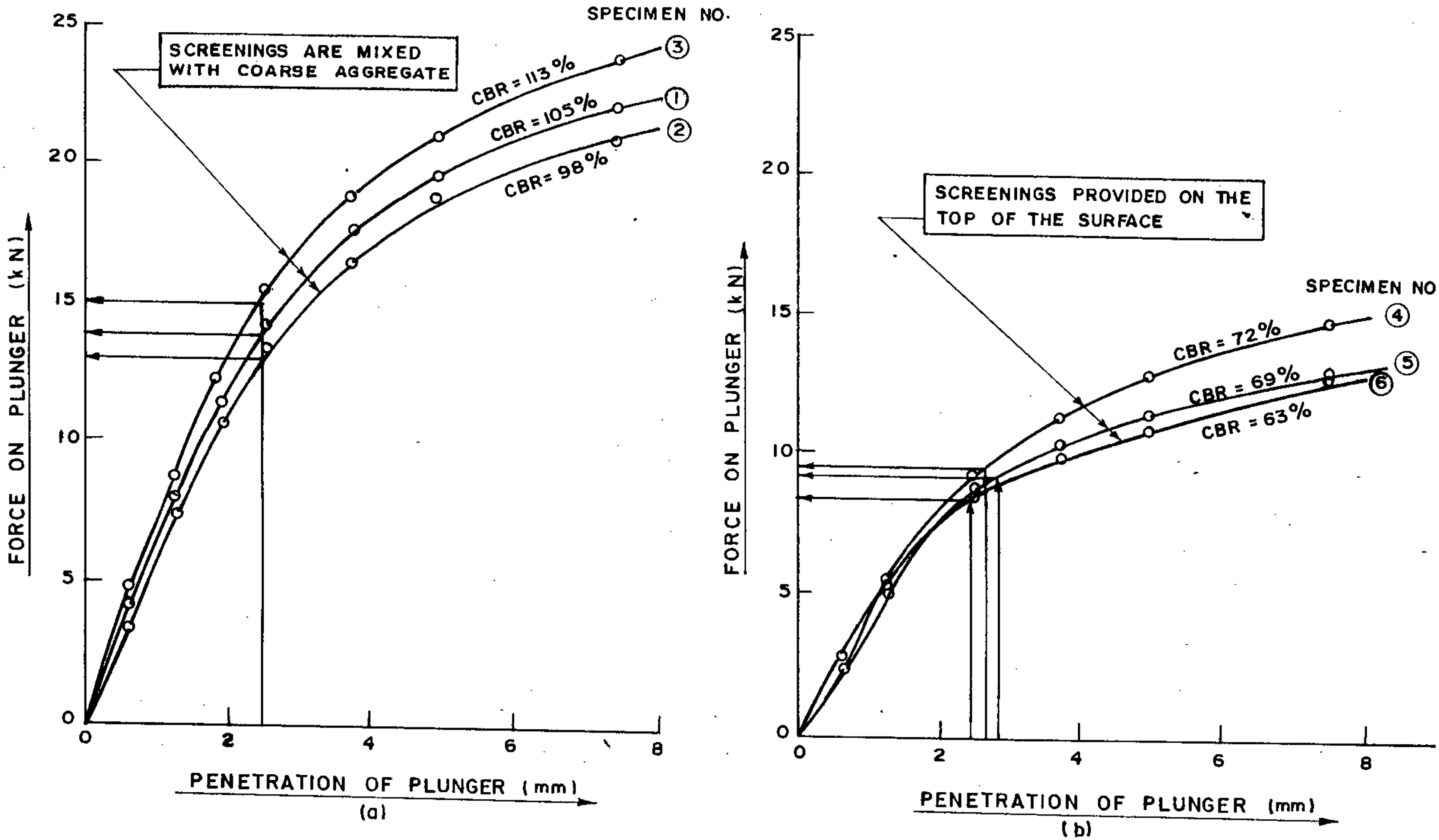


FIG. 4.5 CBR FOR DIFFERENT SPECIMENS. (AASHTO TYPE – 3 GRADATION)

4.2 Aggregate Gradation Requirements of Different Highway Agencies for WBM

Sieve size	Percent passing by weight for gradations			
	X_1	X_2	X_3	X_4
75 mm (3")	100	-	-	-
50 mm (2")	-	100	100	100
38 mm ($1^1/_2$")	85-100	95-100	85-100	90-100
20 mm (3/4")	-	60-80	55-95	60-90
10 mm (3/8")	40-70	40-60	35-75	40-70
5 mm (NO.4)	25-45	25-40	25-60	30-55
2.4 mm (NO.8)	-	15-30	15-50	20-45
600 µm (NO.30)	8-22	8-22	7-35	10-30
300 µm (NO.50)	-	-	-	7-25
75 µm (NO.200)	0-10	0-8	3-15	5-15

Note: X_1 = BSI sub-base (Type-1) gradation (shown in Fig. 4.6)

X_2 = BSI base course gradation (shown in Fig. 4.7)

X_3 = BRRL sub-base (Type-A) gradation (shown in Fig. 4.8)

X_4 = BRRL base (Type-A) gradation (shown in Fig. 4.9)

Table 4.3 Moisture Density-Gradation Data

Gradation	Moisture content (%)	Wet-density (kg/m³)	Dry-density (kg/m³)	Optimum moisture (%)	Maximum dry-density (kg/m³)
X_1	8.5	1586	1462	14%	1520
	11.3	1648	1481		
	13.0	1704	1508		
	14.1	1737	1523		
	16.5	1754	1506		
	18.3	1767	1494		
X_2	9.6	1628	1485	15%	1540
	11.4	1674	1503		
	13.5	1730	1525		
	15.0	1770	1539		
	17.2	1792	1529		
	18.8	1803	1518		
X_3	8.6	1640	1510	16%	1680
	12.0	1750	1563		
	14.5	1888	1649		
	15.7	1950	1685		
	18.0	1948	1651		
	19.8	1946	1624		
X_4	8.4	1613	1488	15%	1634
	10.6	1680	1519		
	12.3	1736	1546		
	14.5	1844	1610		
	16.2	1887	1624		
	17.8	1896	1610		

AGGREGATE GRADATION CHART

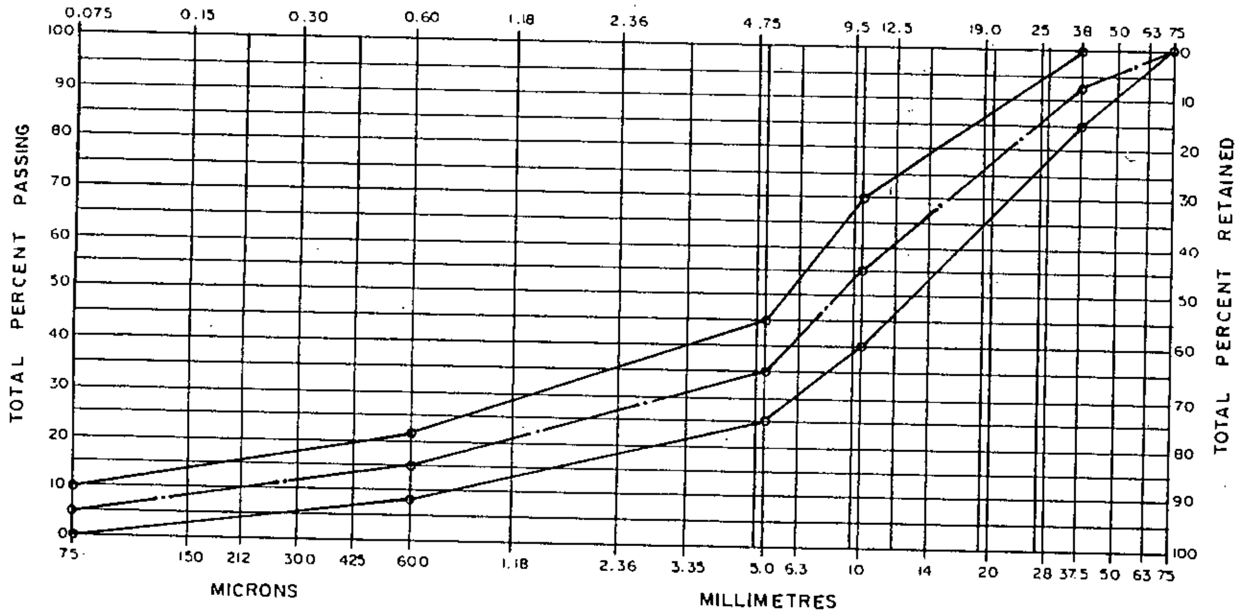


FIG. 4.6 GRADATION X_1 (BSI, SUB-BASE, TYPE-1)

AGGREGATE GRADATION CHART

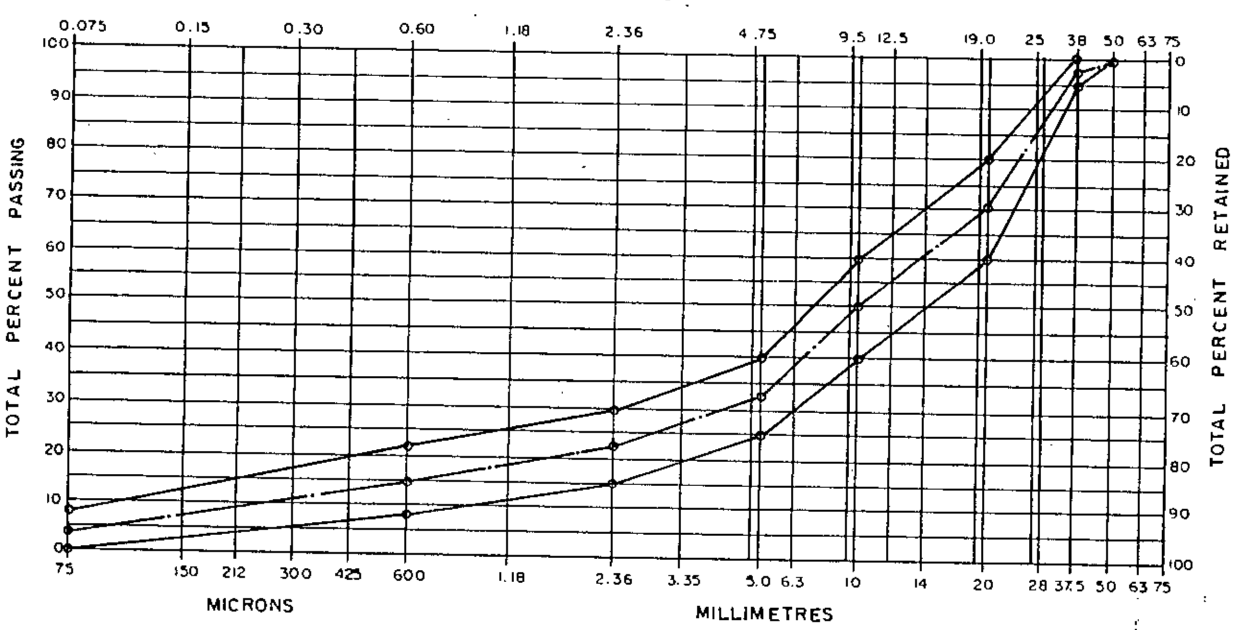


FIG. 4.7 GRADATION X_2 (BSI, BASE)

AGGREGATE GRADATION CHART

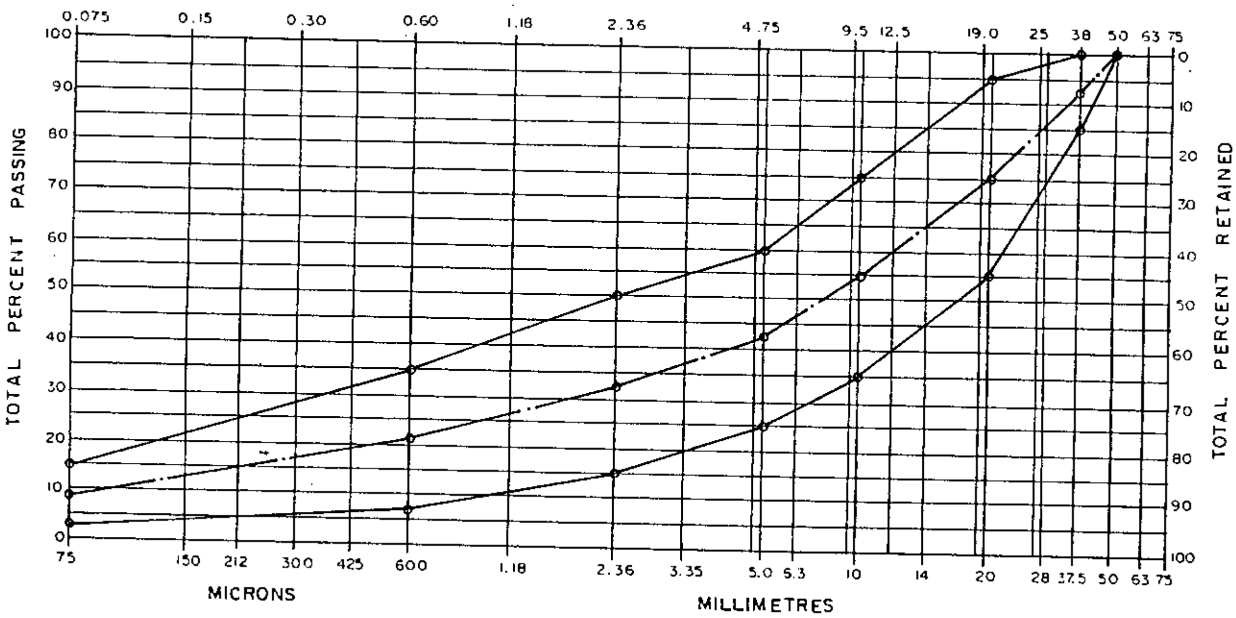


FIG. 4.8 GRADATION X_3 (BRRL. SUB-BASE. TYPE – A)

AGGREGATE GRADATION CHART

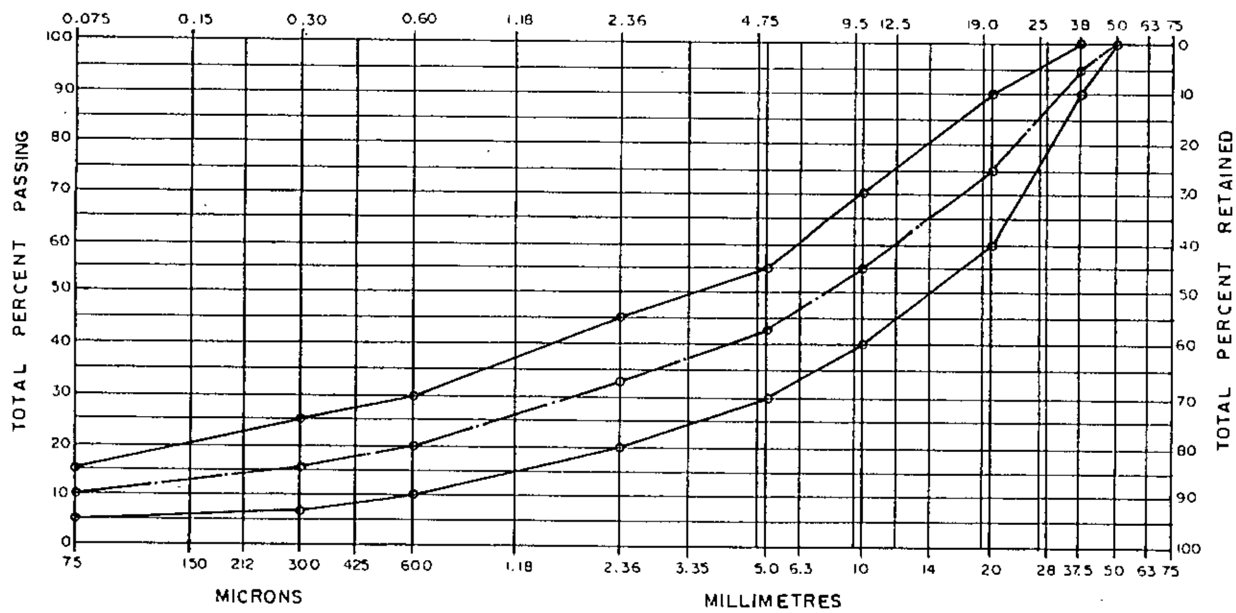


FIG. 4.9 GRADATION X_4 (BRRL, BASE, TYPE – A)

content-density graphs are plotted in Figs. 4.10 to 4.11. MBR tests were performed for the mixtures of gradation X_1, X_2, X_3 and X_4 compacting the samples at respective optimum moisture contents.

4.5 EFFECT OF GRADATION ON MBR/CBR VALUES

Picked jhama brick aggregates with sand and soil at different gradations were compacted in the laboratory to find the CBR value of the compacted mixture. Since CBR test is not satisfactory for aggregates larger than 20 mm (3/4 in.), Modified Bearing Ratio (MBR) test was performed on the aggregates in the laboratory. CBR value was then determined by using the simple linear relationship between MBR and CBR i.e. MBR = 0.33 CBR. Samples were prepared in accordance with the selected gradations by compacting the aggregate mixtures with a BS vibrating hammer. The mixes are compacted in three layers and a compaction time 12 minutes per layer was used. The moisture content of the mixes were maintained at the respective optimum values. The compacted specimens were subjected to four days soaking period applying 72 lbs surcharge and providing swell measuring arrangements. Swell measured after the soaking period was negligible. Soaked specimens after 15 minutes of free drainage were tested in the testing machine (Figs. B-7 to B-10 in Appendix-B). The load-penetration relations are shown in Table A-3 in Appendix-A and MBR/CBR-density-gradation relationship is shown in Table 4.4. After plotting the data in Figs. 4.12 to 4.15 corrected soaked

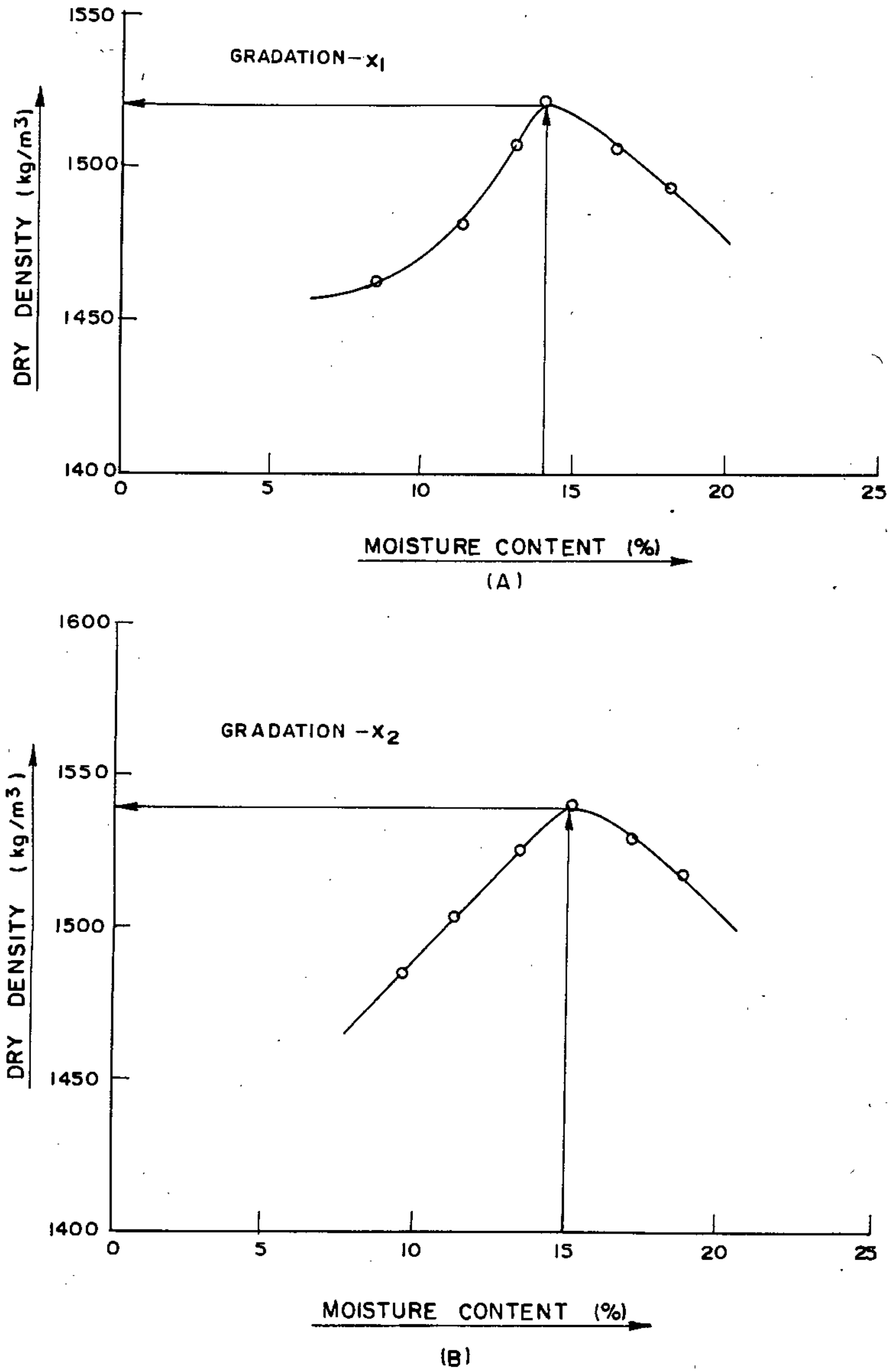
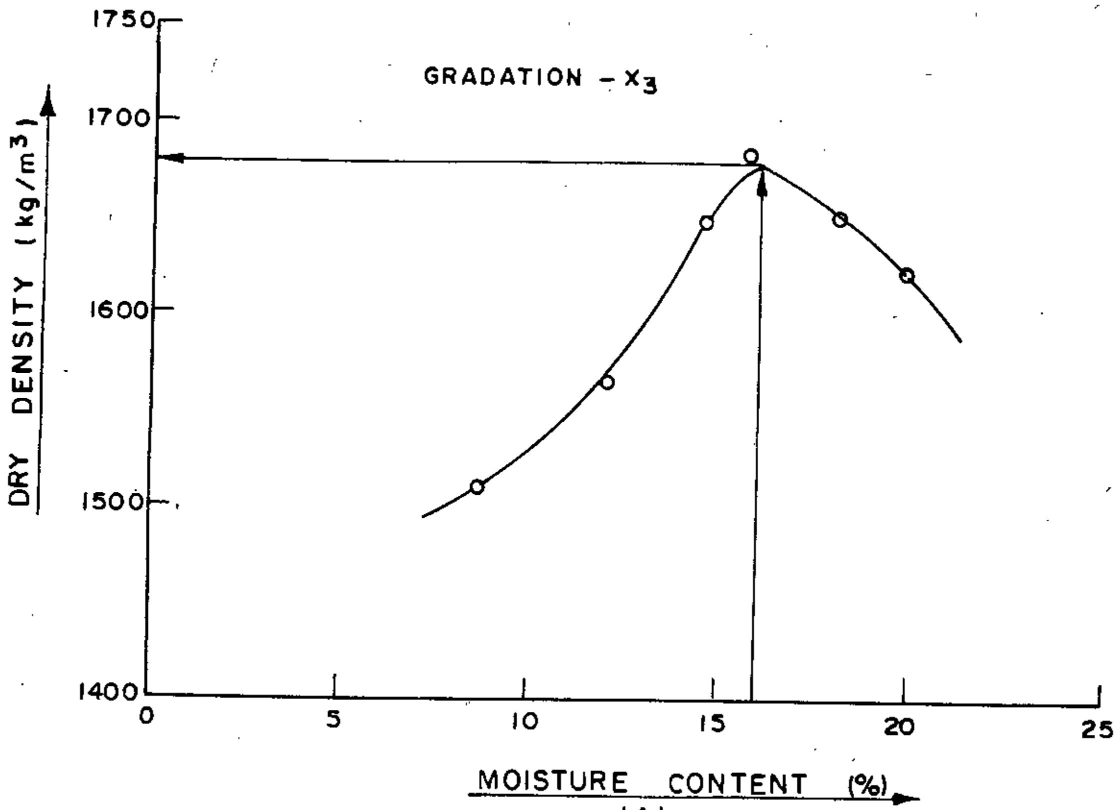
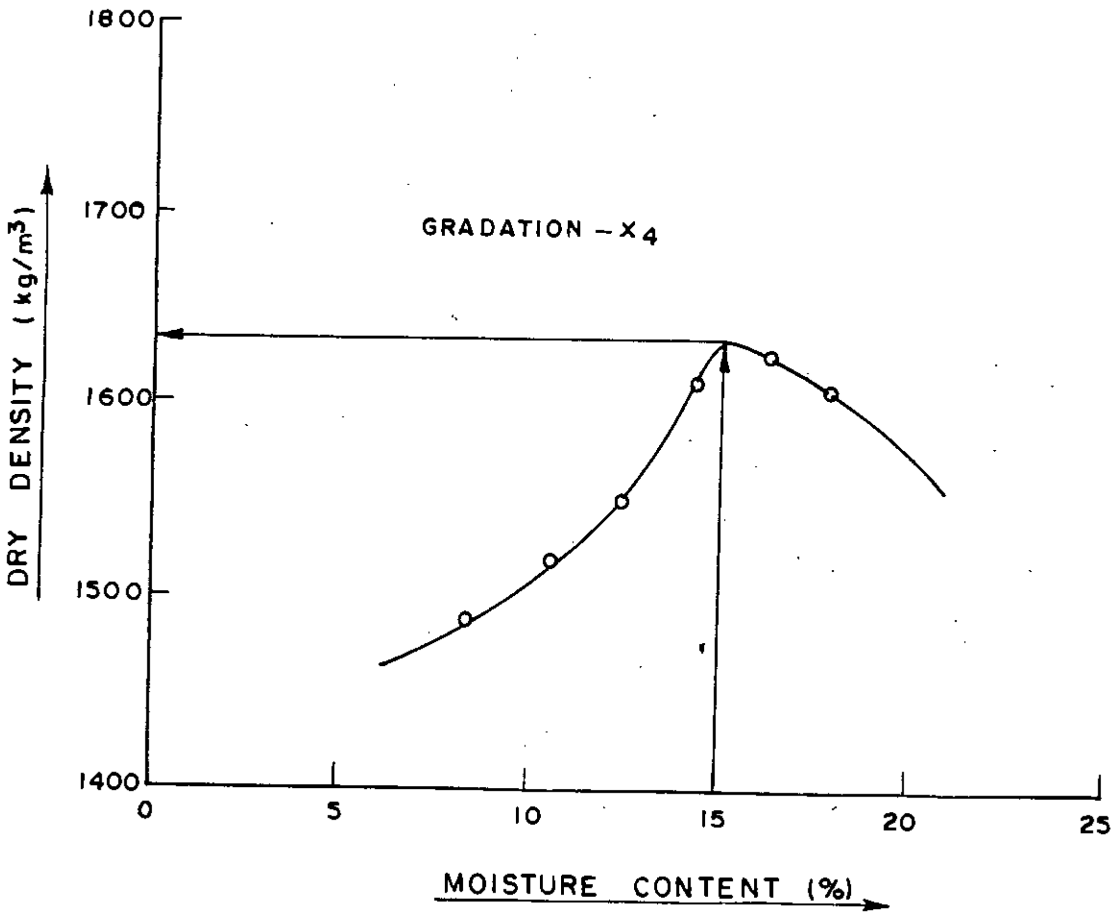


FIG. 4.10 MOISTURE DENSITY RELATIONSHIP: (A) FOR GRADATION – X_1
(B) FOR GRADATION – X_2



(A)



(B)

FIG. 4.11 MOISTURE DENSITY RELATIONSHIP:
(A) FOR GRADATION - X_3 (B) FOR GRADATION - X_4

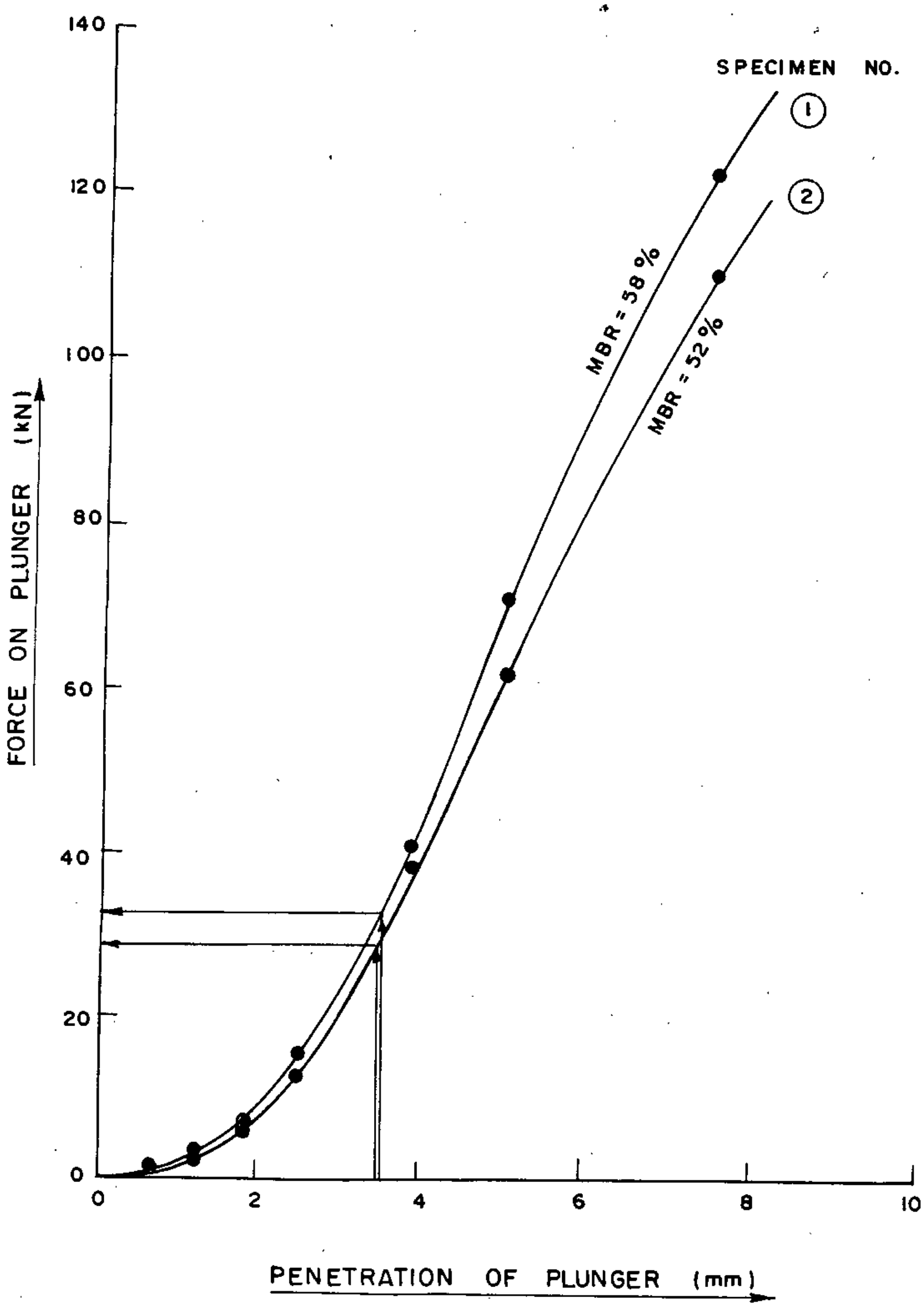


FIG. 4.12 LOAD—PENETRATION CURVES FOR X_1 GRADATION.

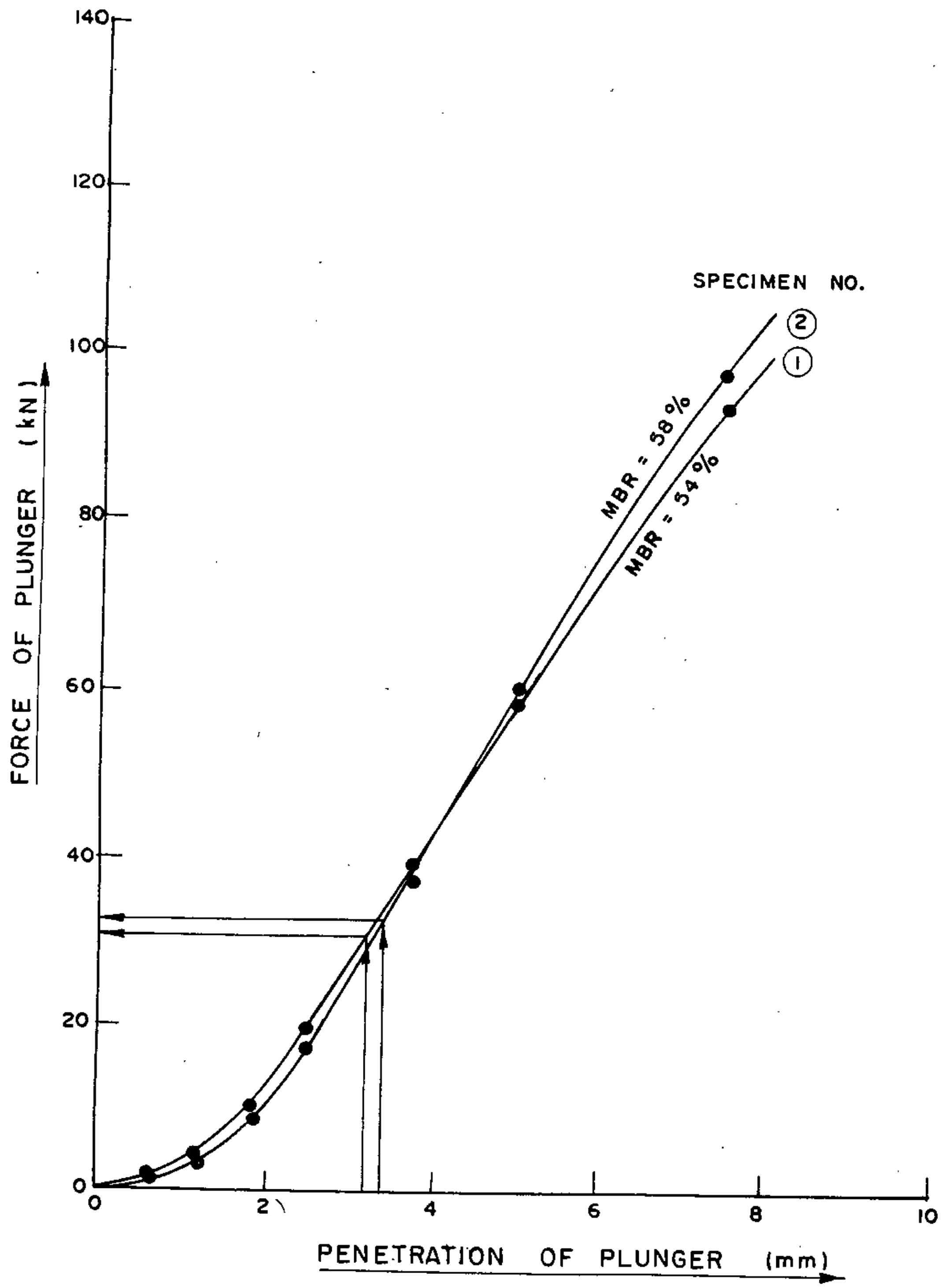


FIG. 4.13 LOAD - PENETRATION CURVES FOR X_2 GRADATION.

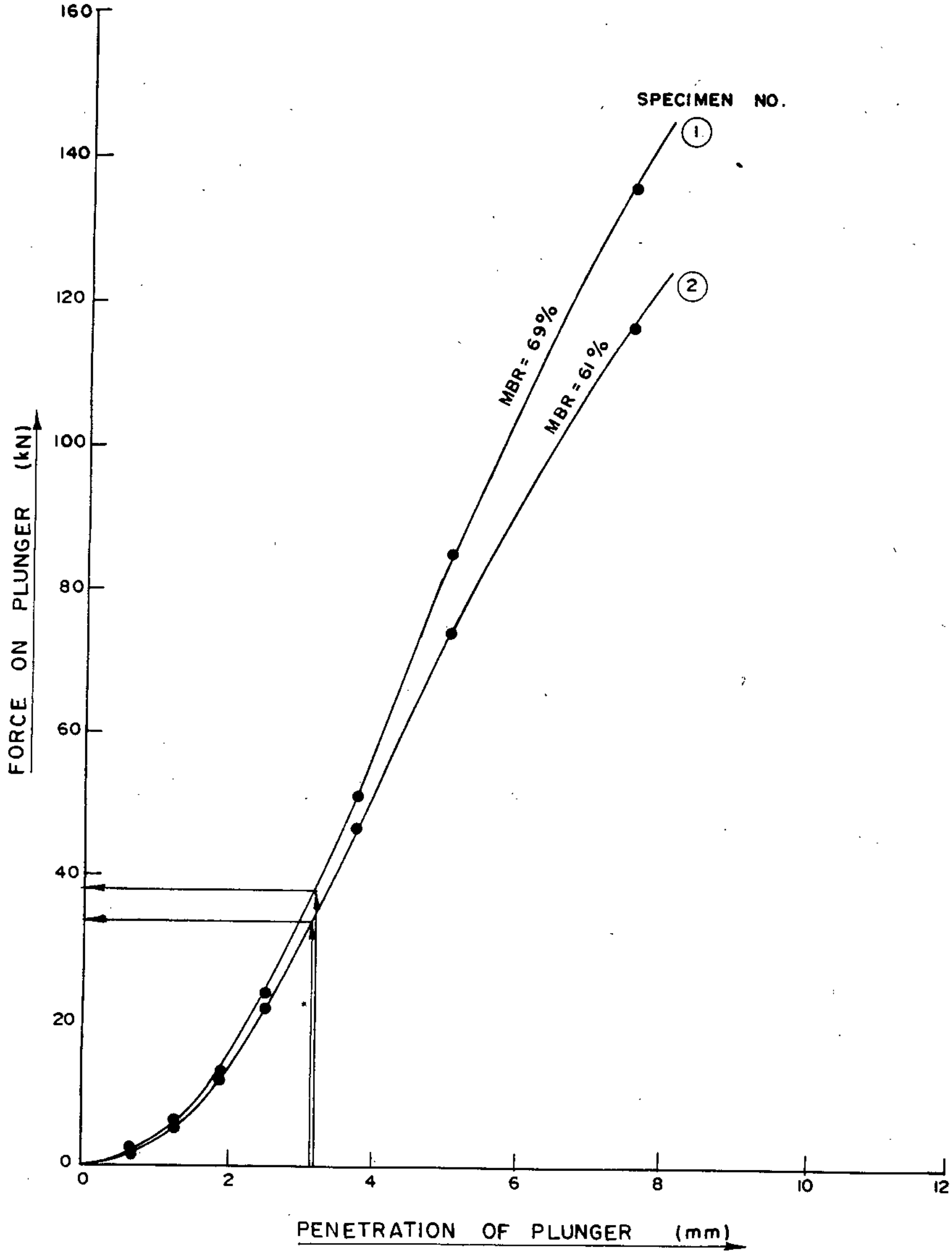


FIG. 4.14 LOAD - PENETRATION CURVES FOR X_3 GRADATION.

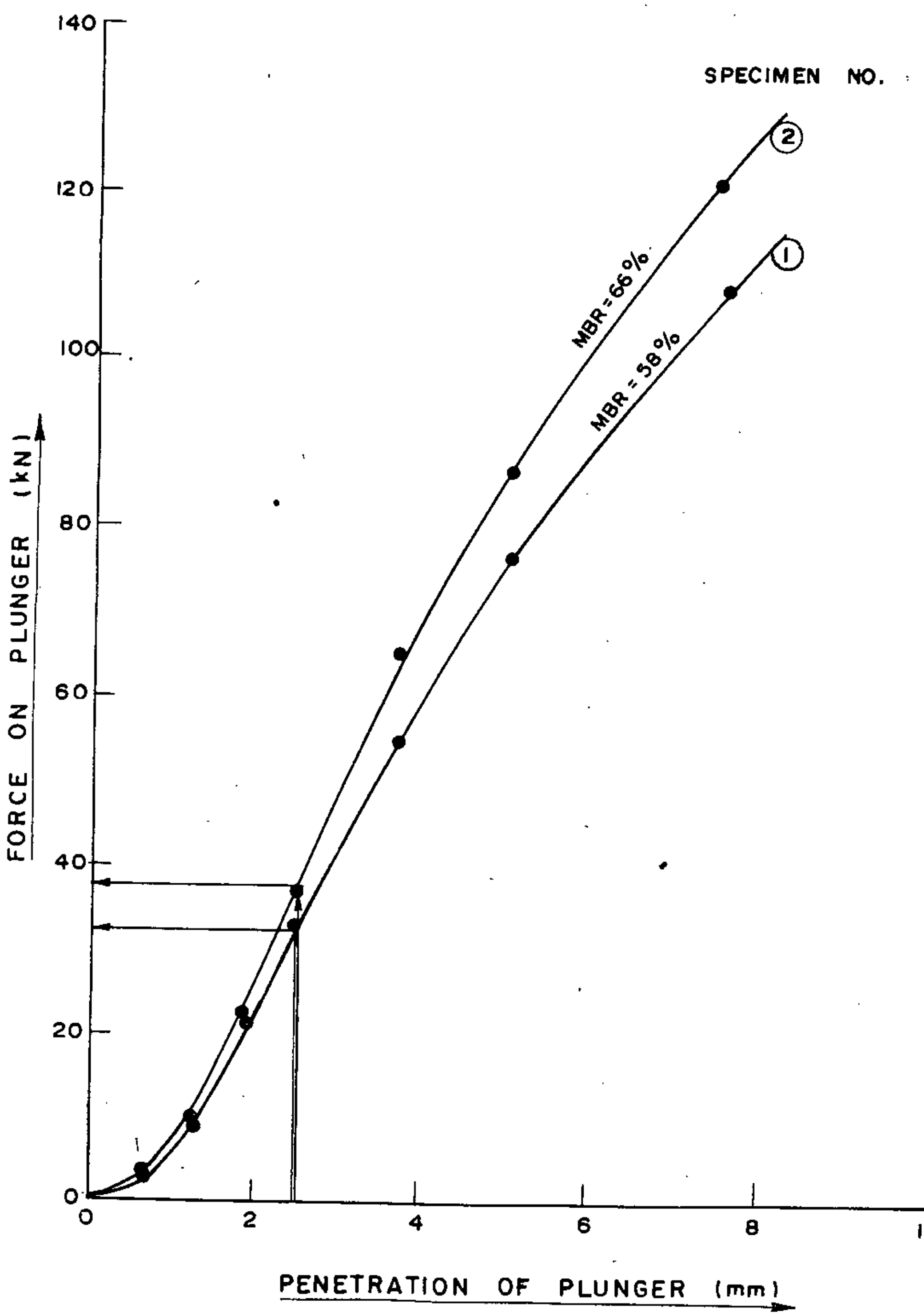


FIG. 4.15 LOAD — PENETRATION CURVES FOR X_4 GRADATION.

MBR values were obtained. For all the samples, MBR values were taken for 2.54 mm (0.1 inch) penetration and then CBR values were determined by using the relationship, MBR = 0.33 CBR.

Table 4.4 MBR/CBR - Density - Gradation Relationship

Gradation	Optimum moisture content (%)	Maximum dry-density (kg/m^3)	Average corrected MBR in (%)	CBR in % from MBR=0.33 CBR
X_1	14.0	1520	55	167
X_2	15.0	1540	56	170
X_3	16.0	1680	65	197
X_4	15.0	1634	62	188

4.6 PLASTICITY CHARACTERISTICS OF FINES

Plasticity characteristics of the soil used in this research was determined. The liquid limit and plastic limit tests were performed in accordance with ASTM/ANSI D423-66 (Reapproved in

1972) and ASTM/ANSI 424-59 (Reapproved in 1971) respectively. The soil passing 420 µm (NO.40) sieve was taken for these determinations. Results are shown in Appendix-A (Table A-4).

A hydrometer analysis of the soil passing 75 µm (NO.200) sieve was done according to the procedure described by LAMBE (24). The results of this analysis is shown in Table 4.5 and in Table A-5 in Appendix-A.

Table 4.5 Results of Hydrometer Analysis of Fines (Soil)

Dia. D (in mm)	0.05	0.0361	0.0263	0.0195	0.0143	0.0109	0.0081	0.0059
Percent Finer	71.66	60.18	48.15	33.03	24.91	18.19	14.28	12.60

CHAPTER-FIVE

ANALYSIS AND DISCUSSION OF TEST RESULTS

For a dense mixture, the general requirements for coarse and fine aggregates are that, "aggregates should be hard, tough, durable and free from excess amount of flat and elongated pieces and vegetable particles or other organic compounds". From the test results of coarse and fine aggregates summarized in Table 3.1 to 3.3 it can be said that crushed picked jhama brick aggregates satisfy the general criteria of the aggregates.

The strength characteristics of coarse aggregate required for WBM construction according to the specifications given by various Highway Agencies were discussed in Article 2.2. These characteristics of the experimental aggregates were shown in Table 3.2 and reveals that the aggregates used for the study satisfy the requirements of IRC, AASHTO, ASTM and BRRL (sub-base) specifications. The experimental aggregates are seen to be marginally below the requirements for BRRL base courses.

5.1 DEGRADATION

Degradation, due to compaction of sample, is very much dependent upon the type of compactive effort applied and the type of aggregate used. Soft aggregate like brick aggregate suffers a higher degradation due to lower strength. So during selection of compaction equipment for compacting such type of aggregate care must be taken. A degradation study (discussed in Article 4.1) was carried out on picked jhama brick aggregate, sand and soil mixture according to BRRL (sub-base, Type-B) grading specification. Grain size distribution curve for initial and degraded aggregates are shown in Fig. 4.1 and their gradations are contained in Table 4.1. From Table 4.1, it is seen that due to degradation increase in percent passing 75 µm sieve are 4%, 5% and 3.5% and from Fig. 4.1, D_{50} ratios* are 0.75, 0.63 and 0.82 for 2.5 kg, 4.5 kg and vibrating hammer respectively. So for vibrating hammer, percent passing 75 µm sieve is less and D_{50} ratio is high. Fig. 4.1 also shows that after degradation grain size distribution curve for BS vibrating hammer is closer to initial grain size distribution curve. Therefore, degradation is less for BS vibrating hammer and field compaction for aggregates should be done by using vibrating compaction method.

5.2 PLACING OF SCREENINGS

Screenings may be used either separately on the top surface of coarse aggregate or by mixing with coarse aggregate. The strength and stability of a WBM mix is markedly influenced by the placing of screenings. A study for selecting the best way of using screenings (discussed in Article 4.3) was carried out for

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* "D_{50} ratio" is the ratio of D_{50} after degradation to the D_{50} before degradation.

two types of aggregate grading IRC (Type-3) and AASHTO (Type-3), where screenings were 28% and 25% of the total aggregate mixture respectively. In this study two types of sample were prepared for each grading and CBR tests were then performed on each type of sample. Test results were tabulated in Table A-1 and Table A-2 and load-penetration plots were shown in Fig. 4.4 (a,b) and Fig. 4.5(a,b) respectively. From Table A-1 and Table A-2, it is seen that for each case, CBR value is higher when screenings are mixed with coarse aggregate. Because when screenings are provided on the top surface of coarse aggregate small voids are present within the coarse aggregate; but when screenings are mixed with coarse aggregate, they fill the voids within the coarse aggregate. Therefore, the best way of placing screenings is by mixing them with coarse aggregate before WBM construction.

5.3 GRADATION

Four gradation X_1, X_2, X_3 and X_4 shown in Table 4.3 were selected with a view to obtain the densest mixture giving maximum stability. Picked jhama brick aggregates and sand-soil mixtures were used for obtaining these gradations. The MBR curves for all these gradations are presented in Figs. 4.12 to 4.15. These curves are similar in nature indicating increasing resistance for higher values of penetrations. Moisture-density graphs shown in Fig. 4.10 and Fig. 4.11 are identical in shape. Maximum dry density varies from 1520 kg/m^3 for X_1 gradation to 1680 kg/m^3 for X_3 gradation. A close study of these results reveals that the

MBR/CBR value increases as the density of the mixture increases and at maximum density there is maximum MBR/CBR value (shown in Table 4.4). The maximum MBR/CBR value is obtained for gradation X_3 which is shown in Table 5.1. The specified gradation X_3 employs 67.0 percent coarse aggregates, 24.0 percent fine aggregates and 9.0 percent fines in the aggregate mixture.

Table 5.1 Gradation X_3

Sieve size	Percent passing by weight
50 mm	100
38 mm	85-100
20 mm	55-95
10 mm	35-75
5 mm	25-60
2.4 mm	15-50
0.600 mm	7-35
0.075 mm	3-15

5.4 MOISTURE CONTENT

The moisture content-dry density relationships for four mixtures shown in Figs. 4.10 to 4.11 indicate that the density increases with an increase in the moisture content. After a certain percentage of moisture in the mixture, the dry density decreases with the increase of moisture. The moisture contents for maximum densities for these mixtures varies from 14.0 percent to 16.0 percent shown in Table 4.3.

For picked jhama brick aggregates, it is said in Article 5.1 that maximum CBR values are obtained at optimum moisture contents. In other words it can be said that the maximum CBR value is obtained when the mixture is compacted at optimum moisture content. From the test results it is seen that for gradation X_3, the mixture with which maximum CBR value is obtained has an optimum moisture content of 16.0 percent of the dry weight of the mixture. The maximum dry density for this mixture at this moisture content is 1680 kg/m³.

5.5 PLASTICITY CHARACTERISTICS

With crushed brick aggregates some sand and soil were mixed to satisfy the designed gradation. Normally the fines produced during hand crushing of bricks are non-plastic. The amount thus obtained is very small in quantity. To have specified percentage of fines in the mixture silt and clay fraction of soil (material passing 75 µm sieve) is used in the Mixture. The combined mixture may sometimes be plastic (PI greater than 6) if there is excess clay particles. Hence plasticity tests were performed according to standard procedure with the portion of the soil material passing 420 µm (NO.40) sieve.

The plasticity tests described in Article 4.6 shows that the material used in this investigation fulfil the requirements for WBM mixes.

CHAPTER-SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

On the basis of experimental results of this investigation the following conclusions are drawn:

1. Aggregates prepared by crushing uniformly virtrified and slightly over-burnt bricks, locally known as "Picked Jhama" bricks are suitable for WBM construction from the considerations of aggregate properties.

2. Modified Bearing Ratio (MBR) test is better than CBR test for the correct assessment of WBM construction materials as CBR test in most cases is not applicable for WBM aggregate.

3. For WBM construction, it is better to use screenings by mixing them with coarse aggregates rather than by placing separately.

4. Gradation of the aggregate mixture is an important factor determining the load carrying capacity of WBM (base or sub-base) courses.

5. The stability of aggregate gradation X_3, which is taken from BRRL specification, is satisfactory from the consideration of MBR test results. The gradation of the crushed picked jhama brick aggregates, sand and soil mixture shown in Table 5.1 gives the maximum MBR/CBR value and is suggested to be used in WBM (base or sub-base) courses.

6.2 RECOMMENDATIONS

The observations of this research are limited in their scope, within the range of variables investigated, the type of test employed and the nature and number of specimens tested. Only one type of aggregate was used throughout the laboratory investigations. The compaction energy was same for all the specimens. Hence further research is necessary for the comprehensive assessment of WBM construction.

6.2.1 Recommendations for Future Study

i) In the present study four gradations were used for investigation. More gradations could be taken to find the MBR/CBR values. In future more close investigation should be taken to find a suitable gradation at which maximum MBR/CBR value would be obtained.

ii) In this research, CBR values were found by using the simple linear correlation between MBR and CBR i.e. **MBR = 0.33CBR** reported by Zakaria (14). In future, a better correlation between MBR and CBR can be developed by using these gradations and more MBR/CBR values.

iii) In the present study the CBR values of all mixtures were determined in the laboratory. The same materials should be compacted in the field at the same densities and field CBR values should be determined. Correlations should be developed between laboratory and field CBR values.

6.2.2 Recommendations for Field Constructions

i) For the construction of WBM base or sub-base courses for high type pavements, picked jhama bricks are recommended to be used.

ii) The mixture is to be blended as in gradation X_3 (Table 5.1) found in the study.

iii) The gradation of the mixture should be strictly maintained during construction in the field.

iv) During compaction, optimum moisture should be maintained in the mixture and water should be added several hours before compaction for thorough distribution of moisture.

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APPENDIX-A

TABLES OF TEST RESULTS

Table A-1 CBR Test Results for Picked Jhama Brick Aggregate Mix

Gradation - IRC (Type-3)
Coarse Aggregate = 72%
Screenings = 28%
Optimum moisture content (omc) = 12%

Condition of sample preparation	Specimen no.	Penetration load in kN for penetrations in mm							Dry density kg/m³	Average dry density kg/m³	Corrected CBR in %	Average corrected CBR in %
		0.625 (0.025")	1.25 (0.05")	1.875 (0.075")	2.50 (0.10")	3.75 (0.15")	5.00 (0.20")	7.50 (0.30")				
Screenings are mixed with coarse aggregate	1	2.99	6.62	10.21	12.93	16.06	19.05	23.32	1576		98	
	2	2.81	6.35	9.71	12.16	14.79	17.42	21.77	1566	1561	94	95
	3	2.63	5.62	8.62	11.52	14.24	16.24	20.68	1540		92	
Screenings are provided on the top surface	4	2.27	4.35	6.99	8.98	11.16	12.25	14.29	1454		70	
	5	2.72	4.76	7.89	10.61	12.70	14.29	16.15	1472	1471	83	81
	6	2.90	5.72	8.53	11.52	13.88	15.42	16.96	1486		90	

Table A-2 CBR Test Results for Picked Jhama Brick Aggregate Mix

Gradation - AASHTO (Type-3)
Coarse Aggregate = 75%
Screenings = 25%
Optimum moisture content (omc) = 11%

Condition of sample preparation	Specimen no.	Penetration load in kN for penetrations in mm							Dry density kg/m³	Average dry density kg/m³	Corrected CBR in %	Average corrected CBR in %
		0.625 (0.025")	1.25 (0.05")	1.875 (0.075")	2.50 (0.10")	3.75 (0.15")	5.00 (0.20")	7.50 (0.30")				
Screenings are mixed with coarse aggregate	1	4.08	7.71	11.11	14.06	17.58	19.50	22.23	1548	1561	105	105
	2	3.63	7.26	10.43	13.38	16.33	18.82	20.87	1546		98	
	3	4.54	8.39	11.79	15.42	18.60	20.87	23.89	1590		113	
Screenings are provided on the top surface	4	2.27	5.44	7.71	9.07	11.11	12.70	14.74	1456	1455	72	68
	5	2.27	4.54	7.26	8.62	10.21	11.34	12.93	1457		69	
	6	2.72	4.99	7.14	8.28	9.53	10.66	12.70	1452		63	

Table A-3 MBR Test Results for Picked Jhama Brick Aggregate Mixes

Gradation	Specimen no.	Penetration load in kN for penetrations in mm							Corrected MBR in %	Average corrected MBR in %	CBR in % from MBR=0.33 CBR
		0.625 (0.025")	1.25 (0.05")	1.875 (0.075")	2.50 (0.10")	3.75 (0.15")	5.00 (0.20")	7.50 (0.30")			
X_1	1	1.59	3.63	6.80	15.42	42.64	70.76	121.56	58	55	167
	2	1.13	2.72	6.35	12.70	38.10	61.69	109.77	52		
X_2	1	1.36	3.63	9.98	19.50	39.01	58.06	93.44	54	56	170
	2	1.13	2.72	8.16	17.69	37.20	59.87	97.07	58		
X_3	1	2.27	5.90	12.70	24.04	53.52	85.28	136.08	69	65	197
	2	1.81	4.98	11.79	22.23	47.17	73.94	116.58	61		
X_4	1	2.49	8.16	21.09	32.66	54.43	79.83	107.96	58	62	188
	2	2.72	9.07	22.23	36.74	64.41	86.18	120.66	66		

Table A-4 Liquid Limit Test Result

No. of observation	1	2	3	4	5
No. of Blows	12	16	24	30	42
Water content (%)	31.2	28.4	25.6	23.3	20.8

The soil is non-plastic as no plastic limit was observed.

Table A-5 Hydrometer Analysis

Soil Sample: Soil material passing NO.200 (75 µm) sieve.
Weight of dry soil = 50 gms. Specific gravity of soil = 2.78
N′ = % finer = 90% = 0.90. Temperature = 30°C.
Hydrometer no. - 12A56650

Date	Elapsed time in minutes	R = 1000(r-1)	R_w = 1000(r_w-1)	R-R_w	Z_r in cm	D in mm	N in %	N′ in %
3.4.91	1/4	26.7	-0.5	27.2	8.9	0.0699	84.6	76.14
	1/2	25.1	"	25.6	9.1	0.050	79.62	71.66
	1	21.0	"	21.5	9.5	0.0361	66.87	60.18
	2	16.7	"	17.2	10.1	0.0263	53.49	48.15
	4	11.3	"	11.8	11.1	0.0195	36.70	33.03
	8	8.4	"	8.9	12.0	0.0143	27.68	24.91
	15	6.0	"	6.5	13.1	0.0109	20.22	18.19
	30	4.6	"	5.1	14.2	0.0081	15.86	14.28
	60	4.0	"	4.5	15.3	0.0059	13.99	12.60
	120	1.5	"	2.0	16.7	0.0044	6.22	5.60
	240	1.2	"	1.7	17.9	0.0032	5.29	4.76

APPENDIX-B

FIGURES OF TEST EQUIPMENTS

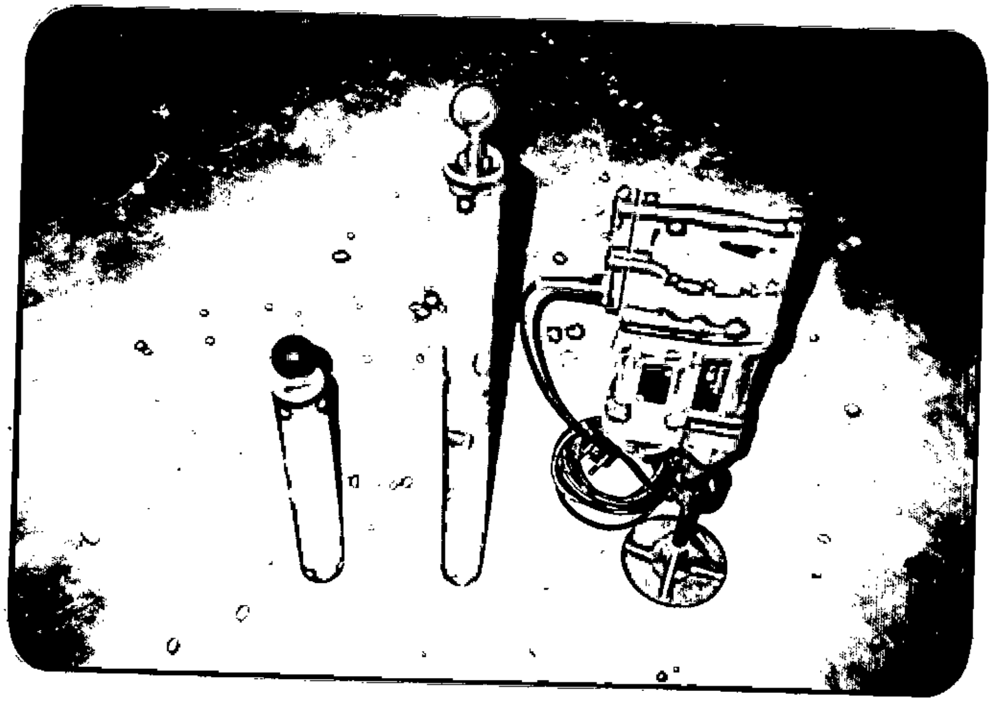

FIG. B-1 DIFFERENT COMPACTION EQUIPMENTS USED IN DEGRADATION STUDY

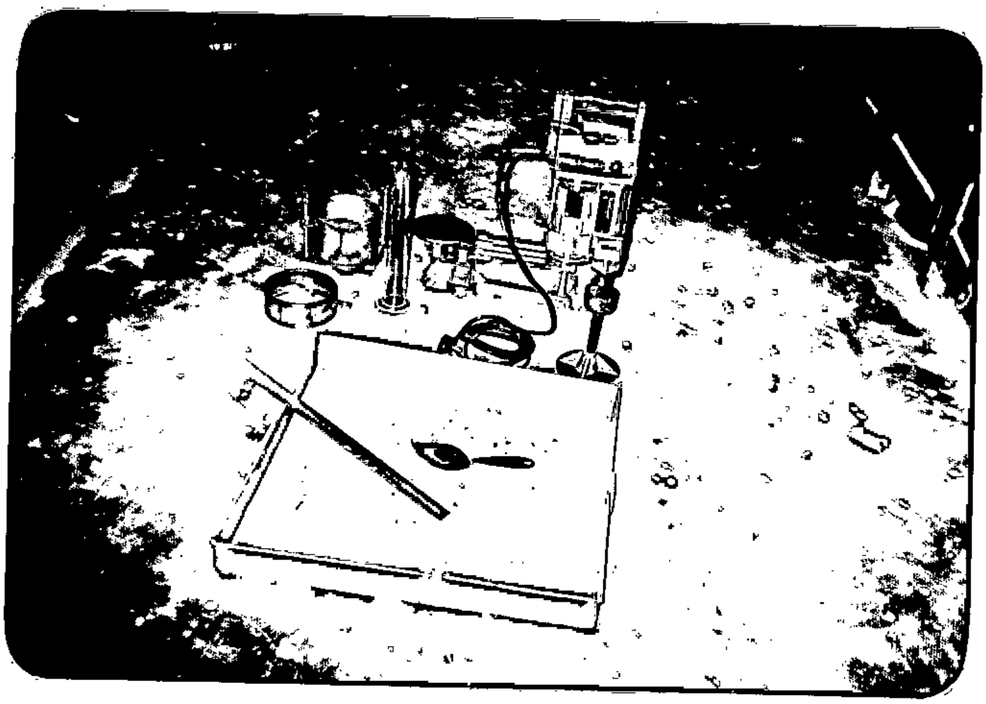

FIG. B-2 DRY DENSITY/MOISTURE CONTENT DETERMINATION EQUIPMENTS

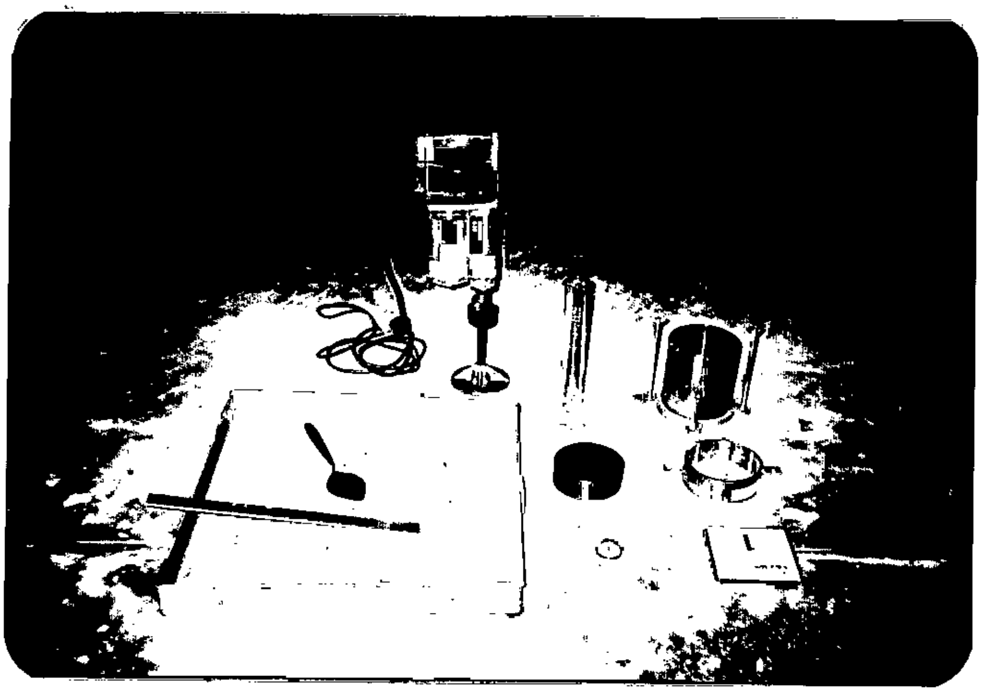

FIG. B-3 EQUIPMENTS FOR THE COMPACTION OF CBR MOULD

FIG. B-4 THE MIXTURE IS READY FOR COMPACTION

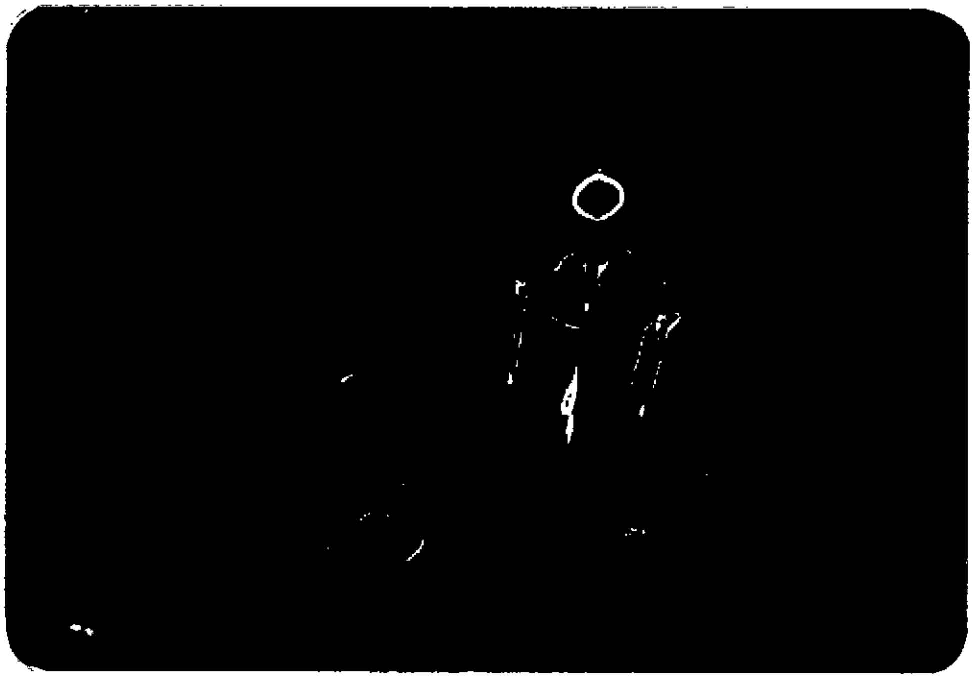

FIG. B-5 COMPACTED CBR MOULD IS READY FOR SUBMERSION IN WATER

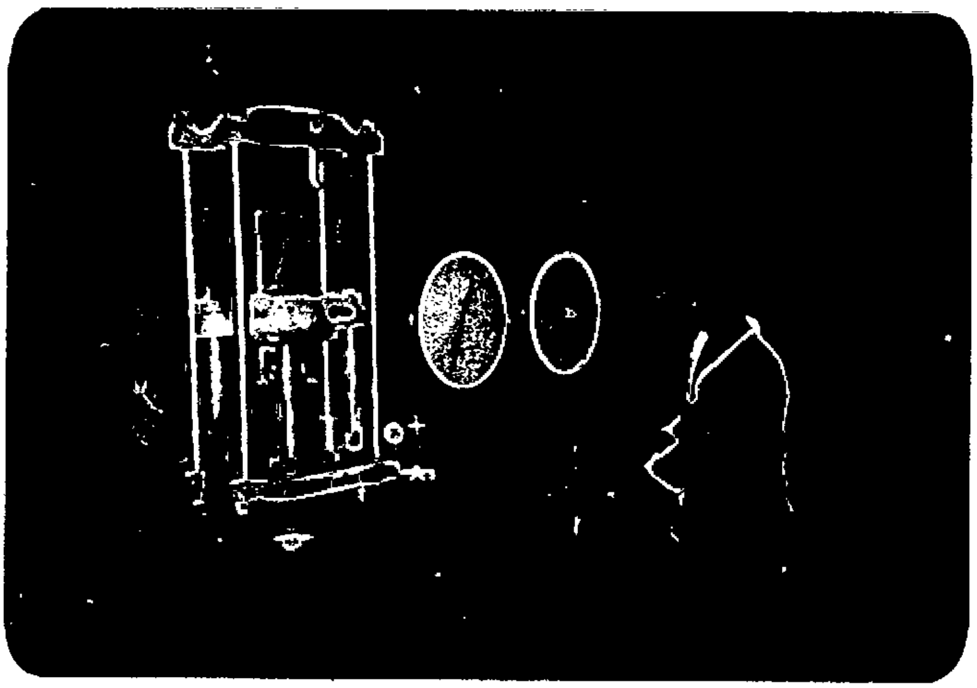

FIG. B-6 TESTING OF CBR SAMPLE IS GOING ON IN A UNIVERSAL TESTING MACHINE

FIG. B-7 EQUIPMENTS FOR THE COMPACTION OF MBR MOULD

FIG. B-8 COMPACTION OF MBR SAMPLE IS GOING ON BY A BS·VIBRATING HAMMER

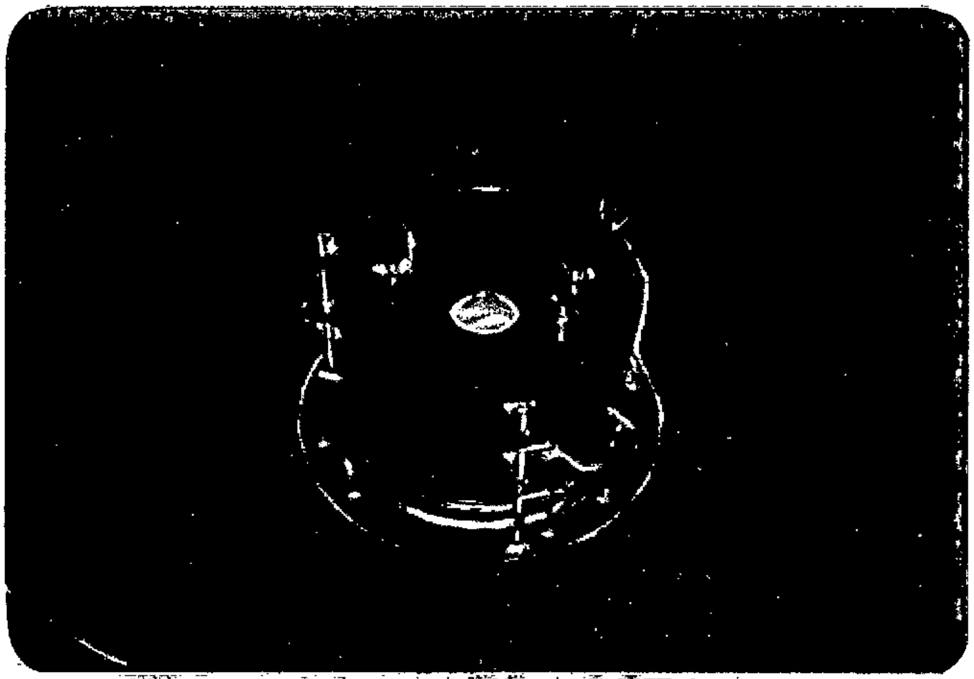

FIG. B-9 COMPACTED MBR MOULD IS READY FOR SUBMERSION IN WATER

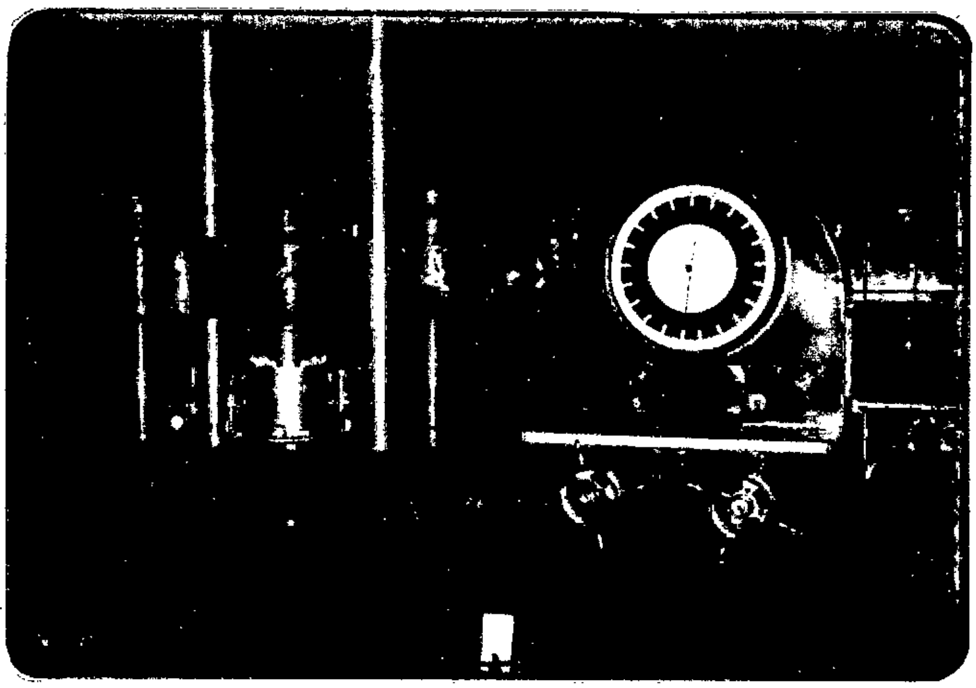

FIG. B-10 TESTING OF MBR SAMPLE IS GOING ON