CHAPTER - 7

GEOTECHNICAL APPRAISAL

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INTRODUCTION

The western Rajasthan in general and along the international border in particular lack in good road frequency and networking, a feature not supposed to be for the strategically sensitive area. The reasons that have been ascribed for such a low road frequency and poor networking are :

(I) Vast tract and thick pile of aeolian sediments that mask the source material of the traditional paving aggregates (hard rocks) to patches and isolated hillocks; result into an acute scarcity of conventional paving aggregate, thereby increase the cost of road construction manifold (Plate 7.1)

(II) Erratic vertical pavement alignment aided by frequent dune migration that completely conceal the roads frequently.

(III) High cost of road maintenance due to frequent failures caused by sandy subgrade Plate 7 2 (A & B)

Thus, cost effective methods of road construction and maintenance are, the need of the hour. Keeping in mind the present requirements, the author endeavoured to understand the efficacy of calcretes and ferricretes, two widely distributed near surface occurring duricrusts as an alternate, cost effective pavement aggregate for roads in the study area.

Plate 7 1

Panoramic view of border roads in an active dunal tract Loc Raniya

Plate 7.1

Plate 7.2

(A) A view of road alignment. Observe the failures (pot holes and headward gullies) developed due to poorly stabilized embankments.Loc. Nachana.

(B) A close view of headward gulley depicting poor compaction and less thickness of pavement courses. Loc. Nachana.

A

B

Plate 7.2

STRUCTURAL ELEMENTS OF ROAD PAVEMENTS

Since, the natural earthern tract cannot withstand the load exerted by the modern days transportation systems, an artificial, hard, stable crust ie.'*the pavement*' need to be constructed over the natural soil with a sole purpose of distributing the transport load as well as to provide adequate wearing surface. Pavements are generally made in several layers as illustrated in Figure 7.2. Each layer has a special function in supporting the load In general a pavement consists of three components viz sub-base, base, and wearing surface; one upon the other and laid over the natural soil called as subgrade.

The purpose of wearing course is to provide a smooth riding surface that is resilient and to resist the pressure exerted by the wheel load The base course is a layer of granular or coarse paving material that are laid immediately below the wearing course. A sub-base is another layer of earthern material sandwiched between the subgrade and base course Base and sub-base courses are laid mainly to distribute the load over a finite thickness of pavement and also to increase the load bearing capacity of the pavements. Though both base and sub-base courses need to have high strength characteristics, the base course need to be more stronger as it lies immediately below the wearing surface

The subgrade is the foundation layer, a layer which is eventually going to support all the loads which is going to be exerted on the pavement. Hence, the subgrade though not directly come in contact with the wheel loads, is instrumental in deciding the performance of a pavement

The salient parameters that govern the performance of a subgrade are strength, drainage, ease of compaction, and permanency compaction (Giddigasu, 1983, Giddigasu et al., 1987, Khanna and Justo, 1991). Embankment is also a pavement layer that is constructed above the existing ground level so as -

(i) To keep the subgrade above the groundwater table

(ii) To prevent damage to the pavement by capillary and surface water.

(iii) To maintain vertical alignment.

APPROACH

The evaluation of calcretes and ferricretes and their efficacy for different road pavement courses are estimated as per the standard methods (IS, 1978) used for the traditional paving aggregates viz.

(i) Collection of undisturbed / disturbed samples using core cutter and sand replacement methods at different locations (Figure 7 1) in accordance to the Earth Manual (1965)

(ii) Estimation of size gradation, Liquid Limit (LL), Plastic Limit (PL), Plasticity Index (PI) etc. in accordance with IS 2720 Part V

(iii) Proctor compaction test to identify Maximum Dry Density ($\gamma_{d\ max.}$) and the corresponding moisture content, Optimum Moisture Content (OMC) as per the methodology of IS: 2720, Part VII.

(iv) California Bearing Ratio (CBR) test to find the strength of the soils as an empirical entity Soils compacted at OMC and rd max are subjected to CBR test in tune to IS :2720 Part XVI.

(v) The Aggregate Impact Values (AIV) to evaluate the resistance of the aggregate materials to impacts as an expression of toughness.

(vi) North Dakota Cone Penetration Test, a test akin to CBR to find the bearing strength of aggregates

TYPES OF PAVEMENT

Based on the support and distribution of loads within the pavement, pavements are broadly grouped in to two categories viz. flexible pavements and rigid pavements (Khanna and Justo, 1991).

FLEXIBLE PAVEMENTS

The principle behind the flexible pavements is, the load exerted on the wearing surface gets dissipated by carrying it deep to the ground through successive layers of granular

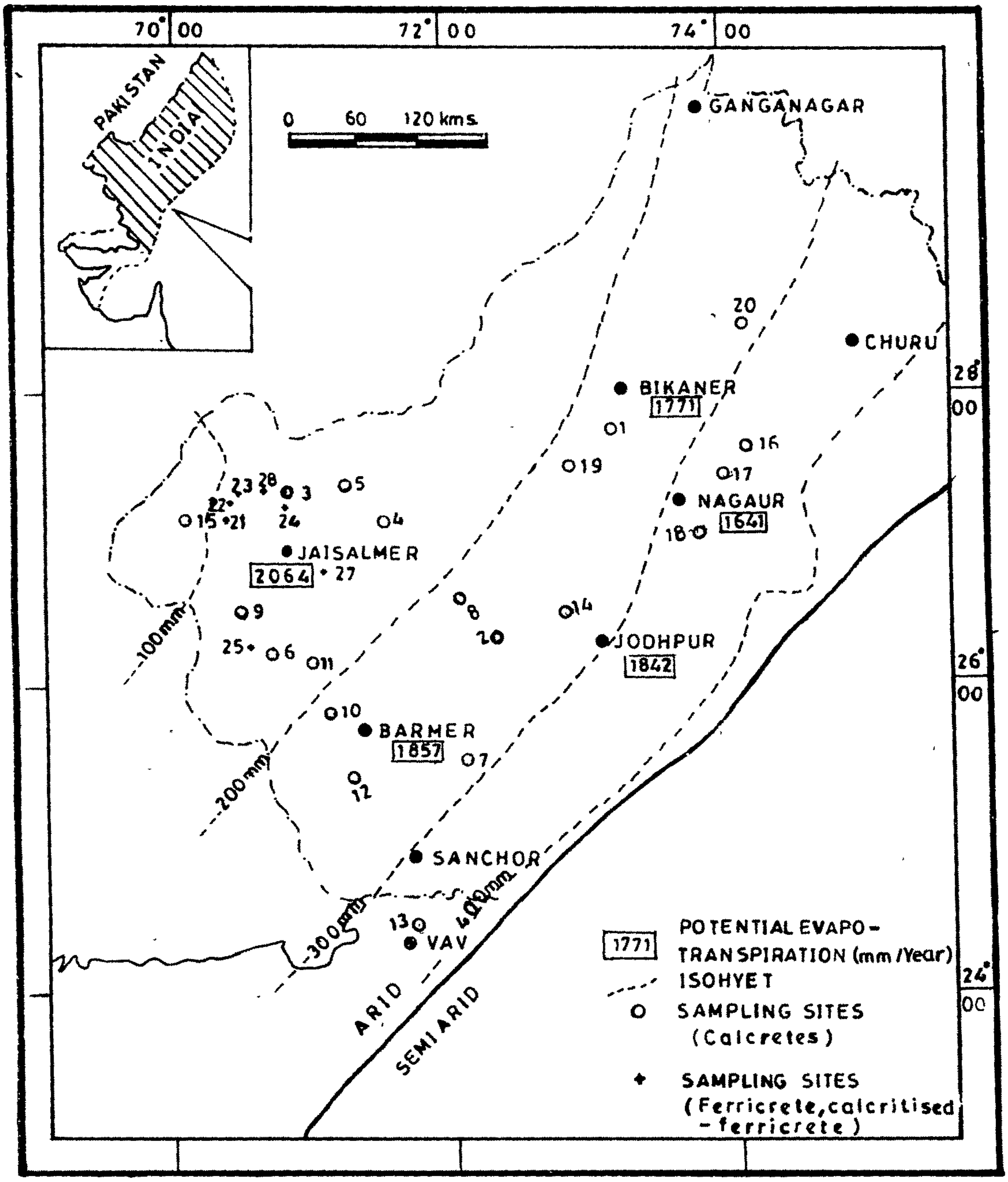
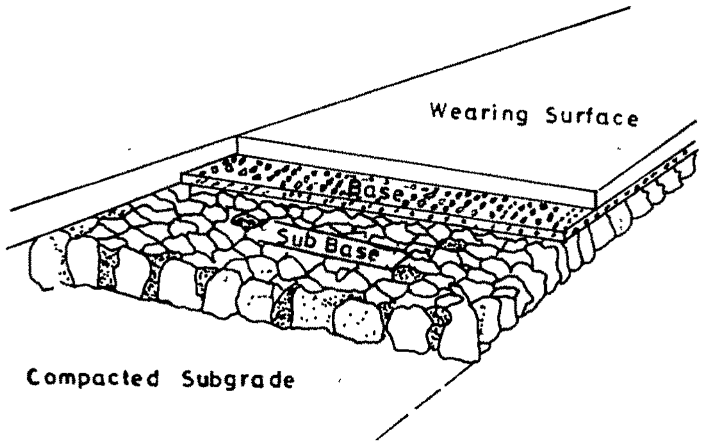
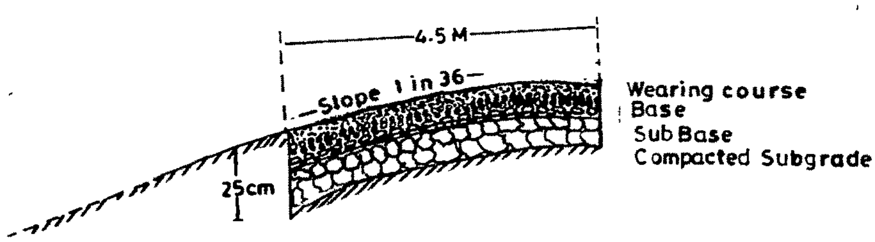


Fig.7.1 Map depecting sampling sites for geotechnical studies



(A) TYPICAL LONGITUDINAL PROFILE OF A ROAD PAVEMENT



(B) CROSS SECTION OF A MACADAM ROAD

Fig. 7.2 (A 3) Sections of typical road pavement.

materials i.e base and sub-base courses. Hence, the strength of a flexible pavement is the result of building up of thick base and sub-base courses Wherein, the optimum thickness of a flexible pavement is primarily depend on the strength characteristics of subgrade

Numerous methods of constructing the flexible pavements have been developed by several workers The prominent among them are

Tresaguet method
Metcalf method
Telford Method
Macadam method

Though all the above methods involve multi layered pavements, Mecadam's method of pavement construction is the first scientifically developed method. This method is based on the principle that the stresses due to the wheel load gets decreased at the lower layers of the pavement, and hence, it is not necessary to provide large stones and boulders as soaling course at the lowest layer of the pavement. A typical cross section of a Macadam method of pavement construction is illustrated in Figure 7.2 (b)

Subsequent improvements in the Macadam method of flexible pavement constructions that have been developed are .

(i) **Waterbound Macadam** (WBM) Roads involve compaction of the granular base and sub-base courses with the aid of water and natural soil binders.

(ii) **Bitumen Bound Macadam** (BBM) consists of base and sub-base courses of compacted, crushed aggregates premixed with bitumen binder.

(iii) **Bitumen Penetration Macadam** (BPM) method of pavement construction involves, spraying high viscous bitumen over a dry, well compacted aggregate spread

The bitumen sprayed penetrate into the voids of the compacted aggregates thereby fill-up part of the voids and also bind the stone aggregates Depending upon the quantity of bitumen sprayed and the extent of penetration BPM is further divided in to : **Full Grout** when bitumen penetrate to full depth of the compacted aggregate and **Semi Grout** when bitumen penetrates to half the depth

RIGID PAVEMENTS

In the rigid pavements, surface load is distributed over a wide area of the subgrade. The major portion of the load is born by the rigid upper layer itself by the bending action. Hence rigid pavements can withstand considerable amount of tensile stresses and also capable of bridging small weaknesses of subgrade The rigid pavements are generally made up of cement concrete, laid over the sub-base or at times directly over the subgrade As the load is distributed over a wide area of the subgrade, uniform subgrade support is the most important criteria for the good performance of the rigid pavements.

ROAD CATEGORIZATION AND PAVEMENT AGGREGATES SPECIFICATION

Depending on the nature of aggregates, binders, pavement design and method of construction, roads are broadly categorized as :

(i) Earthen roads and gravel roads

(ii) Soil stabilized roads

(iii) Water Bound Macadam roads

(iv) Bitumen or Blacktop roads

(iv) Concrete roads.

Before the construction of pavements for any of the above road category, the subgrade of the road alignment need to be improved by way of removal of trees, shrubs; excvation and backfilling to keep the vertical alignment and ensuring proper cross and longitudinal profiles

In poorly drained, shallow groundwater terrains as explained before, the pavement need to be constructed over an embankment, made with the help of locally available earthen material or mixtures

The density and compaction specifications (IRC, 1978) for the embankment construction are provided in Table 7 1 and 7 2 Irrespective of the category of pavement specifications, the granular material used in the construction of base and sub-base courses are broadly grouped in to three categories, depending upon the size gradation, consistency and bearing strength (Table 7.3)

EARTHEN

Earthen road is by far the cheapest type of pavement constructed from the locally available soil at the road site or near by borrow pits. The construction procedure involves, compacting the soil mixture in to layers (10 cm each) at OMC and a minimum 95 percent of maximum dry density ($\gamma_{d\ max}$). The specifications for the materials to be used for the construction of earthen roads are illustrated in Table 7 4

Soil stabilized road involves improving the strength parameters by way of adding stabilizer materials such as lime, cement, ash etc. The percentage of addition of stabilizer depend upon the nature of soil and the drainage conditions. The requisite gradation parameters of aggregates needed for ideal lime and cement stabilization are given in Table 7 5.

WATER BOUND MACADAM ROADS (W B M)

Water bound Macadam method is an improvised Mecadam's method of pavement construction especially for base and sub-base courses. The construction procedure involve mechanically interlocking the course aggregates along with a proper screening and binder materials. The screenings should preferably be non plastic IRC specifications for the coarse aggregates and screenings to be used in water bound Macadam roads are furnished in Table 7.6, 7 7 and 7 8 Depending upon the size variations, the coarse aggregates of the WBM pavements are divided into three grades viz.

Grade I comprising coarse aggregates of size range 90-40mm and is more suitable for sub-base courses

Type of embankment	Laboratory dry density (Kg / m^3.)
Embankment uoto 3m height	Minimum 1440
Embankment exceeding 3m height	Minimum 1520
Top 0 5 m of the embankment below the sub - grade	Minimum 1650

TABLE 7.1 LABORATORY DENSITY SPECIFICATION FOR EMBANKMENT CONSTRUCTION (IRC, 1978)

Embankment material	Field density as a % of laboratory Max. dry density (γ_{dmax}.)
Top 0.5 m portion of embankment below the subgrade leve and shoulder	Minimum 100 %
Other portions of embankment	Minimum 95 %
Highly expansive clays	Minimum 85 - 90 %

TABLE 7.2 MATERIAL AND FIELD DENSITY SPECIFICATIONS FOR EMBANKMENTS (IRC, 1978)

Sieve designation	Percentage by weight passing the sieve		
	Grade I	Grade II	Grade III
80 mm	100	100	100
63mm	90 - 100	90 - 100	90 - 100
4 75 mm	35 - 70	40 - 90	50 - 100
0.075 mm	0 - 20	0 - 20	0 - 30
CBR (minimum)	30 %	25 %	20 %
Liquid limit (Maximum)	25 %	25 %	25 %
Plasticity Index (Maximum)	6 %	6 %	6 %

TABLE 7.3 SPECIFICATIONS FOR GRANULAR BASE AND SUB - BASE COURSES (IRC, 1978)

Material	Base course	Wearing course
Clay content	Max. 5 %	10 - 18 %
Silt content	9 - 32 %	5 - 15 %
Sand content	60 -80 %	65 - 80 %
Liquid limit	Max 35 %	Max 35 %
Plasticity Index	Max. 6 %	4 - 10 %

TABLE 7.4 SPECIFICATIONS FOR MATERIAL FOR EARTHEN ROADS (IRC, 1978)

Sieve designation (mm)	Percentage by weight
50	100
40	95
20	45
10	35
4.5	25
0 600	8
0 300	5
0 075	0

TABLE 7.5 GRADING LIMITATIONS FOR THE AGGREGATES, IN CEMENT AND LIME STABILIZATION (IRC, 1978)

Type of construction	Test	Requirement
Sub - base	(I) Los - Angels abrasion value (OR)	Max. 50 %
	(ii) Aggregate Impact Value	Max. 40 %
Base course	(I) Los - Angels abrasion value (OR)	Max. 50 %
	(ii) Aggregate Impact Value	Max 40 %
	(iii) Flakiness Index *	Max. 15 %
Wearing course	(I) Los - Angels abrasion value (OR)	Max. 40 %
	(ii) Aggregate Impact Value	Max. 30 %
	(iii) Flakiness Index *	Max. 15 %

* Requirement of Flakiness test shall be enforced only in case of crushed, broken stones and slags, if plastic in nature.

TABLE 7.6 SPECIFICATIONS FOR PHYSICAL CHARACTERISTICS OF COARSE AGGREGATES IN WBM PAVEMENTS (IRC, 1978)

Grading No.	Size range (mm)	Sieve size (mm)	Percentage passing by weight.
I	90 - 40	100	100
		80	65 -85
		63	25 -60
		40	0 - 15
		20	0 -5
II	63 - 40	80	100
		63	90 - 100
		50	30 -70
		40	0 -15
		20	0 - 5
III	50 - 20	63	100
		50	95 -100
		40	35 - 70
		20	0 -10
		10	0 -5

TABLE 7.7 GRADINGS FOR COARSE AGGREGATES OF WBM PAVEMENTS (IRC, 1978)

Grading	Size of screening (mm)	Sieve designation (mm)	Percentagte passing by weight
A	12.5	12.5	100
		10.0	90 -100
		4 75	10 -30
		0 15	0 -8
B	10.0	10 0	100
		4 75	85 -100
		0 15	10 -30

TABLE 7.8 SPECIFICATIONS FOR THE SCREENING OF WBM PAVEMENTS (IRC, 1978)

Sieve designation (mm)	Percentage by weight passing			
	For 50 mm compacted thickness		For 75 mm compacted thickness	
	Coarse aggr.	Key aggr.	Coarse aggr.	Key aggr.
63	---	---	100	----
50	100	----	----	----
38	-----	-----	35 - 70	-----
25	35 -70	------	------	100
19	------	100	0 - 15	35 - 70
12	0 - 15	35 - 70	-----	-----
9	------	-----	-----	0 -15
4 75	------	0 -15	-----	-----
2 46	0 - 5	0 -15	0 -5	0 -5

TABLE 7.9 GRADING REQUIREMENTS OF AGGREGATES FOR BBM PAVEMENTS (IRC, 1978)

Grade II incorporates aggregates of size range 63-40 mm and

Grade III aggregates of the size range 50 - 20mm

The screenings of the WBM pavements should adhere to one of the following size gradings as given in Table 7 8, wherein the consistency of the screenings are

Liquid limit	Maximum 20 % ,
Plasticity Index	Maximum 6 %
Percentage of fines	Maximum 10 %

Compaction of the base and sub-base courses in WBM pavement is generally achieved through 6-8 tons roller with vibrators. Specified thicknesses of the compacted layers, depending upon the grade of the aggregates used are :

Grade I	100 mm
Grade II	75 - 100 mm

BITUMINOUS PAVEMENTS

Bituminous pavements are better performing and cost effective among all the flexible pavements Though, bituminous pavement layers are generally used as wearing course over WBM pavements, bituminous base and binder courses are also widely used for the better performance of pavements under heavy traffic loads

The most important types among the bituminous pavements are

(i) Wearing coat or Seal coat is made in one or two layers over an existing pavement to serve as thin wearing coat. They are also applied over the existing previous pavements, to avoid water penetration

(ii) Bituminous Penetration Macadam (BPM) is made as a base or binder course to a thickness of 50 to 75 mm The construction procedure (as elucidated earlier) involves spraying hot, high viscous bitumen over the well compacted layer of coarse aggregates and allow the bitumen to penetrate the voids and bind the aggregates

IRC (1978) specifications for physical requirements of the aggregates of BPM pavements are

Los Angels abrasion value	35 % maximum
Aggregate impact value	30 % maximum
Flakiness index	35 % maximum
Stripping value	· 25 % maximum
Water absorption	: 2 % maximum

Size gradings for the coarse and key (fine) aggregates of BBM are given in Table 7 9 The BBM pavements are suitable only as base or binder course on account of their open graded nature The construction methodology includes spreading and compacting the premixed (aggregates with bitumen) aggregates of any of the grade as specified in Table 7 3 Like BPM pavements, BBM pavements are made at compacted thickness of 75 mm and 50 mm depending up on the gradation of the aggregates. The BBM pavements are exposed as a surface course wherein, a seal coat is mandatory. Physical requirements of aggregates of BBM pavements are

Los Angels abrasion value	: 50 % maximum
Aggregate impact value	: 35 % maximum
Flakiness index	: 15 % maximum
Stripping value	· 25 % maximum

For binder course the specified maximum abrasion and impact values are 40 and 30 percent respectively The gradings of the aggregates for 75mm and 50mm thickness for base and binder courses are given in Table 7 10.

REVIEW OF PREVIOUS LITERATURE

In the tropical countries, the road construction using residual soils and weathered rock aggregates have been gaining momentum due to their durability and cost effectiveness Among the residual soils, utility of laterites in pavement construction as base and sub-base

Grain size (mm)	Percentage by weight passing		
	Base course	Base / binder course	
	Grading 1	Grading 2	Grading 3
(a) for 75 mm compacted thickness			
63	100	100	-----
50	-----	90 -100	-----
40	35 -70	35 - 65	100
25	------	20 - 40	70 -100
12.5	------	5 - 20	-----
10 0	------	------	25 - 50
4.75	------	------	10 - 30
2.36	0 - 5	0 - 5	5 - 20
0 075	0 - 3	0 - 5	0 - 4
Binder content % by weight of mix	3 - 4.5	3 - 4.5	3 - 6
(b) for 50 mm compacted thickness			
50	100	100	-----
40	-----	90 - 100	-----
25	35 - 75	50 - 80	100
20	-----	-----	70 - 100
12 5	0 - 15	10 - 30	------
10	-----	------	35 - 60
4.75	-----	-----	15 - 35
2.36	0 - 5	-----	5 - 20
0.075	0 - 3	0 - 5	0 - 4
Binder content ,% by weight of mix	3 - 4.5	3 - 4.5	3 - 6

TABLE 7.10 GRADING OF AGGREGATES FOR BBM ROADS

courses has gained much attention for over two decades and even separate acceptance specifications for lateritic (Here after referred as ferricrete in accordance to Goudie, 1973) materials are envisaged (Giddigasu, 1980, 1983, Giddigasu et al., 1987; Alao, 1983, Madu, 1977, Johnson, Ola, 1978, Omotosho et al , 1992, Akpokodje et al , 1992) in many tropical countries In India, besides several works (Wadhawa et al ,1966b, Arulanandan, 1969, Mohan et al 1975; Rao S and Raymahashay, 1981, Narayanaswamy et al , 1987, Nambiar and Chandrakaran, 1991; Chandrakaran and Nambiar, 1993) the concept of low cost pavements using residual aggregates (ferricretic) has not gained much attention However, no work has so far been available in India on the utility of the calcretes, the other residual aggregates occurring in arid and semi-arid terrains for the road pavement construction Hence, some of important studies carried out in other parts of the tropical countries are presented below

Giddigasu (1980) evaluated the utility of lateritic aggregates for base and sub-base courses He established the influence of consistency of these aggregates to the strength characteristics and the gradation properties to the compaction characteristics Combination of smooth wheeled roller with vibrator is found to be most effective in achieving the minimum 95% of $\gamma_{d\ max}$ in the field conditions.

Giddigasu (1983) proved the significance of location specific specifications for the residual aggregates. As the general specifications based on index properties found to be misleading, in case of residually formed aggregates. The author stressed the significance of mineralogy, mode of formation of the residual aggregates, before designing the specifications. The classification between the aggregates for base and sub-base materials is drawn on the basis of CBR values ie 80% and above for the former.

Giddigasu et al , (1987) arrived at important correlations between the bearing strength (CBR) on one hand and the product of fines and consistency properties on the other hand He further apprehended the significance of such parameters in making specifications for the residual aggregates.

Mohan and Paul (1975) derived a maximum density under the specified moisture conditions by simply re-adjusting the grain size of an otherwise troublesome, gap graded laterite This property of mechanical stabilization is proved to be of vital significance in improving the density and CBR values of poor, residual aggregates

Akoto and Singh (1981) elucidated another method of stabilizing high plastic residual aggregates by the addition of lime With a lower percentage of lime and longer curing time he has observed loss in the strength, but with higher percentage of lime under prolonged curing time the strength of the aggregates has enhanced appreciably.

Similar to the results of Akoto (op cit), Ola (1977) and Rossi et al (1983) have also proved the enhancement of strength characteristics and reduction in plasticity index, for different category of lateritic soils from A-1-a to A-7-6 He recommended 6% lime addition for the optimum performance of these aggregates

Malomo (1983) identified "collapse behaviour" in compacted lateritic aggregates, a property so far has been observed only in the case of undisturbed aggregates He attributed this phenomena in the compacted aggregates to the communition of degradable soil particles

Chandrakaran and Nambiar (1993) studied the effect of iron oxides on the strength of laterites He concluded that not the total iron oxides present in the laterites are detrimental to the strength but it is the form of iron (ferric and hydroxides) that control the strength

Setty and Rao (1987) explained the enhancement of CBR, tensile and shear strength of cohesive residual aggregates by the addition of small, discontinuous, randomly oriented Garware twine fibres (CH_2 -- CH_2)

Chandrakaran and Nambiar (1987) to understand the effect of environment on the geotechnical characters of lateritic duricrusts, subjected laterites to cyclic wetting and drying

These workers further concluded that lateritic duricrusts exhibit significant increase in permeability and shearing strength while compressibility decreases with the increase in the number of compaction cycles Similar studies on multicyclic compaction of ferricretic aggregates carried out by Omotosho and Akinmusuru (1992) revealed that the aggregates are better graded, less plastic with a decrease in friction angle The cohesion of the soil increases up to 8 cycles and then gets stabilized

Mahalik and Das (1982) established correlation between the bearing strength of the lateritic soils to the texture, density, percent of fines and percent of coarse fractions Though, voluminous data are generated on the geotechnical behaviour of the iron duricrusts (laterites) still, very little information is available on the geo-engineering characteristics of calcretes, the other duricrust widely prevalent in arid and semi arid regions particularly the Thar desert.

Al-Sulami et al. (1990) had carried out a detailed study on the geotechnical characteristics of calcretes and their utility in arid zone road pavements They had further corroborated the high angle of friction in calcretes on compaction can better be utilised in their performance as road aggregate.

Horta (1980) evaluated calcretes and gypsicretes as pavement aggregates in arid terrains. Calcrete is an excellent road paving aggregate if used within the climatic conditions corresponding to the stability of calcite in soil profiles. He further necessitated the significance of introducing special classes (SE, GE for calcrete sands and gravels) to the Unified soil classification.

Doshi and Guirguis (1983) by extensive laboratory analyses of calcretes established good correlation between CBR and OMC, $\gamma_{d\,max}$ as given under :

Soil type	Linear regression equation
A-1-b	CBR = -219 46 + 3.07 (OMC) + 106 7 ($\gamma_{d\,max}$)
A-3	CBR = -188.78 + 2 15 (OMC) + 97.88 ($\gamma_{d\,max}$)

A-2-4	CBR = -147 74 + 1.49 (OMC) + 78 10 ($\gamma_{d\,max}$)
A-4	CBR = 187 81 - 3 12 (OMC) - 69 71 (γ_{dmax})

Akpokodje (1985) in support of Horta's (1980) view of special status for calcretes and gypsicretes in Unified soil classification with a modification that 40 percent carbonate and 30 percent gypsum suggested by the former to 20 percent for both the classes

GEOTECHNICAL CHARACTERISTICS OF CALCRETES AND FERRICRETES OF THE THAR DESERT

(I) INDEX PROPERTIES AND CLASSIFICATION

The index properties of the road paving aggregates control the strength and compaction behaviour to great extent and hence their evaluation forms a pre - requisite in deciphering the geotechnical characteristics. Tests (both field and laboratory) carried out to evaluate the various index properties viz Natural Moisture Content(NMC), Insitu Density, Liquid Limit (LL), Plastic Limit (PL), Grainsize Gradation etc. (Table 7.11, Figure 7 3 a,b,c,d) reveal that

The NMC of these aggregates vary in accordance to the depth of occurrence and their geomorphic associations Powdery calcretes, soft nodular calcretes and calcretes associated with alluvium are found to have more NMC Similarly, the field density of calcretes also found to depend upon the degree of calcretisation and source sediments that have been calcretized.

Nodular calcretes of fine sand association with different percentage of sand matrix have field density around 900 kg/m ^{3} While the coalesced nodular calcretes, hard pan varieties and calcretes associated with gravelly sands, weathered rocks have field density between 1200-1300 kg/m^3

On the other hand the ferricretes have higher field density (1200 - 1537 kg/m ^{3}) with a variation in NMC between 7.1 to 28.7 %. The grain size parameters of calcretes and ferricretes comprise predominantly gravel size particles.

Sa. No.	Location	% Fine	Mean grain dia. (mm)	LL & PL (%)	NMC (%)	Field dens. Kg / m.^{3}	Soil Classification	
1	Napasar	0.2	6.2	NP	7.1	1216	GP	A - 1 - a (0)
2	Shergarh	18.0	1.5	NP	14 8	872	SM	A - 1 - b (0)
3	Sultana	2.9	3 5	NP	15.7	882	GW	A - 1 - a (0)
4	Bhukan	14 3	1.25	NP	18 7	1357	SM	A - 2 - 4 (0)
5	Kanod Rann	20 4	2.5	NP	19.0	972	GM	A - 1 - b (0)
6	Bersi	35.8	0.25	NP	5 6	1181	SM	A - 2 - 4 (0)
8	Dechchu	11 8	4 5	NP	14.3	881	SM	A - 1 - b (0)
9	Khudi	16.5	1.50	NP	9 6	1434	SM	A - 1 - b (0)
10	Shersingh dhani	14.4	0.25	NP	12.6	1450	SM	A -2 - 4 (0)
11	Derasar	4 6	0 15	NP	9.2	1358	SP	A - 3 (0)
15	Longewala	21 1	1 9	NP	18.1	1120	SM	A - 1 - b (0)
16	Surpalia	4.4	0.2	NP	23.7	920	SW	A - 3 (0)
18	Nimbijodhan	1.1	7.0	NP	14.1	Nd.	GP	A - 1- a (0)
19	Raneri	19.9	3.1	NP	31.2	1458	SM	A - 1 -b (0)
20	Sardarsahar	33.0	0.25	NP	27.3	921	SM	A - 2 -4 (0)
21	Bandah	5.9	4.80	NP	24 0	1268	GW	A - 1- a (0)
22	Asutar	3 9	9.0	NP	7 1	1279	GW	A - 1- a (0)
23	Ramgarh I	10.4	1.2	NP	26.0	1147	GM	A - 1- b (0)
24	Khinya	10.9	3 2	NP	17.4	1537	GM	A - 1- b (0)
25	Girab	2.0	30.0	NP	28 7	Nd.	GW	A - 1- a (0)
26	Ramgarh II	*	*	*	*	*	*	*
27	Ramgarh III	*	*	*	*	*	*	*
28	Savanta	*	*		*	*	*	*
29	RD 105	*	*	*	*	*	*	*

*	Aggregates are massive
Nd	Not determined
Sa No 1 - 20	Calcretes
Sa. No 21 - 29	Ferricretes

TABLE 7.11 INDEX PROPERTIES OF CALCRETES AND FERRICRETES

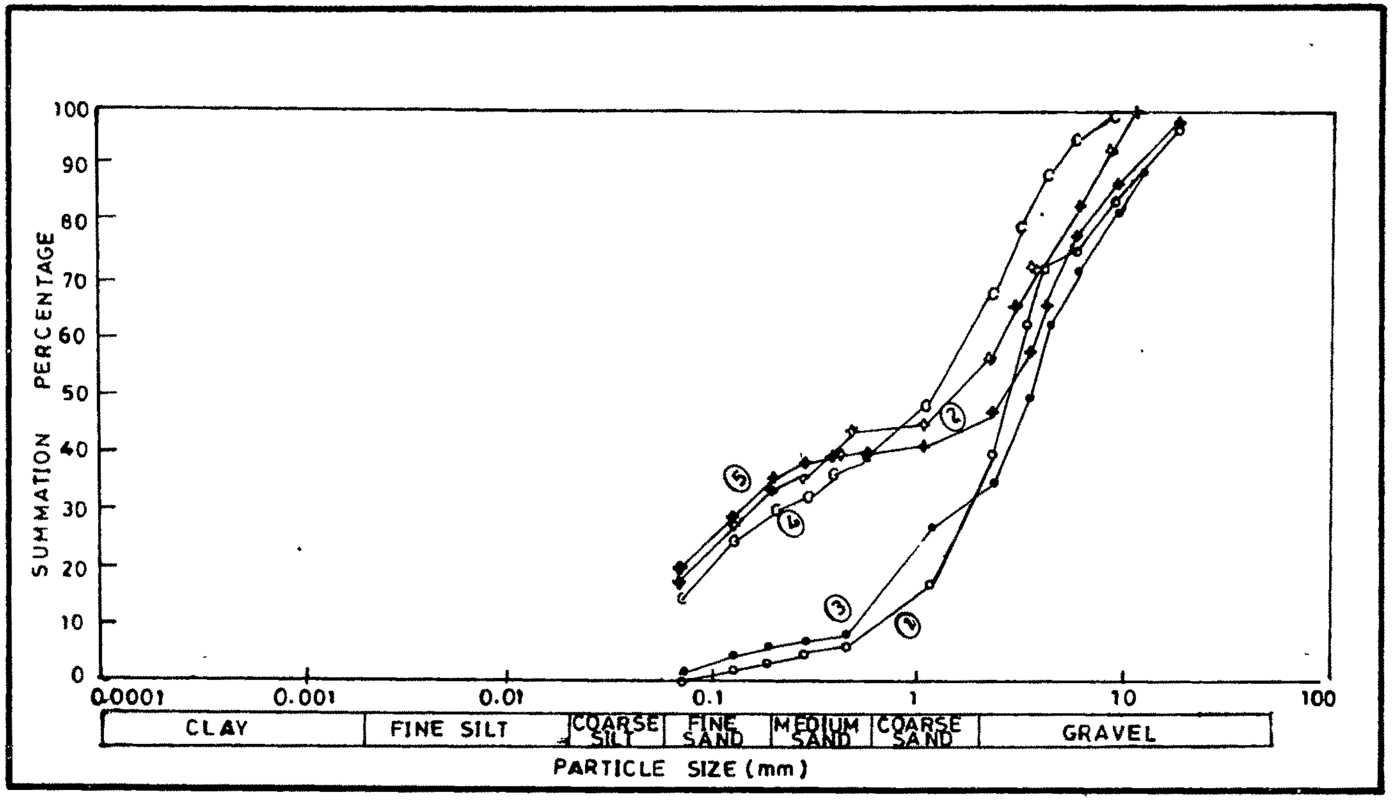


Fig. 7.3.(a) Size gradation curves of calcretic aggregates.

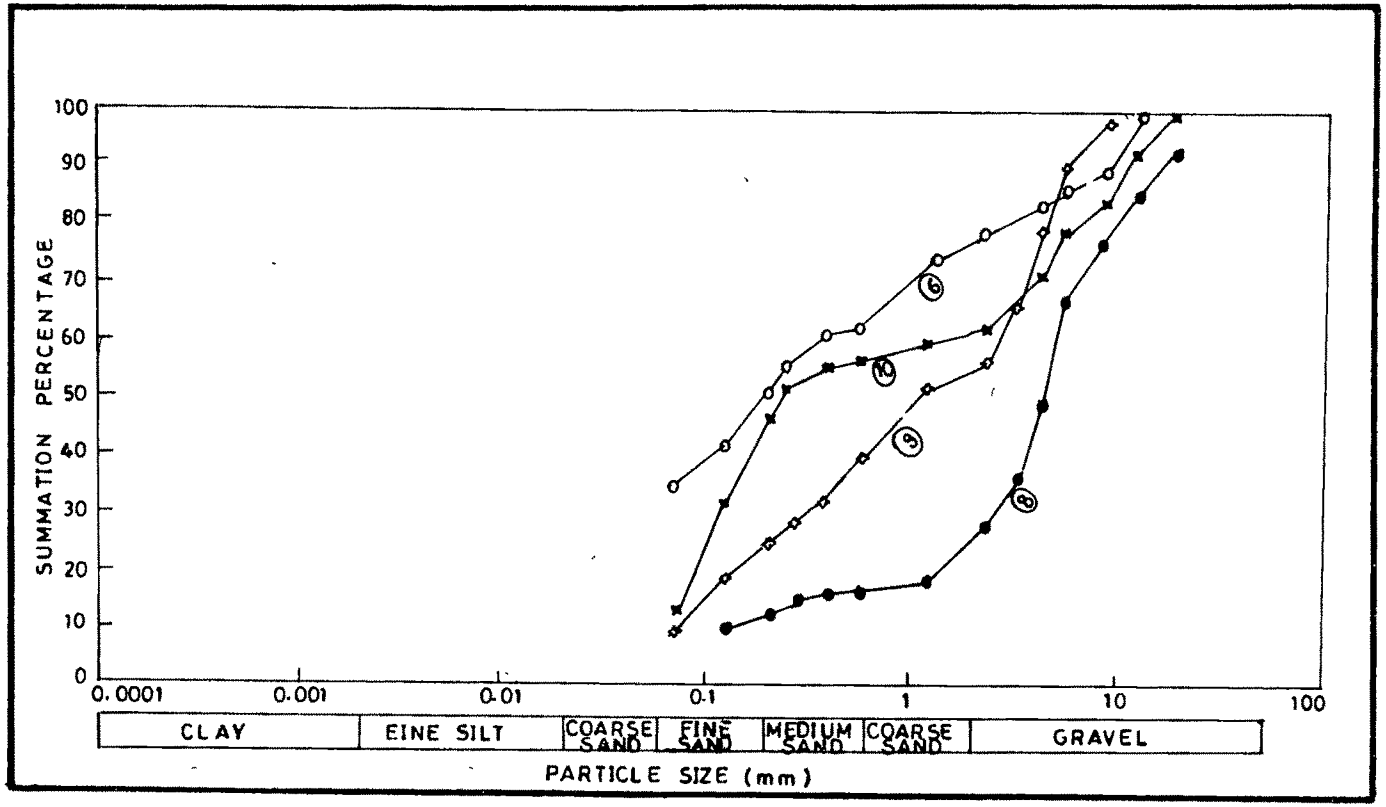


Fig. 7.3.(b) Size gradation curves of calcretic aggregates.

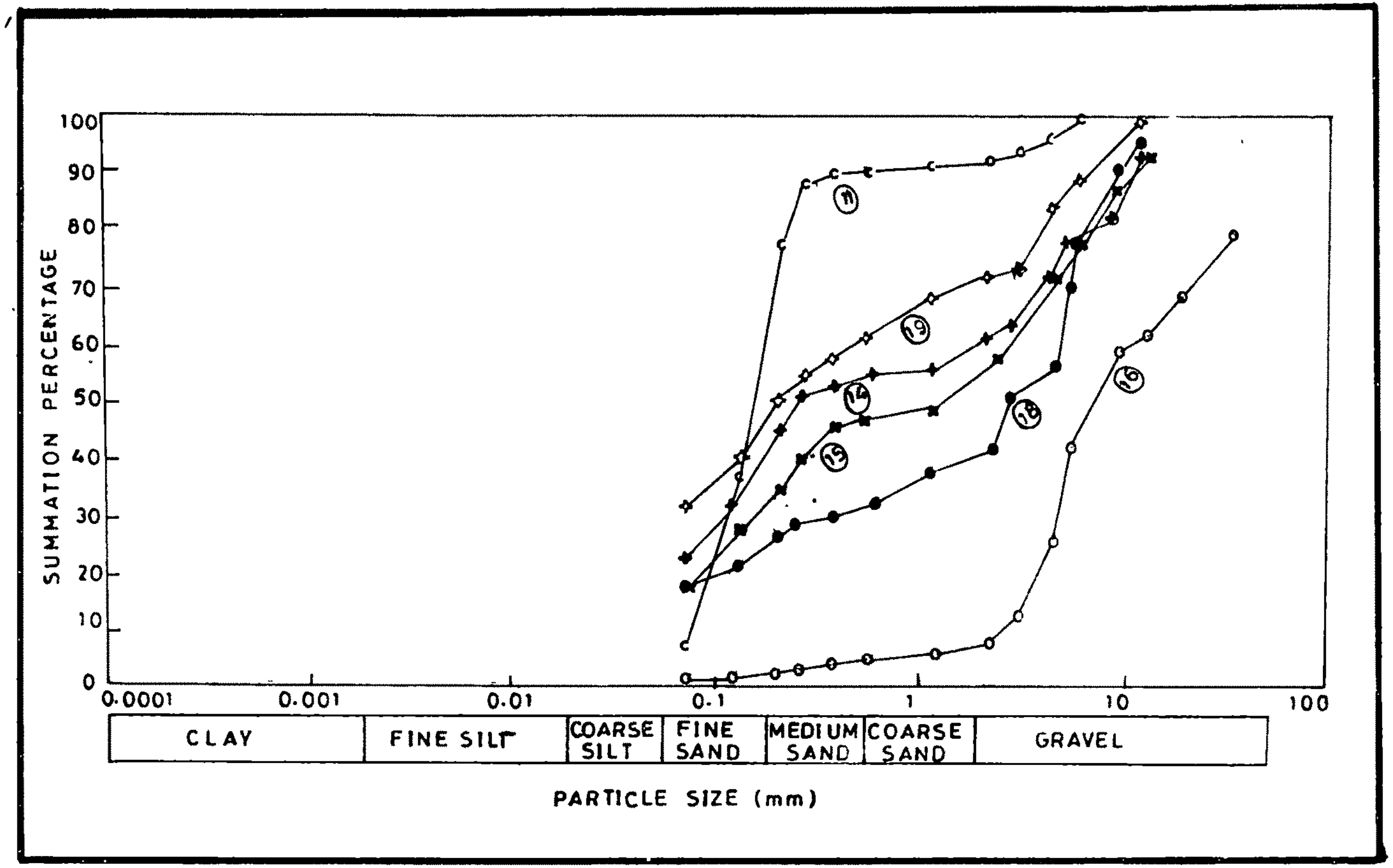


Fig. 7.3(c) Size gradation curves of calcretic aggregates.

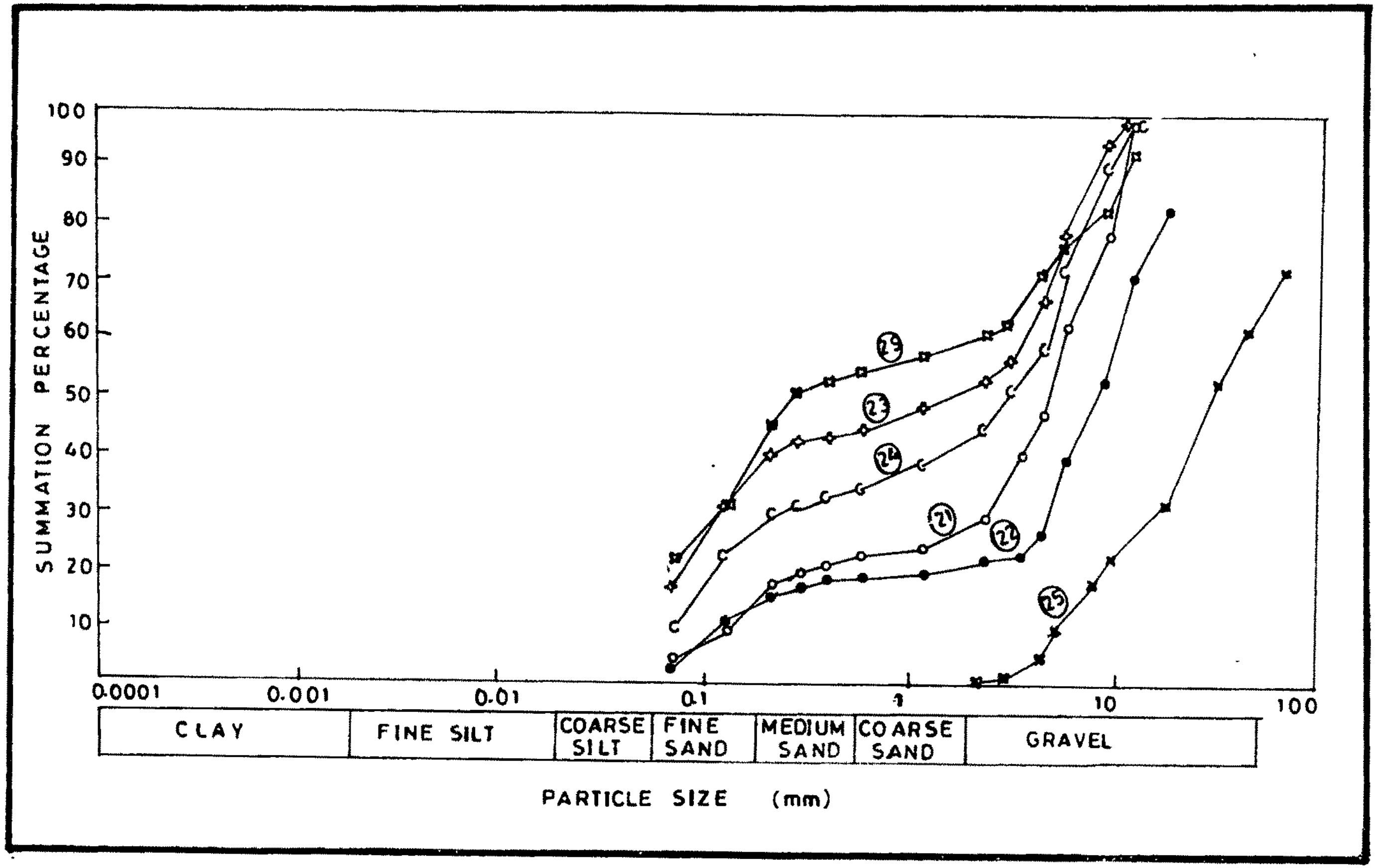


Fig. 7.3(d). Size gradation curves of ferricretic aggregates

From the gradation curves of both calcretes and ferricretes it is evident that they are mostly well graded. However, some of the calcretes (sample No 11,16) and ferricrete(25) are uniformly graded Gap gradation though not commonly observed, a near gap gradation is noticed in some of the ferricretes (sample 14,21,22) The percentage of fine (< 0 075 mm) of the calcretes vary from 0 2% (sample 1) to 35.8 % (sample 6).

Though, the ferricretes in general occur as massives, the dissected and calcretised horizons have percentage of fines ranging between 2 and 23 2. The mean grain diameter (D_{50}) of calcretes varies from 0 25mm to 6.2mm and that of ferricretes are 0.28mm to 30mm there by indicating relatively coarser nature of the ferricretes with little fines than calcretes

The LL and PL values (carried out on the fractions passing 0 4 mm) of these aggregates are found to be Non Plastic (NP) in nature

Based on the gradation and consistency characteristics, the calcretes and ferricretes are classified using the Unified Soil Classification (USC) and American Association of State Highway Official's (AASHO) classification, the two widely practiced for road paving aggregates Calcrete aggregates fall commonly under the gravel and sand classes of USC

The gravel, sand classes again comprise gravel well graded (GW), gravel poorly graded (GP) sand-gravel mixture (SM), sand well graded clearly manifesting the dominance of gravel fractions Ferricretes on the other hand are predominantly fall under gravel class (GW & GM).

In the AASHO aggregate classification the gravel domination is reflected by wide prevalence of A-1 (A-1-a and A-1-b) class in both calcretes and ferricretes The nonplastic sand dominated samples fall under A-2-4 (0) class and gravel-sand mixture samples are represented by A-3 class.

(II) COMPACTION CHARACTERISTICS

Evaluation of the compaction characteristics of calcretes and ferricretes is vital to understand the relationship between moisture content and the dry density variations under a particular energy of compaction Further, as the maximum strength of the aggregate is also obtained only at Maximum dry density ($\gamma_{d\ max.}$) and corresponding Optimum Moisture Content (OMC) at any compacted energy, the $\gamma_{d\ max}$ and OMC of calcretes and ferricretes need to be evaluated.

The compaction characteristics of any aggregate in general is controlled to great extent by the percentage of fine, gradation of aggregates, and consistency (Mohan and Paul, 1975, Giddigasu, 1983, Giddigasu et al., 1987, Saha and Chattopadhyay, 1988) Hence, compaction characteristics of calcretes and ferricretes (having similar physical properties) representing different AASHO and USC classes are evaluated (Table 7.12, Figure 7 4 a,b,c) using the Standard Proctor Energy (625 kJ/M^3)

The $\gamma_{d\ max}$ and OMC of calcretes ranges between 1753 & 1921 kg/m^3 and 11 1 & 13.6 percentage respectively The plotted compaction curves further elucidates that .

(i) Two distinct groups of compaction behaviour prevails within calcretes as indicated by changes in $\gamma_{d\ max}$ values (around 1750 kg/m^3 for the first and 1900 kg/m^3 for the latter) and the almost similar range of OMC

(ii) The compaction curves are having a broad base, pointing to the gradual increment and fall in dry density as moisture increases.

(iii) However the asymmetrical nature of the curves with a steep slope after attaining the OMC, point to rapid fall in dry density with increase in moisture content beyond OMC

The higher $\gamma_{d\ max}$ (sample 5,9,11) can be ascribed to the well graded nature of the aggregates, while the low γ_{dmax} (sample 3,6,10) in some calcretes within a narrow

Sample no	O M C (%)	γ_{dmax}, Kg / m^3,
3 (Calcretes)	14	1764
5	12 8	1782
6	13.6	1753
9	11.2	1921
10	11.1	1911
11	13.1	1902
21 (Ferricretes)	10.5	2130
22	8 4	2254
23	11.1	2263
24	9.4	2231
A (Calc+Ferr.)	8 7	2080
B	10.0	2049
C	10.7	1965

TABLE 7.12 STANDARD PROCTER - COMPACTION CHARACTERISTICS OF CALCRETES AND FERRICRETES

Sample No.	California Bearing Ratio		Aggregate Impact Value (%)
	Un soaked	48 hours soaked	
3	-------	-------	26.8
5	------	-------	47.5
6	8.6	15.2	19.6
9	10.5	34.1	12.8
10	27.3	35.7	23.2
11	14.2	34.1	31.5
21	------	------	30 8
22	24.8	38.5	13.9
23	------	------	29.3
24	14.6	36.2	23.8
B	31.0	37.2	------
C	14.6	42 9	------

TABLE 7.13 STRENGTH PARAMETERS OF CALCRETES AND FERRICRETES

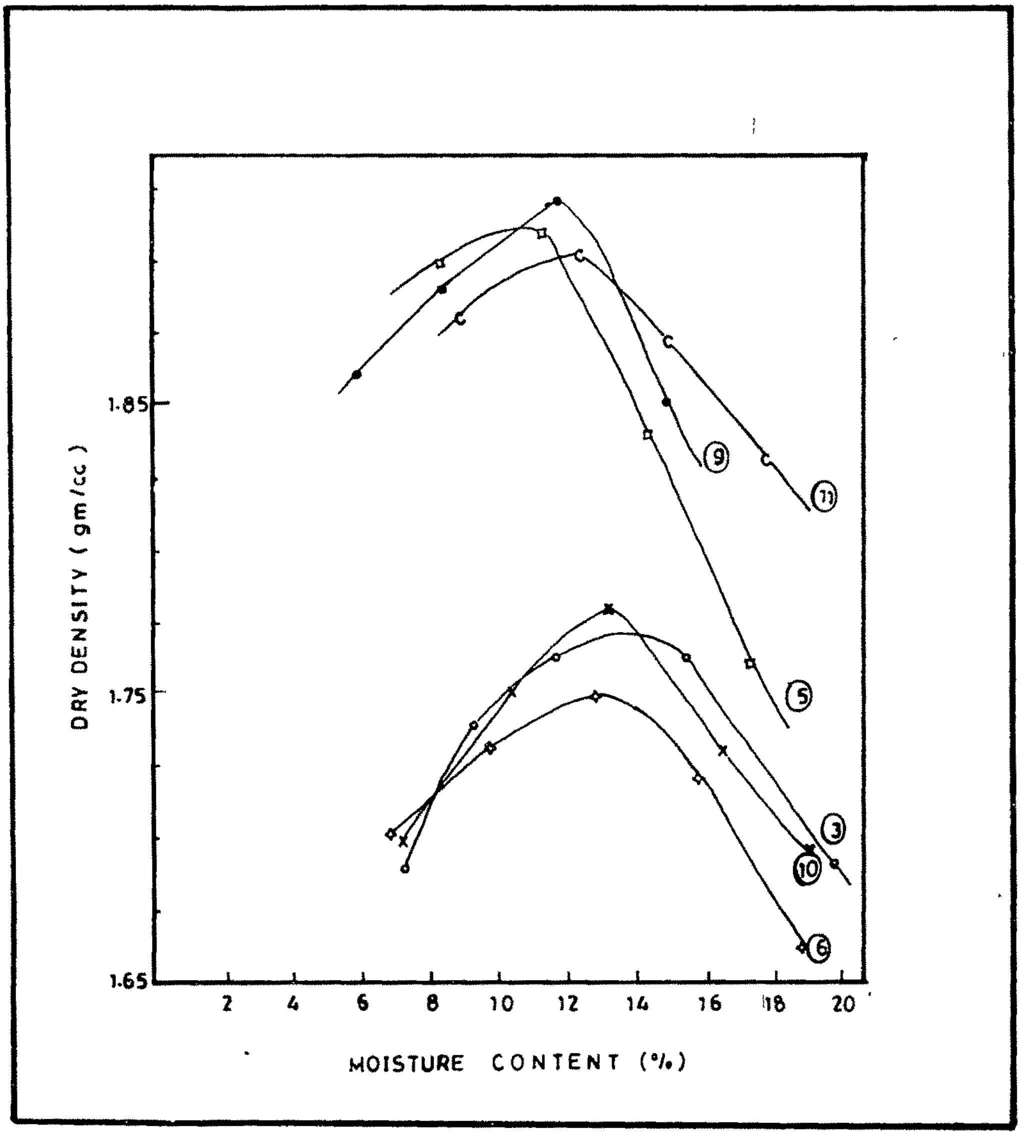


Fig.7.4(a). Procter compaction curves. calcretes.

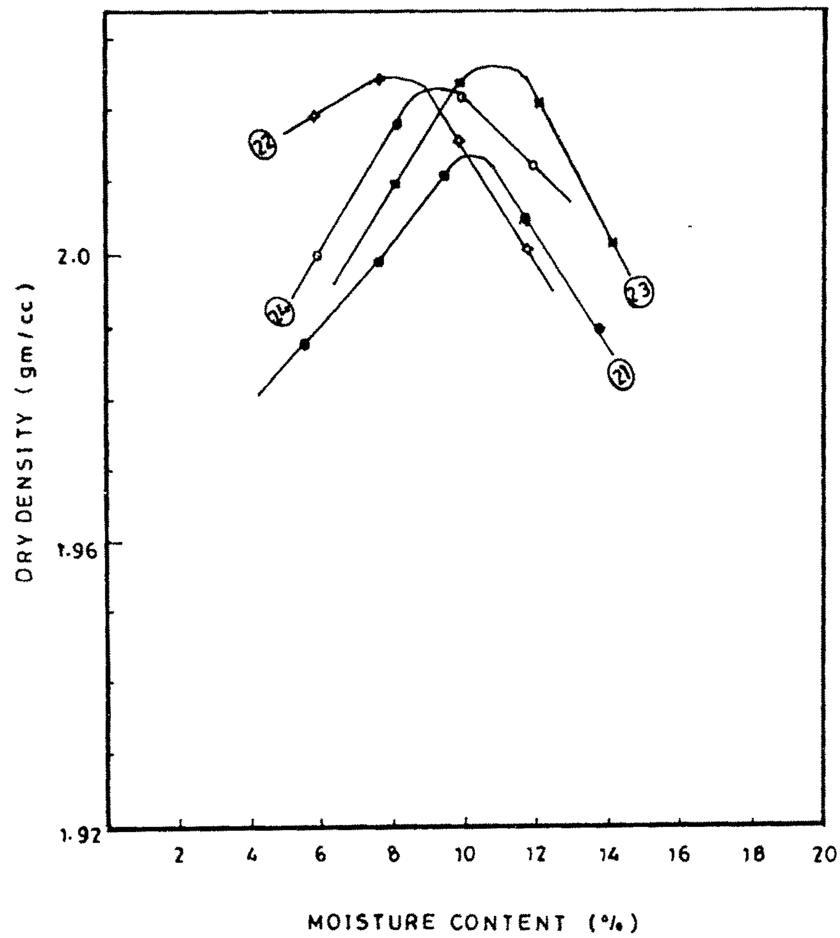


Fig.7.4(b). Procter compaction curves
- ferricretes.

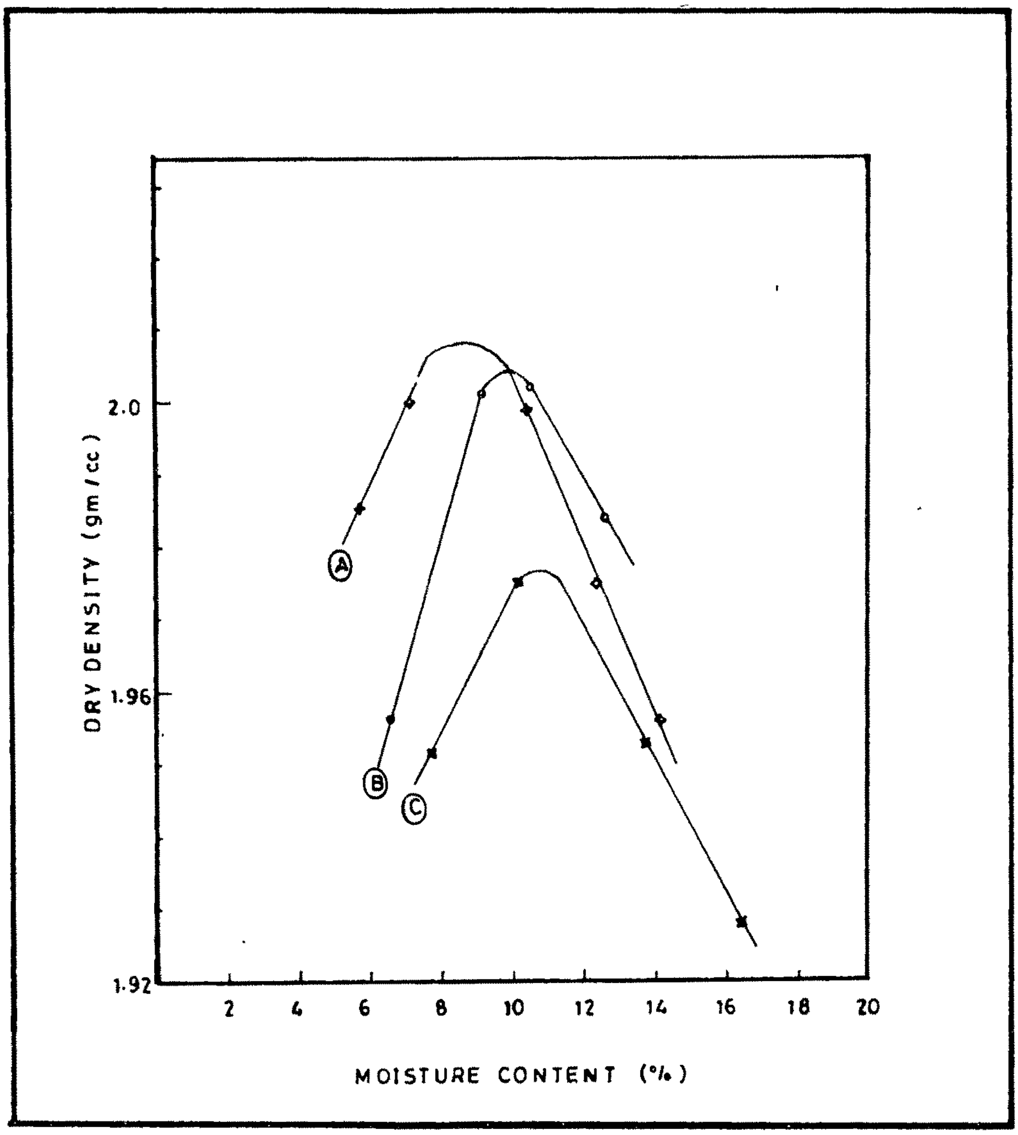


Fig.7.4(c). Procter compaction curves calcrete - ferricrete mixture.

spectrum of OMC can be ascribed to their near uniform gradation and relatively higher percentage of fines

On the contrary, the ferricretes and calcretised ferricretes have a $\gamma_{d\,max}$ range between 2130 - 2263 kg/m^3 at a relatively lower OMC (8.1 - 11 1 percentage) than that of calcretes Further the shape of the compaction curves is peaked implying, that a little change in the moisture content will affect the γ_{dmax} considerably and the density loss is rapid after the OMC The elevated level of γ_{dmax} at lower OMC of ferricretes is attributed to low percentage of fines and well graded nature of the aggregates.

In order to have better understanding, the compaction and strength characteristics of calcretised ferricretes; clacrete-ferricrete sample mixtures (of the same USC class) were prepared in 1 : 1 ratio and the compaction characteristics were evaluated. It is evident from the compaction curves of these samples ((Figure 7.4 [C]) that addition of calcretes to ferricretes resulted in lowering of $\gamma_{d\,max}$ (1965 - 2088 kg/m^3), but without any significant modification in OMC (8.7 - 10 2 percent). Although, the compaction curves have slightly wider base than the ferricretes, they still inherit the peaked nature of the ferricretes

(III) STRENGTH PARAMETERS

Different strength characteristics of the calcretes, ferricretes, calcretised ferricretes and calcrete-ferricrete mixtures are evaluated by California Bearing Ratio (CBR), North Dakota Penetration Values and Aggregate Impact Values so as to assess their suitability in accordance to the existing specifications for road aggregates.

(a) California Bearing Ratio (CBR)

Akin to the compaction characteristics, the bearing strength of the aggregates (Hammond, 1970, Giddigasu et al., 1987) is also controlled by the index properties. The well graded and non plastic nature of these calcrete and ferricrete have been taken as an advantage in evaluating the bearing strength of representative AASHO classes. Wherein, the CBR values of these aggregates are evaluated at OMC and $\gamma_{d\,max}$

To stimulate the worst field conditions caused by the increase in moisture content and subsequent changes in compaction and bearing strength, the bearing strength (CBR values) of these aggregates has been estimated both under unsoaked condition and 48 hours water soaked conditions (Table 7 13; Figure 7 5 a,b,c). The very wide deviation of load-penetration in some calcretes (sample No.6,9,11) to that of the standard curve is attributed to the higher values of OMC and low values $\gamma_{d\ max}$

The CBR values of the unsoaked calcretes range between 8.6 - 27.3 percent, while their corresponding counter parts have a enhanced value of 15.2 - 35.7 percent, without any significant enhancement in moisture content (0.5 percent) and free swelling. The CBR values of the ferricretic aggregates are comparatively higher than the calcretes. The unsoaked and soaked CBR values of A-1-a class of ferricretes are 24 8 and 38.5 percent While the A-1-b class of ferricretes have the unsoaked and soaked CBR values of 14 6 and 36 2 percent. This slight drop in the CBR values of these two classes of ferricretes again reflect the reduction in $\gamma_{d\ max}$ in the latter class.

The CBR values of the calcrete-ferricrete admixtures fall between calcretes and ferricretes (unsoaked CBR 14.6 & 31.0, Soaked CBR 37.2 & 42.9 percent). There is an increment in the CBR values between soaked and unsoaked aggregates, the degree of increment varies between 186 percent (sample C) and 20 percent (sample B)

(b) North Dakota Test

In addition to the CBR method of bearing strength determination, North Dakota tests were also carried out for selected classes. Further, unlike the CBR method (determined at γ_{dmax} and OMC), North Dakota bearing strength has been determined at different moisture content, both above and below the OMC, $\gamma_{d\ max}$. (Table 7.14) The inferences drawn on the basis of the test are :

(i) The bearing strength of the calcretes and ferricretes is maximum, close to the OMC and $\gamma_{d\ max}$.

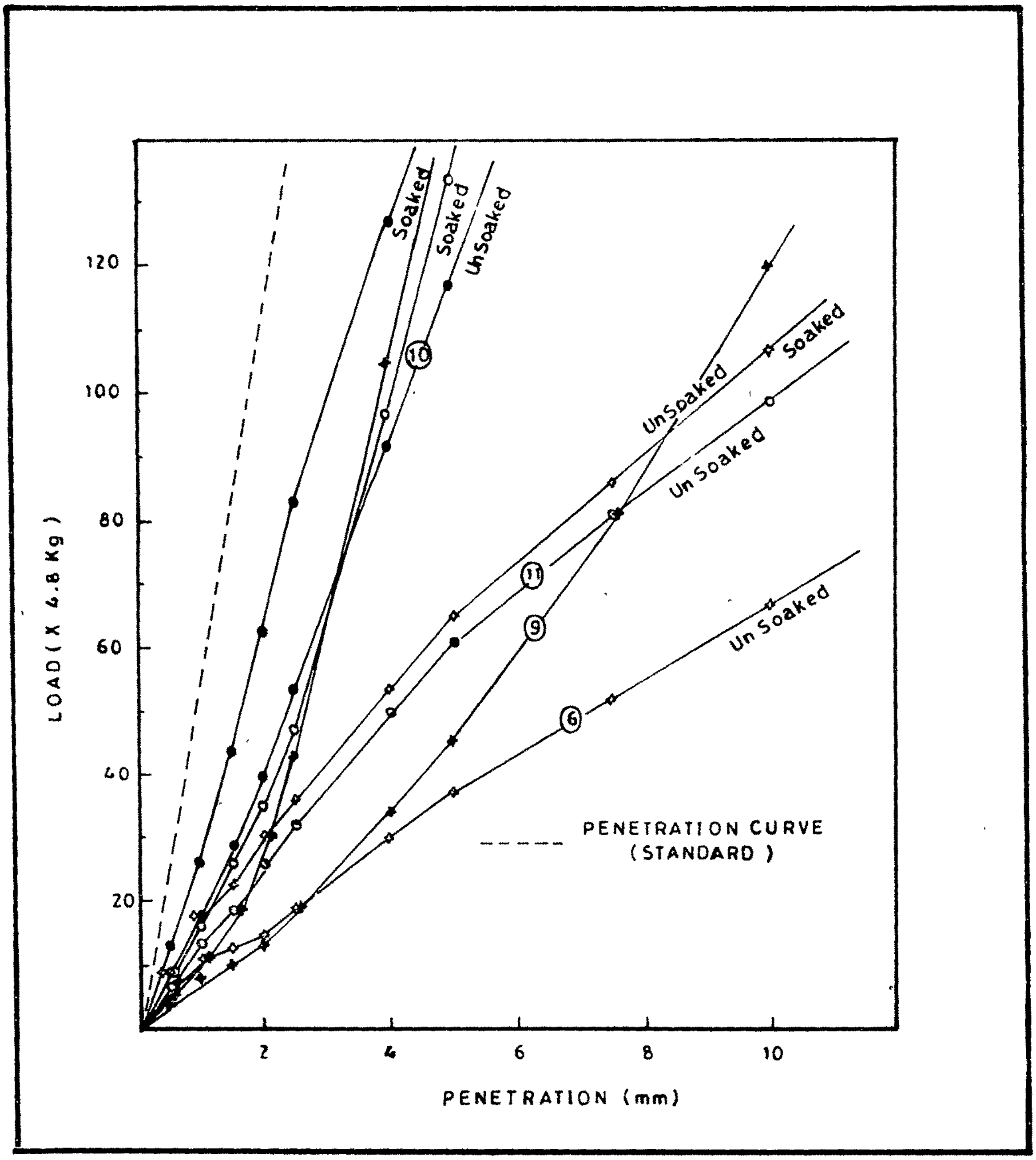


Fig.7.5.(a) CBR-Test curves calcretes.

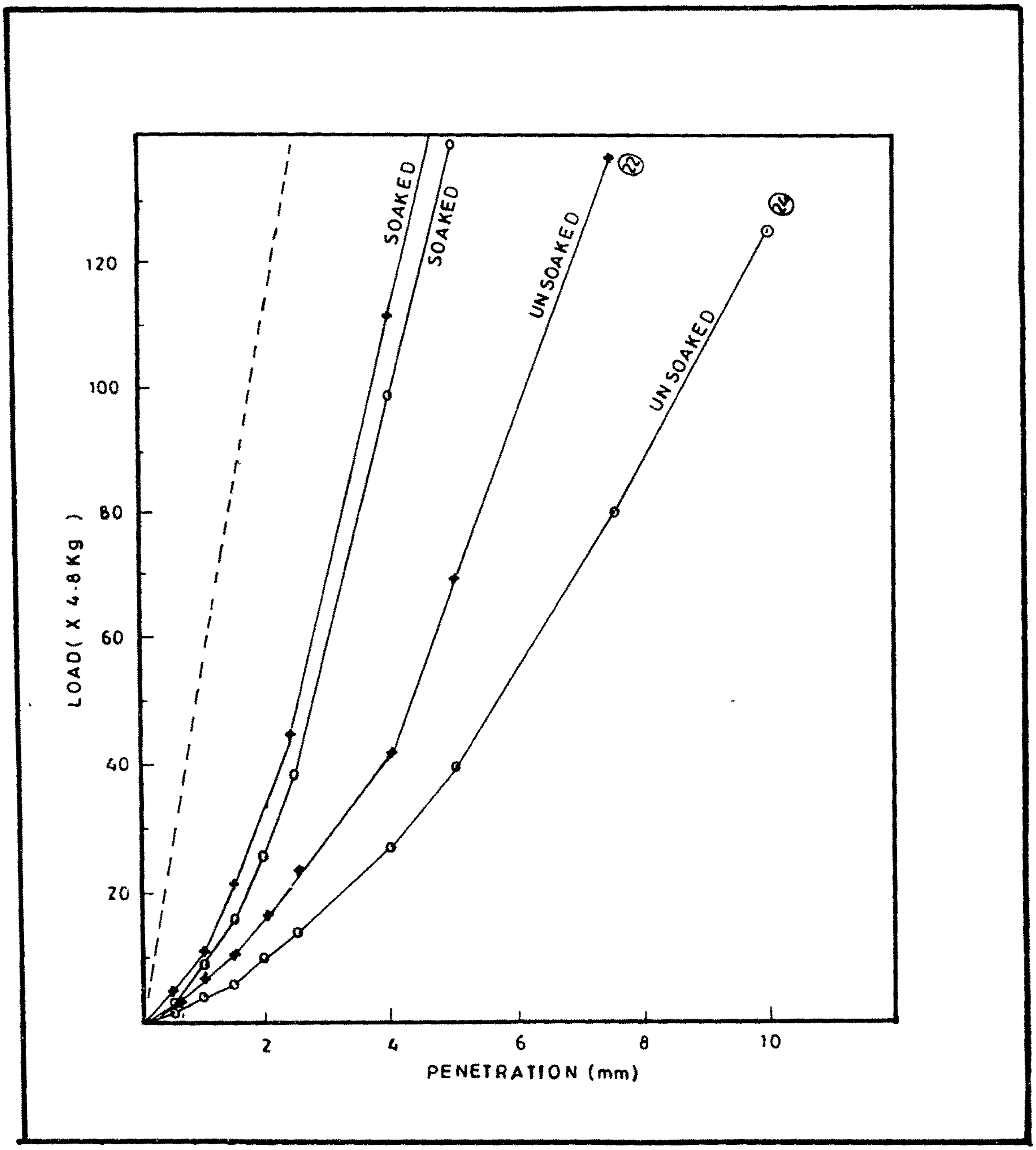


Fig.7.5(b). CBR-Test curves ferricretes.

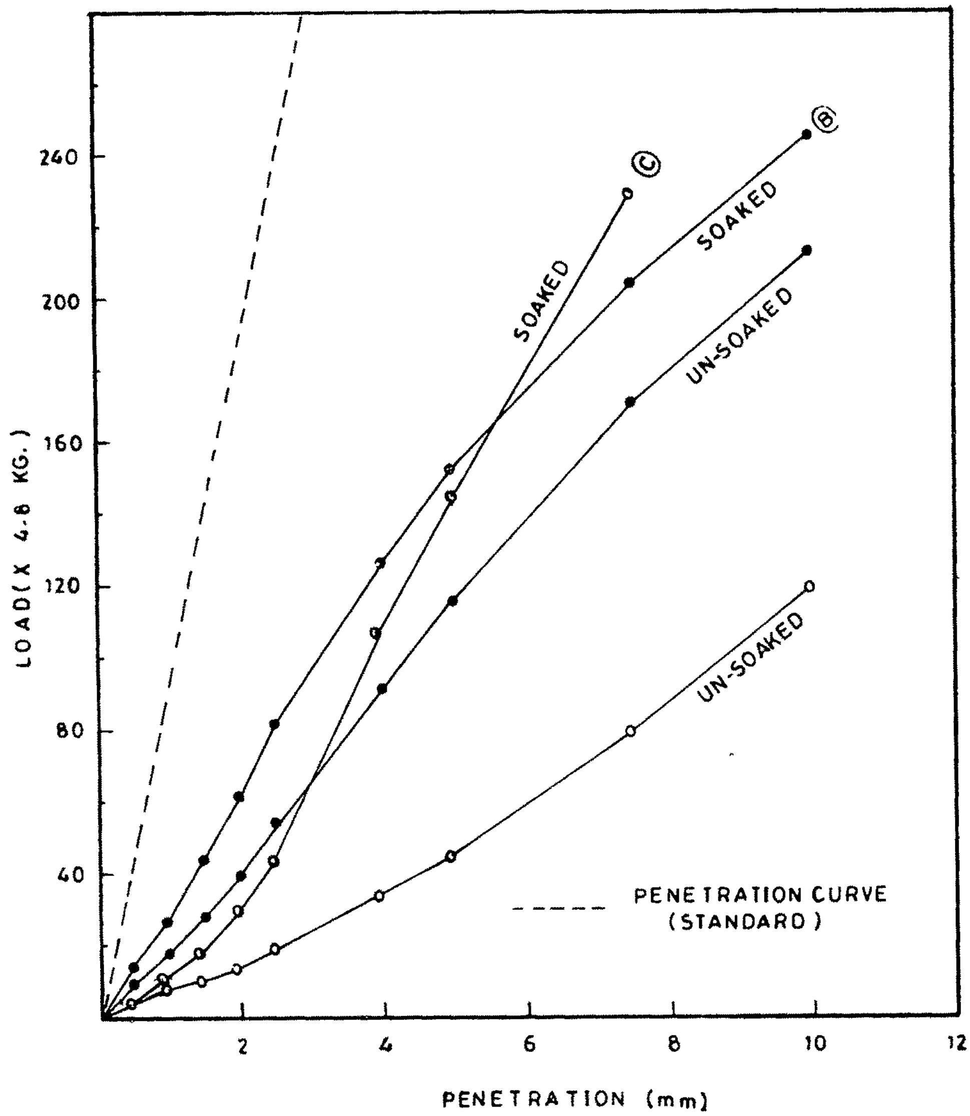


Fig. 7.5(c). CBR-Test curves calcrete-ferricrete mixture.

(ii) The bearing strength drops sharply if moisture percentage increases beyond the OMC.

Aggregate Impact Values (AIV) represents the toughness against breaking down of the aggregates under the stress exerted by the wheel load impacts. This test also form a vital specification parameter in the selection of aggregates for different courses of the pavement. The obtained AIV values of calcretes and ferricretes are presented in Table 7 13 The AIV of calcretes has a wide range from as much as 47.5 to as low as 12.8 percent with a mean value of 26 7. However the AIV of ferricretes are around 25-30 percent with a mean value of 24.5 percent.

STATISTICAL ANALYSES

A number of attempts have been made in the past to correlate the strength phenomena of the pavement aggregates mainly CBR to various fundamental properties viz density, plasticity data, grain size distribution, compaction and dry density (Huekllom and Forster, 1960, Agarwal and Ghanekar, 1972; Doshi and Guiruis,1983). The aim of such attempt was to establish an empirical relationship between the strength (CBR) and the fundamental properties of aggregates

From the above studies it is evident that CBR has definite correlation with OMC and rd max for different kinds of soils. Hence, correlation and regression analyses of CBR (both soaked and unsoaked specimens) and % fine, OMC, $\gamma_{d\ max}$ for calcretes and ferricretes have also been carried out with a view to determine CBR from OMC and $\gamma_{d\ max}$.

The regression equation derived from simple arithmetic, and semilogarithmic relations is found to be insignificant even at 5% level. While, the regression coefficient derived from logarithmic relation is confident at 1% level and the regression equation is significant at 5% levels and hence, the logarithmic values are considered here for analyses and interpretation.

Multivariate correlation analyses of both soaked and unsoaked (Table 7.15, 7.16) samples indicate that CBR is influenced by % fine, OMC and $\gamma_{d\ max}$. The general observations of these studies are

(i) % of fine - $\gamma_{d\ max}$; % of fine - CBR; OMC - $\gamma_{d\ max}$; OMC - CBR have negative correlation, implying any increase in percentage of one component decreases the value of other. But for the OMC - $\gamma_{d\ max}$, all the relations are statistically insignificant

(ii) CBR has positive and significant correlation with $\gamma_{d\ max}$

(iii) The negative correlation of OMC with CBR is enhanced under the soaked conditions

Bivariate (CBR-% fine, CBR-OMC; CBR-$\gamma_{d\ max}$; OMC-$\gamma_{d\ max}$, OMC-%fine) and multivariet (CBR-% Fine-OMC-$\gamma_{d\ max}$; CBR-OMC-$\gamma_{d\ max}$) full modal regression analyses (Figure 7.6, Tables 7 17, 7 18) were carried out to estimate the influence of % fine, OMC, $\gamma_{d\ max}$ on CBR individually and cumulatively.

It is evident from the multivariate regression analysis and scatter plots that regression coefficients of logarithmic relations are statistically significant at 1% levels Hence, the derived linear regression equation would be :

$$Log_{10}\ CBR = Bo + B_1\ Log_{10}\ OMC + B_2\ Log_{10}\ \gamma_{d\,max} \text{ where,}$$

$$Bo = 22.76$$

$$B_1 = -3.74$$

$$B_2 = -5.24$$

The homogeneity/uniformity of the regression coefficients are further verified by the F-test, which is confident at 5% levels.

Further bivarient regression analysis (Table 7.18) revealed that the independent variables (% fine, OMC, $\gamma_{d\ max}$) do not have any significant influence on CBR. On the contrary, these variables cumulatively have pronounced impact on the CBR values.

Sample No	OMC (%)	Moisture content (%)	Bearing strength (lb / Inch ^{2})
3	14 0	8 0	1561
		10 0	1373
		12	1675
		15	1136 4
		18	244 9
11	13.1	11	1134 2
		14	1456.8
		17	345.0
15	11.9	10	143 8
		12	175.2
21	10 5	6	1290.1
		8	1311 7
		10	796.0
		12	626.7

TA BLE 7.14 NORTH DAKOTA BEARING STRENGTH ESTIMATES

	% Fine	OMC	γ_{dmax}	CBR
% Fine	1.0000			
OMC	0 2229	1 0000		
γ_{dmax}	- 0 3516	- 0.9510	1.0000	
CBR	- 0.6526	- 0.5593	0 5330	1 0000

TABLE 7.15 CORRELATION MATRIX FOR CALCRETES AND FERRICRETES - UNSOAKED SPECIMENS

	% Fine	OMC	γ_{dmax}	CBR
% Fine	1 0000			
OMC	0.2222	1.0000		
γ_{dmax}	- 0 3516	- 0 9510	1.0000	
CBR	- 0 5572	- 0 6418	0 6847	1 0000

TABLE 7.16 CORRELATION MATRIX FOR CALCRETES AND FERRICRETES - SOAKED SPECIMENS

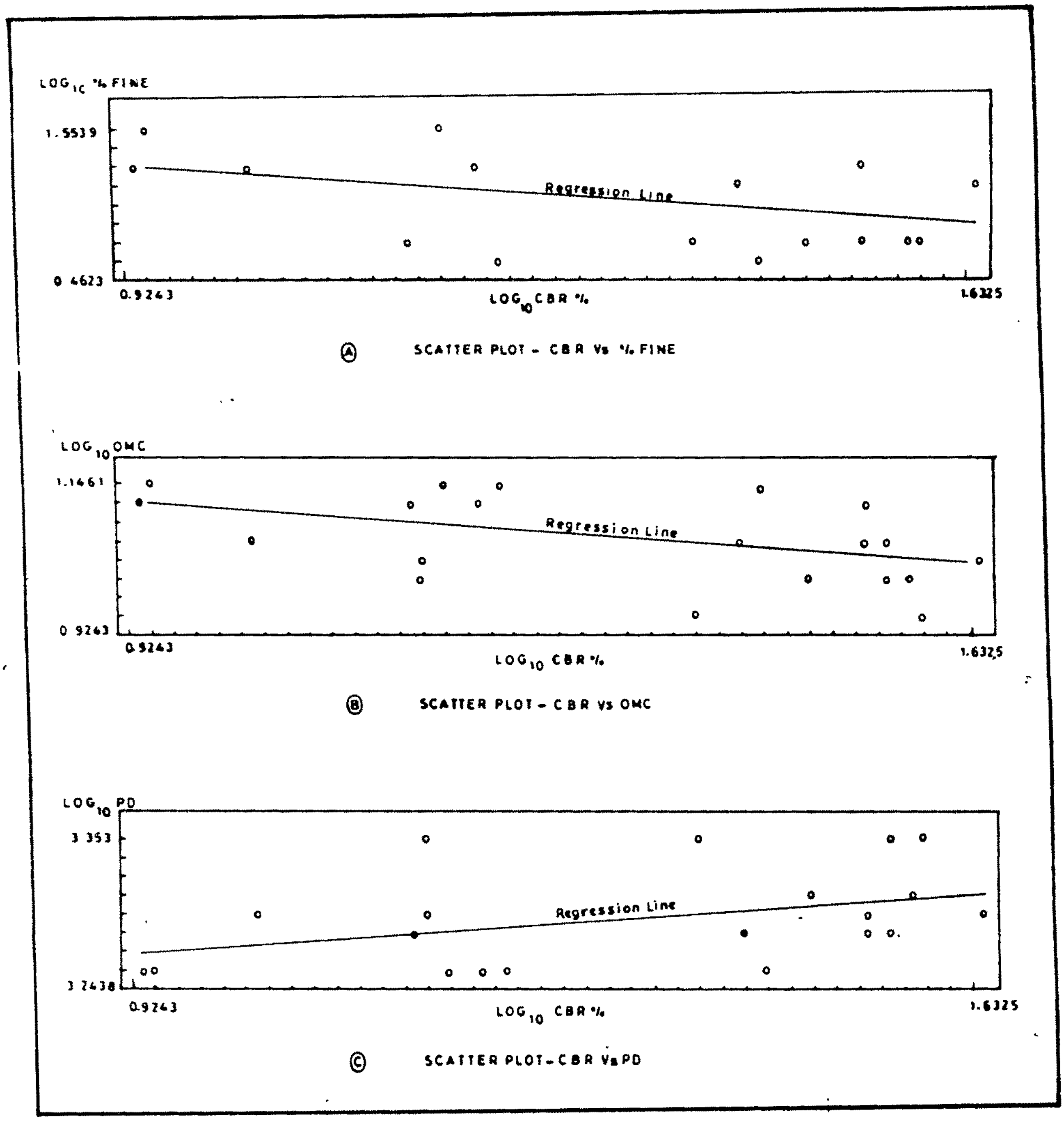


Fig. 7.6. Regression scatter plots.

Varience	Regression coeff.	R - square	D.F	F - ratio
(A) Arithmatic				
% Fine	- 0.48 (- 0.52)	0.57 (0.72)	6	2.60 (5.08)
OMC	- 5.16 (- 4.10)			
γ_{dmax}	- 0.04 (- 0.19)			
(B) Semi - Logarathmic				
% Fine	- 14.61 (- 9 07)			
% OMC	- 179.59 (- 61.93)	0.64 (0.59)	6	3.53 (2.87)
γ_{dmax}	- 271.0 (27.35)			
(C) Logarathmic				
% Fine	- 0.39 (- 0.18)			
OMC	- 3.74 ** (- 0.81)	0.68 (0.59)	6	4.33 * (2.94)
γ_{dmax}	- 5.24 ** (- 0.73)			

Note : Values within parenthesis represent corresponding values are derived for soaked specimen

* Significant at 5% levels

** Confident at 1% levels.

TABLE 7.17 MULTIVARIANT FULLMODAL REGRESSION AMONG CBR - % FINE , OMC AND MAX. DRY DENSITY

Regression coefficient	R - square	D.F	F - ratio
(A) CBR - OMC - 1 53	0.32	8	3.64
(B) CBR - γ_{dmax} 2.75	0.28	8	3 18
(C) CBR - % Fine - 0 36	0.46	8	5.94

TABLE 7.18 BIVARIENT REGRESSION ANALYSES BETWEEN CBR - OMC, MAX. DRY DENSITY AND % FINE

	CBR - range	Distributary roads 15 - 45 vehicles pass with weight exceeding 3 ton	Tributary Roads 150 - 450 vehicles pass with weight exceeding 3 ton	Trunk Roads 450-1500 vehicles pass with weight exceeding 3 ton
(A) Calcretes	15 - 30	12 - 15 cm	20 cm	24 cm
	30 - 40	8 - 10 cm	12 cm	16 cm
(B) ferricretes	30 -40	8 -10cm	12 cm	16 cm

TABLE 7.19 ESTIMATES OF SUB-BASE THICKNESS FOR WBM ROADS BY CBR METHOD

DISCUSSION

It is evident from the foregoing studies that the calcretic and ferricretic aggregates of the study area falls predominantly under the A-1 class, pointing to their gravelly nature with less percentages of fines and non plastic nature. The overall OMC percentages of calcretes although towards the higher side (more than 12%) it is still considerably lesser than the similar calcretes of Algeria, and Australia [15-28%] (Alsulami et al., 1991, Horta, 1980; Akpokodje, 1985) which are widely used in pavement construction under similar environmental conditions The prevalence of $\gamma_{d\,max}$ of the range 1753-1921 kg/m^3 achieved under the 14 percent OMC of these calcretic aggregate are reasonably better than similar aforesaid aggregates. However, it remains a puzzle for a well graded aggregate to yield relatively poor rd max. The low rd max of the calcrete is found to be attributed to two factors viz. (i) Low specific gravity of the Ca/Mg carbonates and (ii) Culmination of the coarse aggregates under compacting energy The latter feature is also fairly supported by the AIV values that have a mean value of 26 7.

Further, as a geotechnical index of actual mechanical behaviour of the soils, the values of Atterberg's limit (LL and PL) are interpreted in terms of the concentration of fines (Giddigasu et al., 1987, Saha and Chattopadhyay, 1988). This principle of use and application of Atterberg limits can not be extended to calcretic and ferricretic aggregates, due to non plastic nature of the fines.This observation has further been substantiated by the statistical analyses wherein the percent of fine doesn't have statistically significant negative impact on the CBR values.

Realizing these peculiarities of aggregates, the author has abided with the suggestion of Akpokodje (1985) that introduction of two new classes to the USC viz (I) **Calcrete Sand (SE)** and (ii) **Calcrete Gravel (GE)** to represent the arid zone calcareous soils with calcium carbonate content above 20 percent.

Ferricretes and ferricretized calcretes on the other hand have definite edge over the calcretes by virtue of higher $\gamma_{d\,max}$ (2130-2263), arrived at relatively low OMC (8.1-11 4)

But, Chandrakaran and Nambiar (1993) observed difficulties in achieving a minimum 95 percent of the laboratory γ_{dmax} in the field for such an aggregates. However, these difficulties can be overcome by adhering the methods viz multicyclic compaction (Amtosho and Akinmusuru, 1992), mechanical stabilization (Mohan and Paul, 1975) and by employing vibratory rollers (Khanna and Justo, 1991). The CBR values though are low at unsoaked conditions, it is interesting note that in contrast to general observation (Giddigasu, 1980, 1983, Giddigasu et al , 1987,) both calcrete and ferricretes of the study area shows manifold increase in the CBR percentages after soaking, implying that the strength increases after soaking This increase in strength after the soaking can principally be ascribed to the non plastic fine sandy nature of the source material that has been calcretised or ferricretized (Terzaghi and Peck, 1967). The peaked compaction curves of ferricretes and calcretised ferricretes though, point to their sensitivity to the moisture content at the time of compaction, the soaked CBR results reveal their stablility against moisture when they are compacted at OMC and γ_{dmax}

Alsulami et al , (1991) further elucidated clearly that similar calcretes having a low CBR values (5-32%) when compacted at modified Proctor compaction (2674kJ/m^3) gave CBR values between 56% and 93% This feature further establishes that the strength of the calcretes enhances manifold when they are compacted at elevated compaction energy.

The author (Ramakrishnan and Tiwari, 1996c) observed **Self-stabilization** (a property of induration after compaction and drying caused by the enhancement of bonding attributed to recrystallization of calcite) in the calcretes and calcretised ferricretes. This property is also of vital significance in improving the strength of these aggregates in a rainfall deficit arid terrains.

Compilation of the geotechnical characteristics of calcretes and ferricretes categorically point to this fact that these aggregates fulfill the gradation and AIV specifications for most of the pavement types (Tables7.1 through 7 10). However, these aggregates have so far been rejected for their use as sub-base and base courses only on account of their low CBR values, in contrast to the specified values (Table 7 3)

Identifying the special properties of these aggeregates such as well graded and nonplastic nature, increase in CBR value after soaking, enhancement of CBR at higher energies of compaction, self-stabilization characteristics, the author has realized about the potentiality of these aggregates as sub-base course and base course construction in particular, the Water Bound Macadam roads With a word of caution the author tend to conclude that *with a good compaction control (100 percent γ_{dmax}. of modified Proctor energy) these aggregates can best be utilised in the construction of sub-base courses and base courses of WBM pavements.*

ESTIMATION OF PAVEMENT THICKNESS

Realizing the efficacy of calcretic and ferricretic aggregates as road paving material for WBM roads, an attempt has been made by the author to estimate the thickness of sub-base courses, following the standard methods based on the CBR curves and North Dakota bearing strength values (Punmia, 1993). In accordance to the traffic density and wheel load, the roads of the study area have been classified in to three categories viz.

Trunk Roads	:	450 - 1500 passes of vehicle having a wheel load exceeding 3 tons
Tributary Roads		150 - 450 passes of vehicles having a wheel load exceeding 3 tons.
Distributory Roads	:	15 - 45 passes of vehicles having a wheel load exceeding 3 tons.

The proposed thickness estimated for the sub-base courses, following the CBR method of pavement thickness estimates is given in the Table 7.19 Pavement thickness evaluated by the North Dakota cone penetration values with variable moisture content and dry densities are estimated (Figure 7.7, Table 7.14). It is interesting to observe from the figure that the minimum thickness computed by this method falls at lower moisture content and density than optimum. This aspect again point to the fact that these aggregates attain sufficient strength even before γ_{dmax} is achieved. However, to avoid any risk of detrimentation to the pavement, proposed thickness for different soil sub grades compacted to 100 % of γ_{dmax} are .

Calcified soil / Powdery calcrete (Subgrade)		8 - 9 inches
Nodular calcretes (Sub-base)		3.5 - 4 inches
Ferricretes / calcretised ferricretes	:	5 inches

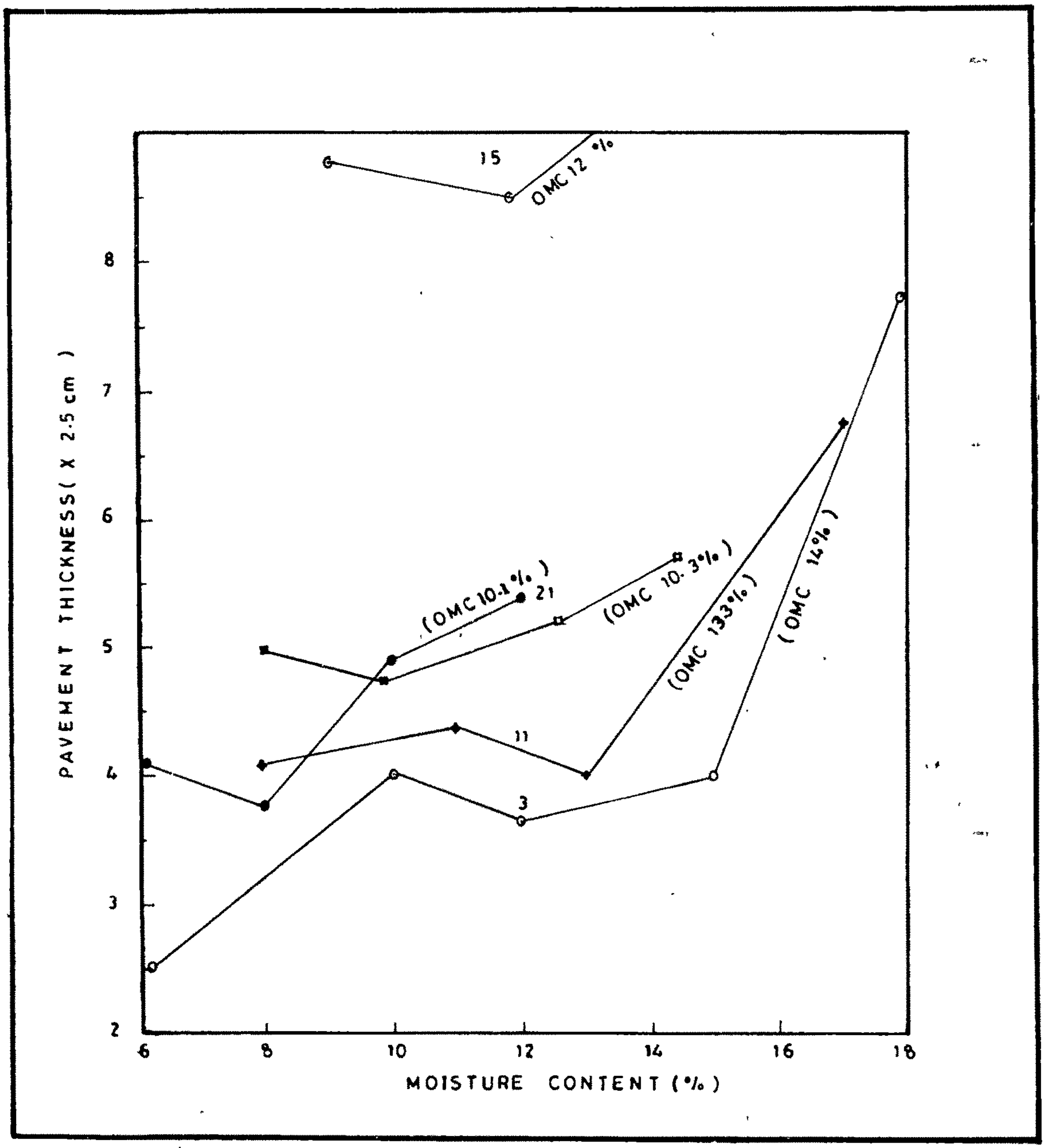


Fig.7.7. North Dakota pavement thickness estimates.