ASSESSMENT OF RECYCLED AGGREGATES FOR USE IN UNBOUND SUBBASE OF HIGHWAY PAVEMENT

By

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PUBLICATIONS

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1) **Influence of Recycled Concrete Aggregates on Specific Gravity and Absorption Rates of Mixes.** presented in' **The 26th international conference on solid waste technology and management'** in **March 27-30, 2011 Philadelphia-USA.** (Vahid Ayan, Mukesh C Limbachiya, Seyed Masoud Nasr Azadani)

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7) **'Investigation of Compaction in Recycled Aggregates for Application in Subbase Layer of Highway'** submitted on 7/11/2010 for **'The Journal of Solid Waste Technology and Management'.** (Vahid Ayan, Mukesh C Limbachiya, Seyed Masoud Nasr Azadani)

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c) PROPOSED PAPERS TO BE SUBMITTED BY 31/12/2012

11) **Evaluation of Mixtures of Recycled Concrete Aggregate and Reclaimed Asphalt Pavement Aggregate in Road Subbases.**
12) **Failure of Recycled Aggregates in Subbase Layer of Highways.**

ABSTRACT

Intendancy for sustainability has made it necessary for the highways industry to adapt its traditional processes to more cost-effective, energy efficient and greener technologies. This research programme was developed with a key aim of investigating the technical viability of aggregates formed with different combinations of Recycled Concrete Aggregate (RCA), Reclaimed Asphalt Pavement (RAP) and Natural Aggregates (NA) when used in construction of unbound subbases of highway pavements. As strongly evident from a comprehensive literature review carried out, little information is available on the application of RCA and RCA/RAP as subbase materials.

The suitability of RCA blended with RAP and NA were investigated and compared to the British Standards, Highway Agency specifications and the AASHTO standards requirements for highway design. Having established their suitability for highway design, then the performance of the materials was assessed under traffic loading.

In compliance with the requirements of the above Standards/Specifications the in-situ loaded behavior of the aforementioned materials were also investigated. This was achieved through numerical analysis of a typical pavement structures comprising subbases made of the above materials. As a consequence, a series of analyses were carried out using KENLAYER™ computer program to model the stresses and deformations in the subbases. The results of the analyses were then applied to the Mohr Coulomb failure model in order to predict the factor of safety against failure of the subbase layer.

Amongst the salient findings from the research was that most of the materials tested complied with the standard requirements. From a large number of tests carried out on materials collected in the UK and Iran, it was also demonstrated that the source of a recycled material and the method of extraction had a significant influence on the engineering properties of the material, especially the CBR. In terms of compaction and CBR requirements, the 50%RAP+50%RCA mix was demonstrated as suitable for unbound subbase application. Also, the presence of RAP in the mixes of RAP/RCA was found to improve the drainage properties of an unbound subbase layer.

From the viewpoint of durability and frost susceptibility, it was verified that all the materials investigated were applicable to the highest significance level (as defined by AASHTO). However, based on stiffness considerations, the materials were found to be applicable only to the lowest significance level. Now, turning to the toughness and shear strength properties, the same materials were found to fall in the middle significance levels. The results of KENLAYER™ modeling indicated that the safety factor against failure of the mixes containing RAP and RCA decreased as the stiffness decreased.

Evidently the research is likely to have a number of implications on the design and construction of highways. Firstly, the demonstrated viability of mixes containing recycled materials (RCA+RAP) can lead to significant cost savings, reduced CO_2 emissions, reduced exploitation of virgin materials and minimization of dumping of civil engineering waste. If embraced in industry, the suggested use of recycled materials can help support National and international targets regarding sustainability, environmental and energy conservation. The research succeeded in cataloguing the specifications of RCA/RAP and RCA/NA for practical works, followed by development and evaluation of different mixes of recycled aggregates obtained from different sources. Detailed assessment based on compliance with requirements, performance prediction and modeling was completed.

KEY WORDS

Highway Pavements, Recycled Concrete Aggregate, Recycled Asphalt Pavement, Natural Aggregates, Subbase, Performance Related Tests, Numerical Modeling, KENLAYER™ Program.

USED ACRONYMS/ABBRIVIATIONS

WRAP	The Waste and Resources Action Programme
WRAC	Waste Reduction Advisory Committee
HA	Highways Agency
RCA	Recycled Concrete Aggregate
RA	Recycled Aggregate
RAP	Reclaimed Asphalt Pavement
RCP	Reclaimed Concrete Pavement
RCM	Recycled Concrete Material
NA	Natural Aggregate
VA	Virgin Aggregate
HBM	Hydraulically Bound Mixture
HMA	Hot-Mix Asphalt
MCHW	Manual of Contract Documents for Highway Works
DMRB	Design Manual for Roads and Bridges
BS	British Standard
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
NCHRP	National Coopràtive Highway Research Program
SHW	Specification for Highway Works
NFG	Notes for Guidance
IANs	Interim Advice Notes
TRL	Transport Research Laboratory
C&D	Construction and Demolition
CRD	Construction, Renovation and Demolition
GGBS	Ground Granulated Blast Furnace Slag
PFA	Pulverized-fuel Ash
CoURAgE	Construction with Unbound Road Aggregate in Europe
CB	Clay Brick
CG	Concrete Gravel
MG	Mixed Granulate
G.B.C	Granular Base Course
RCC	Recycled Crushed Concrete

PCC	Portland Cement Concrete
CBM	Cement-Based Material
CBR	California Bearing Ratio
RLTT	Repeated Load Triaxial Testing
SSD	Saturated Surface Dry
FRCA	Fine Recycled Aggregate
OMC	Optimum Moisture Content
MDD	Maximum Dry Density
LAA	Los Angeles Abrasion= LA Abrasion
UCS	Unconfined Compressive Strength
TST	Tube Suction Test
DOC	Degree of Compaction
DOT	United State Department of Transportation (e.g. FDOT, SDDOT)
M_r	Resilient Modulus
FWD	Falling Weight Deflectometer
PRIMA	Portable FWD
VCS	Visual Condition Surveys
SCRIM	Sideway Force Coefficient Routine Investigation Machine
SSG	Soil Stiffness Gauge
F/T	Freezing and Thawing
ACV	Aggregate Crushing Value
AIV	Aggregate Impact Value
CEI	Construction Energy Index
URC	Jointed Unreinforced
JRC	Jointed reinforced
CRCP	Continuously reinforced
CRCR	Continuously Reinforced Concrete Road-base
FC	Foundation Class
$C_{90/3}$	Category for percentage of crushed or broken particles
M_{DE}	Category for Maximum Value of Resistance to Wear, Micro-Deval Coefficient
MS_{35}	Category for Maximum Magnesium Sulphate Soundness
I_p	Plasticity Index

NP	Non-plastic
PL	Plastic Limit
LL	Liquid Limit
ρ_{rd}	Oven-dry Particle Densities
ρ_{ssd}	Saturated and Surface-dry Density
ρ_a	Apparent Particle Density
WA_{24}	Water Absorption Rates for 24 h
ρ_d	Dry Density
ρ	Bulk Density
ρ_s	Particle Density
ρ_w	Density of Water
G_{sb}	Bulk Specific Gravity
G_{sb} SSD	Bulk SSD Specific Gravity
G_{sa}	Apparent Specific Gravity
Abs	Water Absorption
ϕ'	Effective Angle of Internal Fraction
c′	Effective Cohesion
σ_d	Deviator Stresses
SE	Sand Equivalent Value
C_s	Cement Percentage
SS	Sodium Sulphate Soundness
MS	Magnesium Sulphate Soundness
FG	Frost Group
σ_1 and σ_3	Major and Minor Principal Stresses
F	Safety Factor

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CHAPTER 1 INTRODUCTION

1.1 Sustainability in Highways

Over the past 20 years, the UK construction industry has been pushed to update its methods of design, specification and construction to meet the sustainability aims of the Government. Taxes on primary quarried aggregates and waste disposal to landfill have been introduced in pursuit of goals of increasing the recycling and reuse of materials. The transportation, particularly the Highway Agency, continues to respond to these goals by developing new Standards and guidelines. A notable development has been the move towards design methods that apply performance based parameters of materials (Lambert, 2007).

It has been recognized that many recycled materials can be used as constituents of unbound mixture for road pavements. These materials are defined within the European and British standards and specifications as illustrated in the definitions Table 1.1 (WRAP, 2005):

Table 1.1 Definition of waste (WRAP, 2005)

RCA	A designation used in BS 8500 for recycled aggregate principally comprising crushed concrete.
RAP	Recycled aggregate consisting of crushed or milled asphalt. This may include millings, planings, returned loads (haul-back), joint offcuts and plant waste.

Waste is referred to by previous researches as any substance or objects that the holder discards, intends to discard or is required to discard. As a result of European and national case law over the last few years, the circumstances under which a substance or object may be said to have been discarded (or to be intended or required to be discarded) have broadened considerably. The quality protocol of The Waste and Resources Action Programme (WRAP) provides support to Environment Agency in taking decision for identifying the waste materials i.e. if all the criteria specified in this protocol are met, then it would indicate that the material is probably no longer waste (WRAP, 2005).

For the present research, a comprehensive literature research was carried out and revealed that studies on the usage of RCA in highway subbases are so far limited.

The primary aim of this research was to investigate the performance suitability of Recycled Concrete Aggregate (RCA) independently and also its mixtures with Reclaimed Asphalt Pavement (RAP) aggregates and Natural Aggregates (NA) when used in construction of unbound subbases of highway pavements. Various mixes of RCA/RAP and RCA/NA were studied in order to establish their suitability for highway subbases and to benchmark their characteristics against relevant British Standards, Highway Agency Specifications and AASHTO standards governing highway pavement design. Based on the measured material properties their performance under traffic loading was also assessed.

In developing the research further, the behavior of the test materials in a real pavement structure was also investigated based on numerical modeling with KENLAYER^{TM} computer program, with systematically varied parameters.

1.2 Usage of RCA and RAP in Highway Construction

1.2.1 The definition and use of RCA

Recycled concrete aggregate is aggregate resulting from the processing of inorganic material previously used in construction and principally comprising crushed concrete. Based upon the Specification for Highway Works (MCHW Volume 1, Series 800) and the Design Manual for Roads and Bridges (HD 35/04), Recycled concrete aggregates can be used in (Aggregain, 2011a):

- Bitumen bound materials – recycled concrete aggregate can be used in a variety of base course and binder course applications.
- Concrete – recycled concrete aggregate is permitted for use in certain grades of concrete.
- Pipe bedding – suitably graded Recycled concrete aggregate is used in pipe bedding.
- Hydraulically bound mixtures (HBM) for subbase and base – recycled concrete aggregate can be suitable for use in HBMs.
- Unbound mixtures for subbase – suitably graded recycled concrete aggregate are used as subbase.
- Capping – recycled concrete aggregate is suitable for capping applications.

- Embankments and Fill – suitably graded recycled concrete aggregate is used in these applications.

1.2.2 The definition and use of RAP (Aggregain, 2011b)

Recycled asphalt can comprise millings, planings, return loads and offcuts from bituminous layer joint preparation. Asphalt planings are defined as materials derived from the layers of the pavement using a mobile machine fitted with milling cutters. Granulated asphalt is defined as asphalt bound material recycled from roads under reconstruction or surplus asphalt material destined from bound pavement layers, but unused, which has been granulated (HD 35/04).

Based upon the Specification for Highway Works (MCHW Volume 1, Series 800) and the Design Manual for Roads and Bridges (HD 35/04), recycled asphalt can be used in:

- Bitumen bound materials – base, binder and surface courses (up to 10% by mass Recycled asphalt may be used in surface courses and up to 50% in all other layers). Asphalt can be recycled back into hot asphalt, a process which gains the benefit from the original bitumen and high quality aggregate; or into cold lay foamed bitumen, which is growing in popularity.
- Concrete – can contain up to 5% asphalt as a foreign material, but recycled asphalt is generally not viewed as a concreting aggregate.
- Pipe bedding – where permitted, up to 100% recycled asphalt may be used.
- Hydraulically bound mixtures (HBM) for subbase and base – up to 100% recycled asphalt.
- Unbound mixtures for subbase – 50% recycled asphalt can be used in subbase or 100% if permitted by the contract specification.
- Capping – 100% Recycled asphalt can be used in subbase.
- Embankments and Fill – where permitted, up to 100% recycled asphalt may be used.

1.3 Application of Recycled Materials at a Glance

Construction and demolition (C&D) work constitutes a large proportion of construction debris producing from solid waste. Table 1.2 is a comparison of the proportions of construction solid waste for some selected countries. In 1995, in the United Kingdom, construction activities produce more than 50% of waste deposited in a typical landfill. While about 70 million tons of wastes are produced from C&D projects annually. In Australia, about 14 million tons of wastes are discarded to landfill each year, in which around 44% of wastes come from construction projects. In the United States of America, about 29% of solid-waste is attributed to construction industry, however in Hong Kong, it is about 38% (Tam *et al.*, 2007c).

Among the various types of construction materials in Hong Kong, concrete seems to be the most significant element, as seen from the following list (Tam *et al.*, 2007c):

(a) Construction sites: 75%
(b) Demolition sites : 70%
(c) General civil work : 40%
(d) Renovation work : 70%

Environmental Resources Limited (ERL, 1988) showed that more than 20 million tons of demolition debris is stockpiled at landfill sites in the UK annually. Concrete and masonry with the volume of 50-55% and 30-40% are the most significant part of this material, which may also comprise small percentages of other materials such as metals, timber and glass. According to Table1.2, as much as 40% of demolition materials in the UK is recycled, albeit mainly for low grade and non-structural uses such as fills and hardcore. Higher grade application such as concrete is not widely encouraged because of a lack of proper experience and Standards/specifications (Tam *et al.*, 2007d).

In 1994, in the Netherlands, around 14 million tones of building and demolition waste were produced annually, or which almost 8 million tones were recycled (i.e. 57%), mainly for application in unbound road base aggregates (Tam *et al.*, 2007d). As seen in Table 1.2 this figure was increased by 75% after 2000.

Table 1.2 Comparison of proportions of construction solid waste (Tam *et al.*, 2007c) (Data cited from different paper after 2000 by Tam *et al.*, 2007c)

Country	Proportion of construction waste to total waste (%)	C&D waste recycled (%)
Australia	44	51
Brazil	15	8
Denmark	25-50	80
Finland	14	40
France	25	20-30
Germany	19	40-60
Hong Kong	38	No information
Japan	36	65
Italy	30	10
Netherlands	26	75
Norway	30	7
Spain	70	17
United Kingdom	Over 50	40
United States of America	29	25

In Table 1.3, various causes of building waste and the reasons for occurrence of waste are listed under two main areas, i.e.: (a) site management & practice and (b) transportation of products.

Table 1.3 Causes and examples of building waste on site (Tam *et al.*, 2007c)

Building waste	Causes of building waste on site	Examples
Site management and practice	Lack of quality management system aimed at waste minimization	Lack of waste management plan
	Untidy construction sites	Waste materials are not segregated from useful materials
	Poor handling	Breakage, damage, losses
	Over-sized foundations and other elements	Over design leads to excess excavation and cut-offs
	Inadequate protection to finished work	Finished concrete staircases are not protected by boarding
	Limited visibility on site resulting in damage	Inadequate lighting in covered storage area
	Poor storage	Pallet is not used to protect cement bags from contamination by ground water
	Poor workmanship	Poor workmanship of formwork
	Waste generation inherited with traditional construction method	Timber formwork, wet trade
Delivery of products	Over-ordering	Over ordering of concrete becomes waste
	Method of packaging	Inadequate protection to the materials
	Method of transport	Materials drop from froklift
	Inadequate data regarding time method of delivery	Lack of records concerning materials delivery

Table 1.4 indicates the sales of aggregate in terms of the application in the UK. It is seen that the uncoated roadstone (unbound subbase) has been sold in the UK about 23% of whole road aggregates (ONS Construction Statistics). WRAP believes that the subbase share will have grown slightly by more than 25%(An email from WRAP)

Table 1.4 Sales[1] of aggregate by use in Great Britain: 2008 (ONS Construction Statistics)

Crushed rock **Thousand tonnes**

	Roadstone									
Regions[2]	**For asphalt on site**	**For asphalt off site**	**Uncoated roadstone**	**Surface dressing chippings**	**Concrete aggregates**	**Other screened and graded**	**Rail ballast**	**Other constructional uses**	**Armourstone & gabion**	**Total**
North East	..	..	..	..	679	481	..	1,444	..	4,928
Yorkshire & Humberside	..	1,367	2,342	162	2,644	1,292	-	2,024	..	10,045
East Midlands	2,118	1,435	6,368	..	5,359	2,895	..	3,924	85	24,328
East of England	-	-	11	-	..	-	-	..	-	683
South East	-	-	..	-	..	..	-	..	..	1,029
South West	2,091	2,076	4,397	..	6,360	1,510	..	5,431	63	22,355
West Midlands	1,030	..	1113	..	..	..	..	748	..	3,850
North West	410	1010	1,212	..	1,536	1,352	-	2,212	..	7,962
England	6,233	6,968	16,582	845	16,928	7,756	2,248	17,167	450	75,179
Wales	..	2,108	1,757	250	3,087	1,642	..	5,057	45	15,685
Scotland	..	1,080	7,310	109	2,981	5,639	..	4,331	218	24,215
Great Britain	9,361	10,156	25,649	1,204	22,995	15,036	3,408	26,555	713	115,079

Notes

.. = not available; for reasons of disclosure these figures are not quoted.

\- = nil or less than half the final digit shown.

1. Sales are allocated to either the region of the production site or to the region of landing for marine-dredged sand & gravel.
2. Data are presented on a Government Office Region basis.

1.4 Pavement Structure

A pavement structure essentially comprises a foundation and either bituminous upper layers (in flexible pavements) or cement-bound upper layers (in rigid pavements). Having said that, composite pavements are also used may also be used. The pavement foundation must have adequate stiffness and provide resistance to permanent deformation, both during construction (short-term) and service (medium to long-term). It must also reduce the stresses transmitted to the subgrade to prevent excessive deformation in this bottom component of the pavement. The road foundation typically combines the subgrade and unbound granular or cement/hydraulically bound layers, comprising a capping layer (if needed) and the subbase layer (Lmbert, 2007). As previously mentioned the unbound subbase layer is the principal focus of this research.

1.5 Scope of Thesis

Much research has been carried out on the application of recycled aggregates, (especially RAP and RCA) in bound layers there is still limited knowledge on the application of these materials when it comes unbound layers. This research seeks to investigate the technical viability of applying RAP and RCA in unbound subbase layers and benchmark their characteristics against the requirements of the following Standards:

(i) UK Highways Agency (HA) Specifications (i.e. The Design Manual for Roads and Bridges-DMRB, the Manual of Contract Documents for Highway Works-MCHW, the Specification for Highway Works – SHW

(ii) British Standards (there are several relevant ones which will be discussed later)

(iii) American Association of State Highway and Transportation Officials-AASHTO.

Performance assessments were carried out based on a large number laboratory tests conducted on RCA, RAP and NA, supplemented with numerical modeling of pavements under standard loading conditions representing in-service performance of pavements.

1.6 Research Aims and Objectives

The overall aim of this research is to develop mixtures of recycled materials which are technically viable and fulfill the requirements of current design standards. To demonstrate the success of the research, the subject materials are thoroughly evaluated in order to verify their practical performance as highway subbase materials. The specific objectives of the work are as follows:

- Cataloguing and interpretation of current specifications of RCA and RAP, including designing of a range of mixes
- Characterizing the NA, RCA and RAP mixes based on HA and AASHTO designations
- Prediction of the performance of a typical pavement constructed from the material mixes, based on toughness, stiffness, durability, frost susceptibility and shear strength
- Numerical modeling of hypothetical subbases made of the mixes being investigated, in order to predict the deflections, stresses and strains at various points, under standard wheel loads.
- Comparing the test mixes based upon an established damage model and failure criteria

1.7 Structure of the Thesis

The thesis is structured as described below to define the content and purpose of each chapter of the thesis.

Chapter 1: describes the background and rationale of using recycled materials in highways, in order to develop a justification for the research. It then defines the scope, aims and objectives of the thesis.

Chapter 2: presents an extensive literature review on granular recycled aggregates, discusses published experimental data including making comparisons and gives a commentary on the current level of knowledge relevant to this work.

Chapter 3: outlines and reviews possible research methodologies and analyses those considered to be most appropriate for this research project.

Chapter 4 reviews various subbase construction materials and methods and explains the sources of the test materials and how they were processed and prepared for the intended experiments.

Chapter 5 illustrates the experimental process and presents the results for aggregates tested from two sources: (i) the United Kingdom and (ii) Iran.

Chapter 6 presents the performance evaluation of mixes based on toughness, stiffness, durability, frost susceptibility and shear strength.

Chapter 7 describes quantitative predictions, using KENLAYER™ computer program, of pavements made with various combinations of the tested materials and then compares the results with various failure criteria.

Chapter 8 summarizes the overall results and gives recommendations for future research.

Appendix A presents the large shear box test results.

Appendix B presents the results of KENLAYER™ computer programme.

Appendix C presents the details of a recommendation for future work.

CHAPTER 2 LITRATURE REVIEW

2.1 Sustainable Highways

A widely acceptable definition of sustainable development is "development that meets the needs of the present without compromising the ability of future generations to meet their own needs". It has been a major part of UK government policy for over a decade, at first through the UK strategy for sustainable development, followed by the UK strategy for sustainable construction. The last version of this policy was reviewed, and an updated strategy was published in 2006 and 2008, respectively. The UK national sustainable development strategy was renewed in 2005, and the developed administrations have improved their own regional sustainability strategies which reflect the following priority areas of the UK national strategy:

- Sustainable consumption and production;
- Climate resource protection and environmental enhancement; and
- Sustainable communities.

Highway maintenance and new construction will influence directly on these priorities:

- These activities consume large amounts of construction materials and produce large amounts of waste.
- Major sources of greenhouse gas emissions are: extraction, processing and transportation of materials such as cement and asphalt production.
- Using natural aggregates instead of recycled or secondary aggregates reduces the stock natural resources and contributes to environmental degradation.
- Incorrect application of materials can cause environment pollution.

To achieve sustainable highway maintenance and construction, great emphasis must be attached to recycling of materials from existing roads, utilisation of secondary aggregates and selection of methods which minimize greenhouse gas emissions. These principles are closely connected with the waste hierarchy (Figure2.1) that is the basis of all UK and European legislation on waste material:

- To reduce the amounts of waste produced.
- To reuse products wherever possible.
- To recycle what cannot be reused.
- To recover energy from waste that cannot be reused or recycled.

- To dispose of materials only as a last solution.

Highway engineers and all parties involved in a given project have a duty to ensure the highest possible degree of sustainability in highway maintenance and construction. This requires careful choice of materials and corresponding methods of construction. Sustainability in highway construction and maintenance means working with environmental limits while attaining a sustainable economy (Reid *et al.* 2008, pp.6-7).

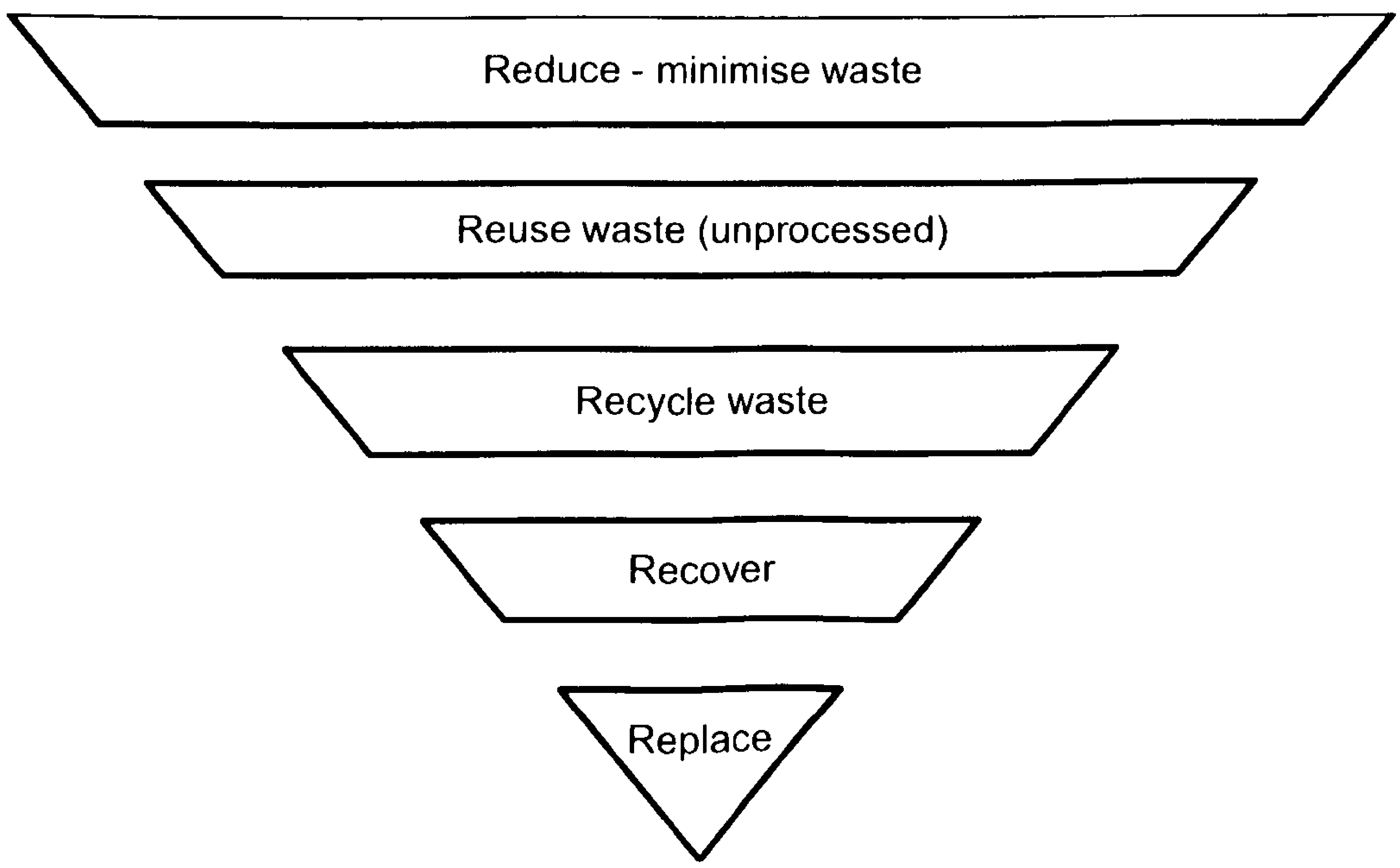


Figure 2.1 The waste hierarchy (After Reid *et al.* 2008, pp.6-7).

2.1.1 Materials, methods and milestones (Reid *et al.* 2008, pp.13-19).

In general, choosing the sustainable materials for highway works should meet the order of priority below:

- reuse of existing highway materials at the suitable level like the same level as existing materials or at the highest possible level which is reasonable in practice;
- use of recycled or secondary materials imported from other project as much as possible if they are acceptable technically in practice;
- use of the primary materials

Chosen methods should meet this order of priority, for example use of in-situ or ex-situ cold recycling of pavements instead of reconstruction with primary materials, or use of lime or cement to stabilize soils in situ instead of excavation and replacement with natural aggregates for capping or subbase layer of highways.

To help in estimating the level of sustainability achieved in a scheme, a series of milestones have been suggested. There is a direct relationship between the milestone number and the overall sustainability of the scheme. To achieve the higher milestone number, greater effort will be needed. The milestones of any scheme are added cumulatively, for example, Milestone 2 includes the measures in Milestone 1, etc. They are described below:

Milestone 0 is application of standard techniques which meet the requirements of highway maintenance purpose, excluding any use of recycled or secondary materials.

Milestone 1 is application of the same techniques as Milestone 0 but with replacement of recycled or secondary materials for primary materials in low-risk and safe applications, such as unbound applications, footways and cycle tracks. In this milestone, recycling techniques that are technically sound and of low risk are used. Examples here include cold recycling of pavements and footways.

Milestone 2 includes Milestone 1 and use of some new techniques and materials that allow use of recycled or secondary materials in the applications which are higher-value but less common than those of Milestone 1. For example the use of high amounts of reclaimed asphalt in new hot mix asphalt, utilization of recycled aggregate in low-strength ancillary concrete or use of hydraulically-bound materials (HBM) with binders except cement for subbase.

Milestone 3 includes Milestones 1, 2 and also includes use of innovative techniques and materials that may or may not be new and might need specialist contractors and designers; these are a little higher-risk than ordinary techniques but allow use of recycled or secondary materials in high-value utilizations, e.g. use of HBM with binders except cement in the base course or recycling of surface course into new surface course.

The aim is to develop the performance of Local Authorities from Milestone 0 to Milestone 3 so as to gain more experience and confidence in working with recycled and secondary materials. Although the milestones should be achieved in the order described above, this does not necessarily mean that Milestone 2 has to be fully achieved before progressing to Milestone 3. Also, making progress in Milestone 2 and 3 actions should not cause delay in achieving Milestone 1, which should be completed, because it is urgent.

The activity required for highway construction and maintenance works has been divided into six applications. There are a number of actions classified under the four milestones, for all applications. The use of different materials, maintenance techniques and the production and reuse of waste are seen in each action. For each milestone, the actions are different among the applications, showing the relative ease with which they can be achieved (Tables 2.1 to 2.5).

Table 2.1 Surfacing (After Reid *et al.* 2008, p.14)

Application	Related Works	Milestone	Action
Surfacing	Surface dressing Slurry surfacing Inlay and overlay Replacement or renewal of surface course New surface course	0	Use primary aggregates and hot asphalt for all applications. Dispose of arisings to landfill or exempt site.
		1	In-situ hot recycling the repave or remix processes. Use materials such as steel slag, if it is available, economic and meets requirements. Recycle arisings as unbound subbase or capping.
		2	Retexture surfacing to avoid having to replace it. In-situ cold recycling using the retread process. Recycle arisings into new surface course at 10 per cent.
		3	Collect surface dressing sweepings and surplus and reuse in new surface dressing. Recycle arisings into new surface course at >10 per cent. Recycle thin surfacing into new thin surfacing.

Table 2.2 Pavement reconstruction (After Reid *et al.* 2008, p.15)

Application	Related Works	Milestone	Action
Pavement reconstruction	Reconstruction of structural (base and binder) course of pavement; Construction of new base and binder course.	0	Use primary aggregates and hot asphalt or concrete for all applications. Dispose of arisings to landfill or an exempt site.
		1	Cold in-situ or ex-situ recycling of existing bituminous or concrete pavement. Use up to 30 per cent reclaimed asphalt in new hot asphalt. Recycle arisings as unbound sub-base or capping.
		2	Use greater than 30 per cent reclaimed asphalt in new hot asphalt. Use recycled or secondary aggregates in new hot asphalt if it is available, economic and meets requirements. Use crack and seal for concrete pavements. Use saw cut and seal for bituminous overlays on concrete pavements.
		3	Use HBMs with binders other than cement (e.g. pulverized fuel ash or slag) as a combined base and subbase with recycled aggregates. Improved foundations using HBMs may allow reduction in thickness of upper layers. Use recycled or secondary aggregates in new pavement quality concrete if it is available, economic and meets requirements. Separate surface course during planning or break-out and recycled in new surface course.

Table 2.3 Edges (After Reid *et al.* 2008, p.16)

Application	Related Works	Milestone	Action
Edges	Repairs to existing verges and haunches (rural roads); Haunching to widen rural roads; Repairs/replacements to verges, kerbs and drainage.	0	Use primary aggregates/concrete/hot asphalt for all applications. Dispose of arising to landfill or exempt sited.
		1	Use recycled aggregates (e.g. planings) for unbound applications. Send arisings to recycling centre.
		2	Use lime or cement to stabilize existing soils and avoid need for imported capping. Use up to 30 per cent reclaimed asphalt in new hot asphalt. Use recycled aggregates in ancillary concrete for kerb bedding and backing. Use lightweight kerbs. Segregate arisings on site into asphalt, concrete and soil and send to recycling centre for higher-value applications.
		3	Use lime and/or hydraulic binders to stabilize existing soils and avoid need for imported subbase and base. Use greater than 30 per cent reclaimed asphalt in new hot asphalt. Use recycled or secondary aggregates in new hot asphalt if available, economic and meet requirements. Use recycled or secondary aggregates in new pavement quality concrete if available, economic and meet requirements. Use reclaimed bricks to support gullies and manholes.

Table 2.4 Capping and sub-base (After Reid *et al.* 2008, p.18)

Application	Related Works	Milestone	Action
Capping and subbase	Repairs to existing capping and subbase; Capping and sub-base in new construction.	0	Use primary aggregates for all applications. Dispose of arisings to landfill or exempt sites.
		1	Use recycled aggregates for unbound applications. Send arisings to recycling centre. Incorporate some of existing subbase in full-depth cold recycling of existing pavement.
		2	Use lime and/or cement to stabilize existing soils and avoid need for imported capping (beware of sulfates/sulfides). Segregate arisings on site into asphalt, concrete and soil send to recycling centre for higher-value applications. Replace subbase with HBMs (other than cement).
		3	Use lime and/or cement to stabilize existing soils and avoid need for imported subbase (beware of sulfates/sulfides). Use HBMs (other than cement) as a combined base and sub-base with recycled aggregates and secondary binders.

Table 2.5 Footways and cycle tracks (After Reid *et al.* 2008, p.17)

Application	Related Works	Milestone	Action
Footways and cycle tracks	Repairs to existing footways and cycle tracks; Construction of new footways and cycle tracks.	0	Use primary materials and hot asphalt for all applications. Dispose of arisings to landfill, exempt sites or farmers' tracks.
		1	Use cold recycled bitumen bound material for repairs to existing footways for subbase, base and binder course. Use recycled aggregates as unbound subbase. Send arisings to recycling centre for use in other as cold recycled bitumen bound material- closed loop system.
		2	Use quarry fines for repairs and new construction of surfacing for footpaths and cycle tracks where these are locally available and environmentally appropriate. Add cement where necessary. On-site hot recycling of asphalt footways into new hot asphalt using mobile plant. Use lime and/or hydraulic binders to treat weak sub-grade materials rather than excavate and replace with imported granular materials.
		3	Use lime and/or hydraulic binders to treat existing soils and avoid need for imported subbase and base.

2.1.2 Specification and quality control (Reid *et al.* 2008, p.22-24)

Concern about quality and durability is one of the principal reasons for not using recycled and secondary materials. This is covered by means of appropriate specifications and quality control protocols. In the past, many specifications did not allow the application of recycled and/or secondary materials and recycling techniques, and some engineers had to think that this is still the case. In fact, all the common specifications have been significantly updated over the last ten years. This is why most of these specifications, now permit the use of recycled materials and recycling techniques, because they meet quality and performance requirements.

The most commonly used documents in the UK are (i) The Manual of Contract Documents for Highway Works (MCHW), (ii) The Design Manual for Roads and Bridges (DMRB). Both of these documents are jointly published by The Highways Agency (HA), Transport Scotland, The Welsh Assembly Government and The Northern Ireland Department for Regional Development. The DMRB is made up of separate volumes with each being divided into a number of sections. Most sections are further sub-divided into parts. Documents are allocated to a volume, section and (for documents issued since 1992) part according to their subject matter. Each document is given a reference number prefixed by a series code, i.e. BD, BA, GA, GD, HD, HA, TD, TA, nominally: "B" for bridges and structures; "G" for general; "H" for highways; and "T" for traffic engineering and control, followed by: "D" for Design Standard; or "A" for Advice Note. Section 1 of Volume 7 of DMRB presents a document HD 35/04 which is of particular importance to this PhD research. HD 35/04 deals with conservation and the use of secondary and recycled materials. Table 2.6 cited from HD 35/04 summarizes the applications for which a range of recycled and secondary materials may be used. If the materials comply with the requirements of the SHW then the Table 2.6 can be used as an initial guide as to whether the materials would be suitable in local authority highway works.

Volume 1 of the MCHW constitutes the Specification for Highway Works (SHW), which contains the requirements and approvals procedures for work, goods or materials used in the construction, improvement or maintenance of the Trunk Road network. Volume 2 the MCHW contains Notes for Guidance (NFG) on the Specification of Highway Works and gives advice and guidance in the implementation of the specifications given in Volume 1, as well as examples of tables and appendices necessary for the contract.

Another relevant document in use is the Interim Advice Notes (IANs), which are issued by HA from time to time. IANs are not part of DMRB or MCHW but contain specific guidance to be used in connection with works on motorways and trunk roads in England, subject to any specific implementation instructions contained within the IAN. IANs must be read in conjunction with the DMRB and MCHW and may incorporate amendments or additions to documents in these manuals.

The aforementioned documents are updated regularly and contain frequent references to (a) Transport Research Laboratory (TRL) Reports, (b) British Standards and (c) Euro-norms, the last two of which mainly focus on natural aggregates. However, some clauses may be included on the application of recycled or manufactured aggregates.

Information related to the use of recycled and secondary aggregates according to a number of specifications, including the SHW, is given in the Specifier section of the AggRegain website. The website also contains many case studies on the use of recycled and secondary aggregates in Local Authority highway works.

Assurance about the quality of recycled and secondary materials can be gained by the use of quality protocols, which present that the suppliers have required procedures to ensure the consistency and quality of their products. The Waste and Resources Action Programme (WRAP) Quality Protocol for the production of aggregates from inert waste is an example for these kinds of protocols. WRAP has special versions for England and Wales, Scotland and Northern Ireland are available. These protocols cover wide range materials likely to come from highway and footway works, including asphalt plants, concrete, brick, soil and stones. The use of recycled aggregates according to SHW depends on the supplier using the WRAP Quality Protocol. Local authorities should determine these conditions of the use of recycled aggregates in their own highway works and requires their suppliers to apply these protocols

Environment Agency and WRAP currently develop the waste protocols for a number of secondary materials that could be used in highway works, including pulverized-fuel ash, contaminated soils and shredded or crumbed tyres.

Many suppliers of recycled and secondary aggregates, such as steel and ground granulated blast furnace slag (GGBS), pulverized-fuel ash (PFA) and incinerator bottom ash aggregate, have developed protocols for their products to ensure that they are consistent with high quality. Many have also produced some guidelines on the application of their products in road construction and other applications. Recycled and secondary aggregates should only be used in projects that the producer can present evidence proving the quality and consistency of the materials.

Table 2.6 application of secondary and recycled aggregates

(After Reid *et al.* 2008, p.24)

Application and series	**Pipe bedding**	**Embankment and fill**	**Capping**	**Unbound mixtures for subbase**	**Hydraulically bound mixtures for subbase and base**	**Bitumen bound layers**
Material	**500**	**600**	**600**	**800**	**800**	**900**
Blastfurnace slag	✓	✓	✓	✓	✓	✓
Burnt colliery spoil	✗	✓	✓	✓	✓	✗
China clay sand/stent	✓	✓	✓	✓	✓	✓
Coal fly ash/pulverized fuel ash (CFA/PFA)	✓	✓	✓	✓	✓	✓
Foundry sand	✓	✓	✓	✓	✓	✓
Furnace bottom ash (FBA)	✓	✓	✓	✗	✓	✗
Incinerator bottom ash aggregate (IBAA)	✓	✓	✓	✓	✓	✓
Phosphoric slag	✓	✓	✓	✓	✓	✓
Recycled aggregate (RA)	✓	✓	✓	✓	✓	✓
Recycled asphalt (RAP)	✓	✓	✓	✓	✓	✓
Recycled concrete (RCA)	✓	✓	✓	✓	✓	✓
Recycled glass	✓	✓	✓	✓	✓	✓
State aggregate	✓	✓	✓	✓	✓	✓
Spent oil shale	✗	✓	✓	✓	✓	✗
Steel slag	✓	✓	✓	✓	✓	✓
Unburnt colliery spoil	✗	✓	✗	✗	✓	✗

2.2 Efforts to Utilisation of Recycled Materials in Road Construction

Solid wastes have been with us since the beginning of human civilization. It consists of a broad range of materials. It can be seen that in all over the world, at present, the majority of aggregate materials, for all construction applications, are gained from virgin resources such as crushed rock and sand and gravel. Material extraction causes many negative environmental impacts and consumes a finite natural resource. Industrial and domestic activities in the UK produce large quantities of waste and by-product materials that require management or disposal. To maintain levels of development and construction but reduce the application of natural aggregates, two main options are available: (i) optimise the use of natural materials or (ii) use of alternative materials. The CoURAgE (Construction with Unbound Road Aggregates in Europe) project is European-wide research collaboration, centred at Nottingham University that has evaluated possibilities for optimising the application of aggregates in the unbound layers of roads. (Hill *et al.* 2001)

In France, since the 1990s, increasing emphasis on sustainability in road construction has encouraged numerous studies into the technical and environmental feasibility of alternative materials. Therefore it is important to understand and forecast the in-service performance of alternative materials in highway pavements. To facilitate the understanding, there is need to develop experience through continuous monitoring of the on-site performance of roads built with alternative materials. Thus a national project called CAREX (2003–2005) was set up in order to help bridge the gap between current level of knowledge and the reality of using alternative materials in highways. Based on a stress-response approach applied to both the alternative material and the road structure and with the details of external factors, a standard framework for field data classification and analysis was implemented. To perform this analysis, a set of 17 documented field experiments was identified through a specific national survey and performed on 12 alternative materials. Seven of the materials were used in subbase while 5 were used in road base. The alternative materials in the CAREX project were different types of bottom ash (non-combustible residue) and blast furnace slag As case studies had different materials, it was not possible to run a general behavior for each of those materials. Structure monitoring was usually brief and mechanical loads too weak, which limits the importance of field testing. Therefore future testing methods should take into account the actual field conditions (François *et al.* 2009).

Edwards *et al.* (2006) attempted to increase the application of alternative materials through adopting performance related specifications, rather than the more traditional methods, which tend to favour materials from established aggregate sources.

The use of recycled and secondary materials in the construction industry is a key factor in attaining sustainable construction, so that recycling targets must become a most common part of the tendering process. The policy of driving organization such as Highways Agency is towards the use of performance related specifications. This policy and adoption of European wide aggregate standards on the one hand, and sustainable construction pressures on the other, all strongly emphasize on further need for more developments to specifications and performance assessment methodologies instead of creating barriers to the use of suitable materials. The use of performance specifications prepares the ground to improve resource efficiency and eliminate the potential barriers to the use of recycled and secondary materials in pavement construction. Laboratory equipment for investigating the performance of structural pavement layers is well established and performance related specifications were started since the 1990's. Performance related specifications for pavement foundations are being developed and are primarily based around *in-situ* control and compliance testing. Laboratory based tools for assessment of the performance of foundation materials and their durability under adverse conditions would be a key factor to the successful use of alternative materials (Edwards *et al.* 2006).

According to the Highways Agency (HA) guidance documents on recycling and secondary materials must undergo continuous development and make use of the latest research and knowledge (Sherwood, 1995). Edwards (2007) focused on determining which materials should have specific provision or a general provision within the MCHW. In an earlier publication, Edwards (2003) detailed the changes to material specification and the range of recycled and secondary aggregates which can be used within highway works. This study led to the development of the document HD35/03 from HD35/95. As a consequence of the work of Edwards (2007), a range of recycled and secondary materials was detailed according their appropriate areas of application in highway construction. This made it possible to publish a series of guidelines on the potential use of different materials. An extension and simplification of existing guidance was produced through this study.

A revision of HA guidance on recycling and secondary materials was undertaken on the basis of the work reported by Edwards (2003), which was issued as HD23. The aim of this revision was to remove the barriers for clients, materials suppliers and designers to considering recycling and use of secondary materials. The materials recommended in HD23 did not include those known to be technically unsuitable, especially on the basis of their durability properties. This approach to material assessment, for foundation materials had more emphasis on having suitable material characterisation techniques, including laboratory related performance testing. The HA guidance document on the utilisation of recycled and secondary aggregates (HD35, 2004) presented the permissible materials by application. HD35/04 and its background work clearly showed that some engineering properties of specific secondary and recycled materials are highly advantageous and relatively well understood (Edwards, 2003 and HD 35/04).

Paranavithana and Mohajerani (2006) investigated the effects of recycled concrete aggregates on properties of asphalt concrete. The following asphalt mixes were prepared in this study:

Mix I: Control mix containing fresh crushed basalt aggregates, 5.0%, 5.5% and 6.0% bitumen.

Mix II: (0–4.75 mm) Fresh aggregates and (4.75–20 mm) RCA, 5.1%, 5.5%, 6.0% and 6.5% bitumen.

The physical properties of this mixes are summarised in Table 2.7:

Table 2.7 Physical properties of mineral matter (From Paranavithana and Mohajerani, 2006)

Type of aggregate	Physical property	Mix I	Mix II
Coarse aggregates (1141.6.2-1996)	Particle density on a dry basis (t/m^3)	2.839	2.333
	Particle density on a saturated surface dry basis (t/m^3)	2.847	2.471
	Apparent particle density (t/m^3)	2.863	2.706
	Water absorption (%)	0.3	5.9
Fine aggregates (1141.5-1996)	Particle density on a dry basis (t/m^3)	2.701	2.701
	Particle density on a saturated surface dry basis (t/m^3)	2.735	2.735
	Apparent particle density (t/m^3)	2.798	2.798
	Water absorption (%)	1.3	1.3
Combined aggregates (1141.6.2-1996)	Particle density on a dry basis (t/m^3)	2.779	2.471
	Particle density on a saturated surface dry basis (t/m^3)	2.799	2.573
	Apparent particle density (t/m^3)	2.835	2.743
	Water absorption (%)	0.7	4.0

Cho and Yeo (2004) investigated the laboratory application of recycled waste aggregate in lean concrete subbases of rigid pavements in Korea. The research concentrated on basic tests for mechanical and toxicity properties of the materials. The research concluded that the use of waste aggregates significantly decreases the strength of concrete in comparison to conventional aggregates, the effect being more marked in the case of flexural strength. Typically the work showed that concrete made from waste aggregates had a flexural strength of only 60% of that of conventional aggregate concrete. These findings have obvious implications on quality control and design for concrete pavements, where strength is a major item of consideration. Despite the strength reduction effects of waste aggregates, they can be used in lean concrete bases to increase the economy of construction by up to 73% compared to natural aggregates.

2.3 Application of Recycled Aggregates in Unbound Mixtures

Most RCA and other recycled aggregates (RA) in general come from demolished buildings whereas RAP mainly originates from asphalt pavements undergoing repair. Because of the fundamental differences between general RA and RAP, it is reasonable to discuss literature on these materials within two distinct sections i.e. (i) RA/RCA and (ii) RAP.

2.3.1 Studies on application of RA and RCA and results

A study was recently conducted at the Hong Kong Polytechnic University to assess the possibility of applying RCA and crushed clay brick (CB) as aggregates in unbound subbase materials. The properties of natural aggregate (NA), RCA, and CB were investigated in this study as shown in Table 2.8. The results indicated that the use of 100% RCA increased the optimum moisture content and decreased the maximum dry density of the subbase materials compared to those of natural subbase materials. The following RCA/CB mixtures by mass were also investigated: (a) 75 % RCA + 25%CB, (b) 50 % RCA + 50%CB.

With higher percentages of CB than the above values, there was a further increase in optimum moisture content and decrease in maximum dry density. This behavior may be due to the lower particle density and higher water absorption of CB compared to RCA. Both the soaked and un-soaked CBR values for subbase materials having 100% RCA were lower than those for natural subbase materials. The CBR values decreased further as the replacement level of RCA by CB increased. Additionally, it was found that the soaked CBR values for all subbases, regardless of the fractions of RCA and CB constituents, were greater than 30%. This is the minimum strength required by Hong Kong's road design standards. It was also found that the swelling percentages for all subbases were less than 0.13%, which is acceptable by the Hong Kong's road design standards (Poon and Chan, 2006).

Table 2.8 Investigated properties for NA (From Poon and Chan, 2006)

Properties	Aggregate size				Test method
	40mm	20mm	10mm	<5mm	
Density-SSD (kg/m^3)	2622	2660	2577	2579	BS 812 Part 2
Density-oven-dry (kg/m^3)	2594	2644	2562	2492	
Water absorption (%)	1.06	0.57	0.59	3.51	
Ten percent fines – dry (kN)	-	190	-	-	BS 812 Part 111
Ten percent fines – soaked (kN)	-	190	-	-	
Water-soluble sulphate content (g/L)	-	-	-	0.025	BS 812 Part 111
Soundness %	-	97.5	-	-	BS 812 Part 121
Particle size distribution (mm)	Percent passing (%)				BS 812: 103.1
50.0	100	-	-	-	
37.5	96.9	100	-	-	
20.0	2.09	92.1	-	-	
14.0	0.1	36.0	100	-	
10.0	-	8.35	95.9	-	
5.0	-	0.41	13.5	97.3	
2.36	-	-	1.18	77.7	
1.18	-	-	-	58.0	
0.6	-	-	-	41.9	
0.3	-	-	-	19.2	

Table 2.8a Investigated properties for RCA (From Poon and Chan, 2006)

Properties	Aggregate size				Test method
	40mm	20mm	10mm	<5mm	
Density-SSD (kg/m^3)	2487	2546	2580	2310	BS 812 Part 2
Density-oven-dry (kg/m^3)	2411	2493	2523	2093	
Water absorption (%)	3.17	2.17	2.29	10.3	
Ten percent fines – dry (kN)	-	146	-	-	BS 812 Part 111
Ten percent fines – soaked (kN)	-	109	-	-	
Water-soluble sulphate content (g/L)	-	-	-	0.032	BS 812 Part 111
Soundness %	-	96.3	-	-	BS 812 Part 121
Particle size distribution (mm)	Percent passing (%)				BS 812: 103.1
50.0	100	-	-	-	
37.5	96.4	100	-	-	
20.0	3.98	98.4	-	-	
14.0	0.23	31.4	100	-	
10.0	-	4.73	93.8	-	
5.0	-	0.18	7.6	100	
2.36	-	-	1.6	73.6	
1.18	-	-	-	48.3	
0.6	-	-	-	31.1	
0.3	-	-	-	17.7	

Table 2.8b Investigated properties for CB (From Poon and Chan, 2006)

Properties	Aggregate size			Test method
	20mm	10mm	<5mm	
Density-SSD (kg/m^3)	1916	2147	2042	BS 812 Part 2
Density-oven-dry (kg/m^3)	1618	1797	1560	
Water absorption (%)	18.4	19.5	30.9	
Ten percent fines – dry (kN)	49	-	-	BS 812 Part 111
Ten percent fines – soaked (kN)	35	-	-	
Water-soluble sulphate content (g/L)	-	-	0.206	BS 812 Part 111
Particle size distribution (mm)	Percent passing (%)			BS 812: 103.1
37.5	100	-	-	
20.0	98.9	-	-	
14.0	54.2	100	-	
10.0	8.75	94.9	-	
5.0	1.37	7.6	100	
2.36		1.4	65.5	
1.18		-	43.5	
0.6		-	31.3	
0.3		-	23.1	

Poon *et al.* (2006) in another research attempted to examine the fundamental causes of the self-cementing properties of the fine (<5 mm) portion of RCA and its possible effects on the properties of the overall RCA subbase materials. A series of tests was initially carried out to assess the properties of different particle sizes of the fine recycled concrete aggregates (FRCA). From the results of X-ray diffraction, pH, compressive strength and permeability tests (on five fractions of FRCA such as: <0.15, 0.15-0.30, 0.30-0.60, 0.60-0.18 and <5mm), it was concluded that the most likely reason for the self-cementing behavior was the presence of particle sizes less than 0.15mm and between 0.3–0.6mm. Therefore these size ranges were referred to as active fractions. Furthermore, the results indicated that, if the active fractions of the FRCA were limited to a certain threshold (by weight of the total fine aggregate), the self-cementing properties of the FRCA had a negligible effect on the performance of the overall subbase materials having RCA. In two samples tested in this research the active size fractions of the FRCA reduced from 43.2% to 28.5%. According to the results of this study, the self-cementing properties were attributed to the intrinsic properties of the FRCA, which could be influenced by the age, strength and content of cementitious materials in the parent concrete from which the RCA was obtained. The age, grade and mix proportions of the original concrete would be the crucial factors affecting the residual un-hydrated cement and C_2S in the FRCA mortar. Other tests showed that the coefficient of permeability of RCA samples was higher than that of NA samples. This result can be explained by the porous structure of the RCA.

The OMCs were measured to be 8.6% and 11.8% for the NA and RCA subbase materials respectively. The corresponding MDDs were 2.15 Mg/m^3 and 2.02 Mg/m^3 for the NA and RCA subbase materials respectively. Due to the high water absorption and low particle density of RCA as compared to NA, the OMC of RCA was higher than that of NA. In contrast, the MDD of the RCA was lower than that of the NA. CBR tests were carried out for unsoaked samples and for samples soaked for 4 days. The results (Table 2.9) demonstrated that the bearing strength of RCA subbase materials was lower than that of NA but soaking had very little effect on strength, for both materials. Both RCA and NA materials showed negligible swelling (i.e., <0.07%) after a 4-day soaking period and so should pose no significant field problems in saturation conditions. The research results confirmed that the use of 100% RCA was adequate as granular subbase materials since according to Hong Kong Standards, materials with CBR>30% are considered suitable for use in subbases.

Table 2.9 Result of CBR test (From Poon *et al.* 2006)

Aggregate	**Unsoaked (%)**	**Soaked (%)**
NA	85	83
RCA	66	66

A project at the University of Central Florida's Circular Accelerated Test Track evaluated the performance of the pavement sections made with RCA under actual dual-wheel loading. RCA was used as a material for base layer, hot mix asphalt (HMA) surfacing and aggregate in Portland cement concrete pavements. (Chini *et al.* 2001)

The RCA samples for road base were tested for gradation, limerock bearing ratio (LBR), Los Angeles (LA) abrasion, sand equivalent, soundness loss, and compaction. The particle-size distribution was checked for conformance to the gradation specified by FDOT for graded aggregate bases. It was found that the tested RCA sample had a low content of material finer than 9.525 mm (3/8 in.). As shown in Table 2.10, the RCA met the requirements of LBR, LA abrasion, and sand equivalent for graded aggregate bases. However, the RCA did not meet the specified standards for soundness loss by sodium sulfate. A study by Mindy (1965) cited by Chini *et al.* (2001) indicated that the shape of the curves relating water absorption to the size of aggregate and of the percentage loss in the sulphate test to the size of aggregate is very similar. This shows that the result of the sulphate soundness test may be only a reflection of the water absorption of the stone. The high water absorption potential of the stone accounts for the high sodium sulphate loss in RCA. It was found that the cement mortar coating the RCA particles was reactive to sodium sulphate and contributed to increased loss of aggregate in the sodium sulphate test. Therefore, the sodium sulphate test is clearly not an appropriate soundness test for RCA and an alternative method, such as wetting and drying, should be adopted for investigation of durability of RCA (Chini *et al.* 2001).

Table 2.10 Lab test results for RCA base course (From Chini *et al.* 2001)

Laboratory Test	**Natural Aggregate Standards**	
	Pass	**Fail**
Sieve analysis (AASHTO T 27-93)		*
Limerock bearing ratio (FM 5-515)	238% > 100%	
LA abrasion (AASHTO T 96-94)	40% < 45%	
Soundness sodium sulfate (AASHTO T 104-94)		34% > 15%
Sand equivalent (AASHTO T 176-86)	75% > 28%	

Tam and Le (2007a) studied the relationships of six parameters describing the characteristics of recycled aggregate: (i) particle size distribution, (ii) particle density, (iii) porosity and absorption, (iv) particle shape, (v) strength and toughness, and (vi) chemical composition. In this work have taken 11 set of samples and compared their characteristics with normal aggregates. Their study was based on making the concrete from RA but interestingly it reveals that there is strong correlation among the parameters, and by measuring two of them: either "particle density" or "porosity and absorption" or "particle shape" or "strength and toughness", and "chemical content", it is sufficient to test recycled aggregate. They studied properties of aggregate using 23 standard tests which can be categorised into above six groups. These tests are listed in Table 2.11:

Table 2.11 Aggregate assessment (From Tam *et al.* 2007a)

Parameters	Tests
Particle size distribution	Test1: 10mm size aggregate of particle size distribution
	Test2: 20mm size aggregate of particle size distribution
Particle density	Test 3: 10mm size aggregate of particle density on an oven-dried basis (in Mg/m^3)
	Test 4: 20mm size aggregate of particle density on an oven-dried basis (in Mg/m^3)
	Test 5: 10mm size aggregate of particle density on a saturated and surface dried basis (in Mg/m^3)
	Test 6: 20mm size aggregate of particle density on a saturated and surface dried basis (in Mg/m^3)
	Test 7: 10mm size aggregate of apparent particle density (in Mg/m^3)
	Test 8: 20mm size aggregate of apparent particle density (in Mg/m^3)
Porosity and absorption	Test 9: 10mm size aggregate of water absorption (in % of dry mass)
	Test 10: 10mm size aggregate of saturated time for water absorption (in h)
	Test 11: 20mm size aggregate of water absorption (in % of dry mass)
	Test 12: 20mm size aggregate of saturated time for water absorption (in h)
	Test 13: 10mm size aggregate of moisture content (in % of dry mass)
	Test 14: 20mm size aggregate of moisture content (in % of dry mass)
Particle shape	Test 15: 10mm size aggregate of flakiness index (in %)
	Test 16: 20mm size aggregate of flakiness index (in %)
	Test 17: 10mm size aggregate of elongation index (in %)
	Test 18: 20mm size aggregate of elongation index (in %)
Strength and toughness	Test 19: ten percent fine value (in kN)
	Test 20: aggregate impact value (in %)
Chemical composition	Test 21: 10mm size aggregate of chloride content (in %)
	Test 22: 20mm size aggregate of chloride content (in %)
	Test 23: sulfate content (in %)

Tam *et al.* (2007b) pointed out that a possible reason for the rare application of RA for high-grade construction activities is its poor quality and variable nature caused by high porosity and water absorption rates. The main reason of this is the cement mortar remains on the surface of RA leading to a porous, highly absorptive and cracked layer during crushing of concrete waste. Tam *et al.* (200b) studied three pre-soaking treatment methods: $ReMortar_{HCL}$, $ReMortar_{H2SO4}$ and $ReMortar_{H3PO4}$ aiming at reducing the old cement mortar pasted onto the RA. Test results showed that the water absorption of the pre-treated RA reduced significantly and also the mechanical properties of the RA improved.

The compaction behavior of RA mixed with 5% to 25% sand, cement and clay brick was evaluated by Melbouci (2009) who found that 10% sand and 10% cement was the optimum combination to give the MDD and the minimum water content (Figures. 2.2 and 2.3). With concrete gravel (CG) added to 5% crushed brick particles of sizes less than 0.125mm, the increase in dry density was more marked as compared to the cases of mixtures containing sand. Amongst all the blended mixes studied, the one comprising CG + 10% cement had the highest dry density, as illustrated in Figure 2.4.

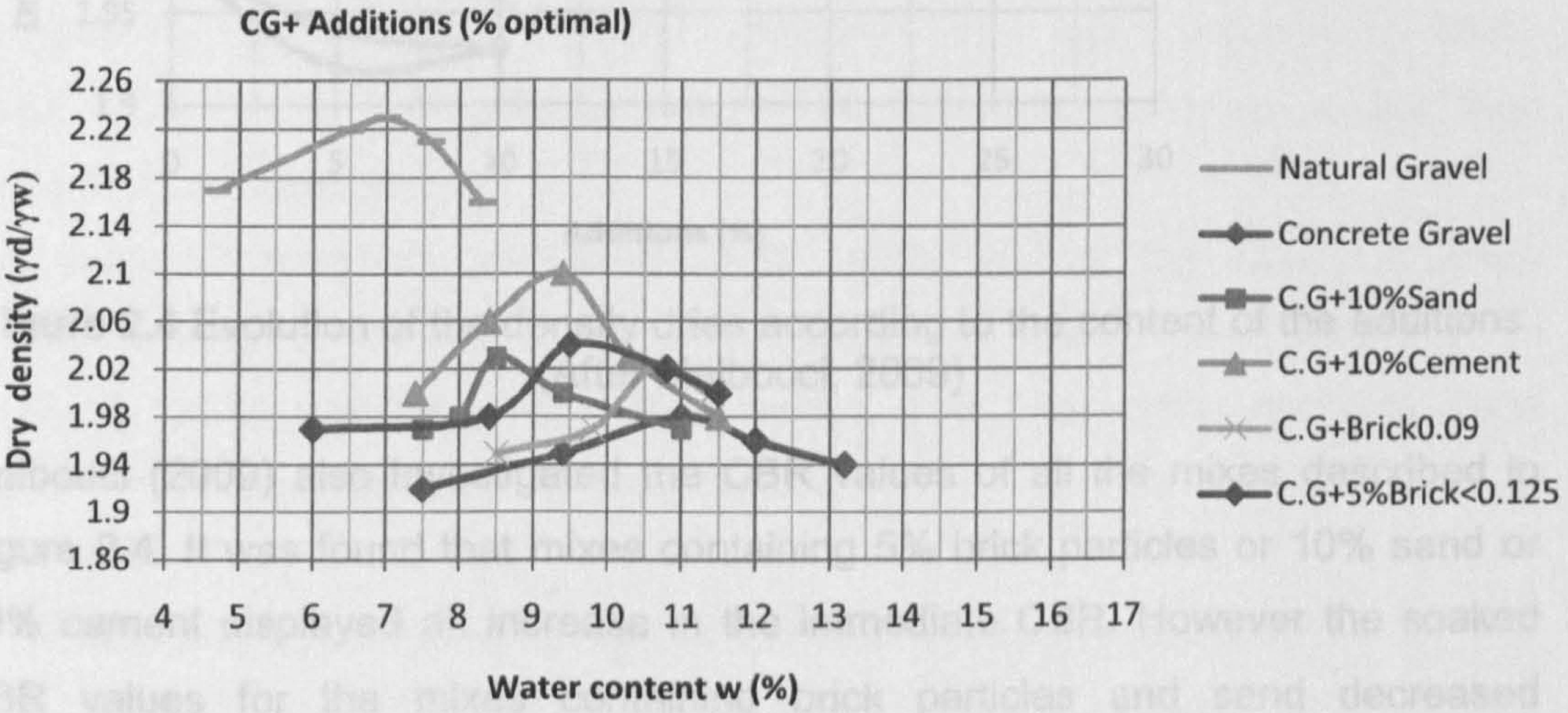


Figure 2.2 Comparison of the proctor curves of the natural aggregates and demolition with or without additions (After Melbouci, 2009)

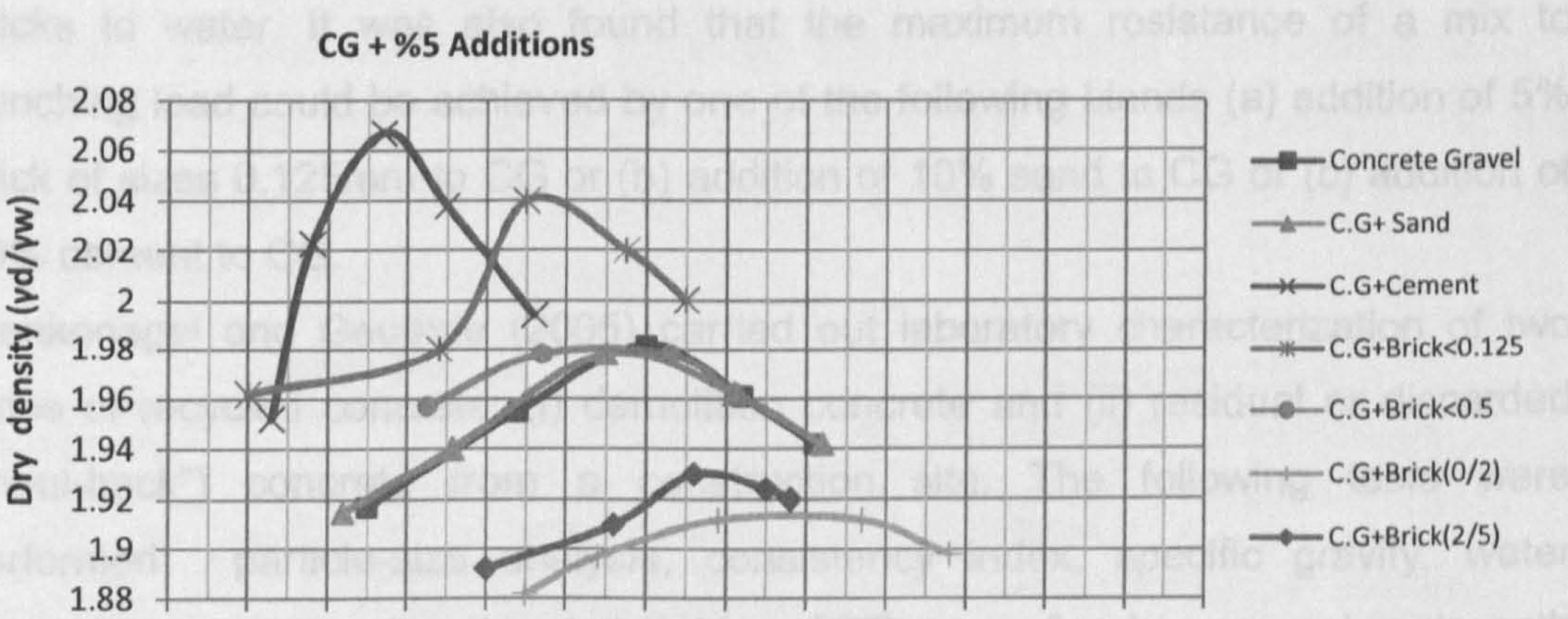


Figure 2.3 Proctor curves for the demolition aggregates with or without additions (After Melbouci, 2009)

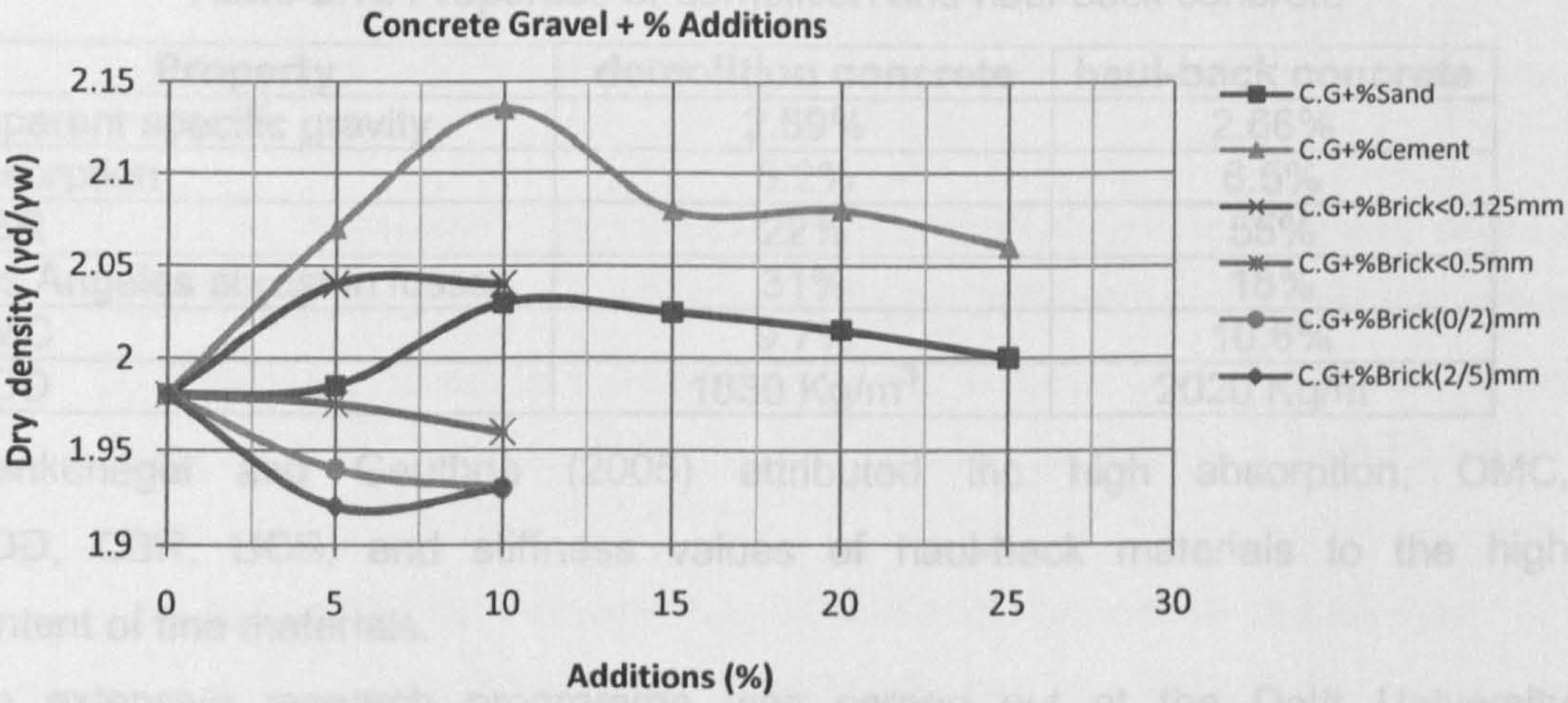


Figure 2.4 Evolution of the density dries according to the content of the additions (After Melbouci, 2009)

Melbouci (2009) also investigated the CBR values of all the mixes described in Figure 2.4. It was found that mixes containing 5% brick particles or 10% sand or 10% cement displayed an increase in the immediate CBR. However the soaked CBR values for the mixes containing brick particles and sand decreased comparing to those of CG. The soaked CBR values for the mixes having cement increased. The above observations can be explained by the sensitivity of the clay bricks to water. It was also found that the maximum resistance of a mix to punching load could be achieved by one of the following blends (a) addition of 5% brick of sizes 0.125mm to CG or (b) addition of 10% sand to CG or (c) addition of 10% cement to CG.

Blankenagel and Geuthrie (2005) carried out laboratory characterization of two types of recycled concrete: (i) demolition concrete and (ii) residual or discarded ("haul-back") concrete from a construction site. The following tests were performed: particle-size analysis, consistency index, specific gravity, water absorption, moisture-density relationships, CBR, unconfined compressive strength (UCS), stiffness, resistance to freeze-thaw cycles, and Los Angeles abrasion losses. It was found that both the demolition and haul-back concretes were non-plastic. The other properties are presented in Table 2.12.

Table 2.12 Properties of demolition and haul-back concrete

Property	demolition concrete	haul-back concrete
apparent specific gravity	2.59%	2.66%
absorption	5.2%	6.5%
CBR	22%	55%
Los Angeles abrasion losses	31%	18%
OMC	9.7%	10.6%
MDD	1830 Kg/m^3	2020 Kg/m^3

Blankenagel and Geuthrie (2005) attributed the high absorption, OMC, MDD, CBR, UCS, and stiffness values of haul-back materials to the high content of fine materials.

An extensive research programme was carried out at the Delft University of Technology (Molenaar and Van, 2002) to study the behavior of unbound base course materials made from recycled concrete and masonry rubble in relation to following factors:

- Gradation
- Composition (relative amount of concrete to masonry granulate)
- Particle shape (as possibly related to type of crusher used)
- Degree of compaction (DOC)
- Curing time

The mix of crushed concrete and crushed masonry materials was referred to as a mixed granulate (MG).The influence of gradation and composition on the mechanical behavior was investigated for the following 6 specifications of grading and 3 specifications of composition.

GRADATION

UL (upper limit)- the finest allowable gradation

CO (continuous)- a gradation with a high amount of fines (0-2mm) and of course material (8-40mm) and a bit of a gap in the fraction 2-8mm

UN (uniform)- the opposite of CO. a high percentage in the fraction 2-8mm

AL (average)- the average of the upper and the lower limits

LL(lower limit)- the coarsest allowable gradation

FL (Fuller)- the well-known theoretically ideal Fuller gradation. with an exponent n=0.45

COMPOSITION

A lower limit 50% RCA+50%RA

An average amount 65% RCA+35%RA

An upper limit 80% RCA+20%RA

Based on triaxial test results obtained in this study, the following conclusions were drawn:

- Cohesion increased with increasing fines content, curing time and degree of compaction (DOC). However it was the DOC that had the greatest effect on cohesion. As for the angle of internal friction, all the material mixes produced the results ranged from 38.7° to 40°as ind icated in Table 2.13.

Table 2.13 Change in cohesion and angle of internal friction with composition (After Molenaar and Van, 2002)

Material	Cohesion (KPa)	Angle of Internal Fraction (degree)
50% RCA+50%RA	91.4	40.4
65% RCA+35%RA	98.0	40.5
80% RCA+20%RA	125.5	38.7

- Resilient modulus values (M_r) were greatest in grading categories CO and AL, intermediate in UL and LL and lowest in grading FL and UN. It was also found that DOC had a greater effect on resilient modulus than gradation.
- DOC had the greatest effect on resistance to permanent deformation followed by composition and finally grading.

Generally, among the parameters analyzed, DOC had the highest influence on the mechanical properties of unbound base course materials. This was a significant result from a practical viewpoint since, for a given stockpile of material DOC can be controlled on site, albeit to a certain limit.

A research by Applied Research Associates (Saeed, 2006) used 8 case studies to investigate the potential application of RCA in base layers of airport and highway pavements. From the work, it was reported that RCA could be used successfully for airport and highway projects as base and compacted fill material. A survey carried out as part of the overall research revealed that:

(i) The biggest advantage of RCA over virgin aggregates is economy. By this it is meant that RCA enables savings to be made in transportation, landfill stockpiling fees and high cost of virgin aggregates.

(ii) Contractors experienced in the use of RCA generally prefer it to natural aggregates because of the following reasons: (a) easier compaction, (b) perceived better qualities than virgin aggregates (c) easy handling during construction (d) stable working platform allowing work to continue even when wet, (e) special construction equipment not required and different equipment may be used to spread RCA.

(iii) Despite the general view that RCA tends to degrade during transportation and placement, the survey found no evidence to support this.

(iv) RCA has a relatively high water demand and should therefore be compacted at a moisture level higher than the true optimum value.

A summary of laboratory investigation in this research is presented in Table 2.14 (Saeed, 2006).

Table 2.14 Summary of laboratory investigation (Saeed, 2006)

STANDARD	TEST	RESULT
ASTM C 136	Sieve Analysis of Fine and Coarse Aggregate	RCA could be produced and blended to provide the desired open and dense gradings.
ASTM D 1557	Modified Moisture/Density Test, Procedure C	Generally, RCA has a higher OMC and lower MDD than typical virgin aggregate.
ASTM D 4767	Static Triaxial Shear Test	The distressed RCA is comparable to a typical average virgin aggregate material.
NCHRP RPT 453	Repeated Load Triaxial Test	RCA permanent deformation characteristics are comparable to virgin aggregate material.
AASHTO T309	Resilient Modulus of Sub-grade Soil and Untreated Base/Sub-base Materials	RCA compared well with a typical virgin aggregate at a failure permanent deformation strain of 10%.

Maria Arm (2001) carried out experimental investigations into the stiffness of unbound layers made with crushed concrete from demolished structures. Batches of the material were wrapped in plastic foil and stored indoors for the following periods of time (in days) since the demolition: 1, 7, 15, 28, 60, 80, 365 and 730 days. Resilient moduli were measured through laboratory triaxial and field falling weight deflectometer (FWD) tests for the samples of different ages as stated above. Similar tests were also done on natural aggregates for comparison purposes. The results showed that the resilient modulus of the crushed concrete materials increased with increasing storage time. The rate of increase of resilient modulus was much higher in the first few months than in subsequent months. This growth was the benefit from self-cementing. This behavior was not observed in the case of natural aggregates.

Many secondary materials are taken into account for use as replacements for natural aggregates in highway works because of their suitable engineering and economic properties. The durability of aggregates and their resistance to the forces of weathering is one of the most important requirements in choosing aggregate for highway construction. The first concern is alternate freezing and thawing (F/T). During the design life of the pavement, recycled aggregates are exposed to F/T cycles and other aging processes like carbonation, coupled with intermittent infiltration and wetting by precipitation events. Current test methods for evaluating the soundness of aggregates due to F/T are based only on meeting physical test criteria and do not include the environmental impacts of the materials.

A research by Sanchez *et al.* (2009) presents results of the effect of F/T on the leaching behavior of major and minor constituents from a laboratory formulated granular cement-based material (CBM). This research has shown that F/T exposure can result in consolidation and self-cementing properties of granular CBM and can impact the long-term release of constituents from granular CBM, depending on the flow scenario. For applications where run-off is the primary route for leaching from granular CBM, F/T exposure can result in a significant reduction in the cumulative release of constituents as a function of time. A higher reduction was observed when the fine fraction of granular material was reduced.

A research (Cooley *et al.* 2007) by the United States Department of Transportation (DOT) into evaluation of recycled Portland cement concrete pavements for base course and gravel cushion material led to the following conclusions:

- Recycled Portland cement concrete pavements were a feasible option for utilisation in gravel cushion and aggregate base course layers.
- When gravel cushion and aggregate base course layers are relatively impermeable then leaching or precipitation effects caused by RCA should pose no problems. This finding was utilized by the South Dakota Department of Transportation (SDDOT) specification document for RCA in highways.
- In this research, alkali-silica reactivity was found to have no detrimental effect on the gravel cushion and aggregate base course layers.
- There was a concern about sulphate attack in thick layers of gravel cushion and aggregate base course layers however the critical thickness was not explored during this study.

- There was no discernible relationship between the results of Micro-Deval test and other durability tests. Thus the Micro-Deval test results did not truly represent the durability of the RCA.
- There was a strong correlation between the results from the following tests: (i) sodium sulphate soundness (ii) Los Angeles abrasion (iii) Impact test and (iv) combined New York F/T and Micro- Deval test. The results of the sodium sulphate test were considered to be the best indicator of durability.
- For some test cases, there was a possibility that the sulphate present in the sodium sulphate solution during soundness testing could attack the cement mortar existing in the RCA hence resulting in artificially high material loss values. In these cases, the New York F/T test could be used instead of the sodium sulfate soundness test (Figure 2.5).

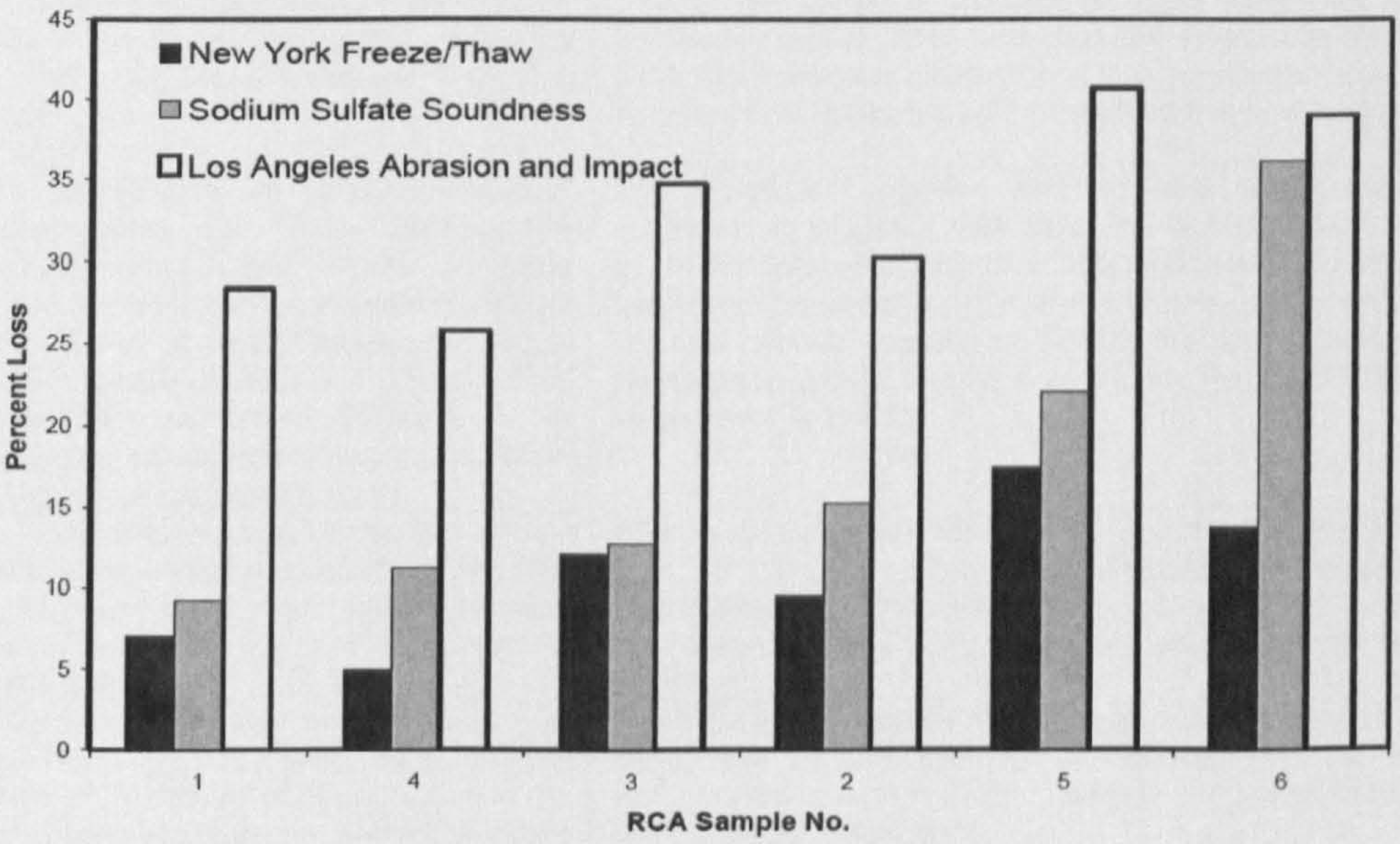


Figure 2.5 Comparisons of results from selected durability tests (From Cooley *et al.* 2007)

- To detect any fine materials in the RCA, which could affect durability, Atterberg limits tests were carried out and produced the results in Table 2.15.

Table 2.15 Durability related tests and specification limits (After Cooley *et al.* 2007)

Test	Property	Maximum Test Result
Atterberg Limits	Liquid Limit	25
	Plasticity Limit	6
Los Angeles Abrasion and Impact	% Loss	40
Sodium Sulfate Soundness	% Loss	15
Freeze/Thaw	% Loss	15

- Strong and stable gravel cushion and aggregate base course layers could be related to the gradation, angularity and cleanliness of the RCA materials. Current gradation requirements in SDDOT specifications for gravel cushion and aggregate base course were applicable.
- RCA pavements had a relatively high rate of water absorption and could therefore cause variability in compaction effectiveness of gravel cushion and aggregate base course layers.

This research recommended the points of Table 2.16.

Table 2.16 Recommendations (From Cooley *et al.* 2007)

NO	Recommendation	Reason/Benefit of Recommendation
1	Recycled Portland cement concrete pavements should be allowed within gravel cushion and aggregate base course layers.	Results from this study as well as the experience of other agencies suggest that recycled Portland cement concrete pavements are an acceptable alternative for granular pavement layers. The RCAs should meet the requirements of the Revised Sections 260 and 882 of the South Dakota Standard Specifications presented in Appendices C and D of this report.
2	Only recycled Portland cement concrete pavements owned by the Department should be allowed on new Department projects.	A number of references within the literature reported variability in the properties of RCAs when produced from construction/demolition debris. Research results derived from this study were based upon the testing of recycled rigid pavements and, therefore, are not applicable to construction/demolition debris.
3	RCAs crushed from Department owned pavements can be blended with conventional aggregates.	The literature states that RCAs can be blended with conventional aggregates; however, the RCAs should still meet all applicable requirements for gravel cushion or aggregate base course.
4	The cleanliness of RCAs should be specified. It is recommended that the Department maintain the requirements of a maximum liquid limit of 25 and maximum plasticity index of 6.	One of the six RCA samples included within this study contained natural, dirty and rounded fine materials. This RCA had low shear strength and was deemed undesirable. Results of the Atterberg limits identified this poor performer.
5	In order to minimize any potential effects of sulphate attack on RCA layers, the Department should test nearby subgrade soils and surface water for sulfates. ASTM C1580. Standard Test Method for Water Soluble Sulphate in Soil, and ASTM D516. Standard Test method for Sulphate Ion in Water should be used. Requirements within this research should be followed.	The literature suggested that sulphate attack may be applicable to relatively thick unbound layers of RCAs used as fill material. No definitive information was found that limited the thickness of RCA layers with respect to potential sulphate attack problems. Within the single reference identifying potential, sulphate problems, the fill thicknesses ranged from 2 to 5 ft.
6	The sodium sulphate soundness test should be used to evaluate the durability of potential RCAs for gravel cushion and aggregate base course. A maximum value of 15 percent is recommended.	Section 882 of the Departments Standard Specifications does not currently include a test to specify the durability of aggregates for granular bases. The sodium sulfate soundness test was identified as related to durability during this study.
7	The "Resistance of Coarse Aggregates to Degradation by F/T" test contained in Appendix B of the Final Report should be utilized if the results of the sodium sulphate soundness test are greater than 30 percent. A maximum value of 15 percent for this F/T test is recommended.	The literature suggests that some RCAs perform poorly during the sodium sulphate soundness test. This poor performance is related to the sulphates contained within the sodium sulphate solution.
8	The LAA test is recommended to evaluate the toughness and durability of recycled concrete aggregates. A maximum percent loss of 40 percent is recommended.	The LAA test showed a strong relationship with the sodium sulphate soundness and F/T tests. Based upon the results of this study and the literature, the LAA test is warranted within the Department's standard specifications.
9	No changes are recommended to the current gradation requirements for gravel cushion or aggregate base course.	The literature suggests that gradation requirements for RCAs should be similar to that of conventional aggregates. No data collected within this study suggests otherwise.
10	RCA stockpiles should be maintained at a moisture content representative of a saturated surface-dry condition.	The RCA samples included within this study were highly absorptive. Water absorption values for all six of the samples were above 5 percent. Problems with conducting Standard Proctor testing suggested that the RCAs need to be maintained at a moisture content near saturated surface-dry conditions or compaction of these materials may be highly variable.

Nataatmadja and Tan. (2000) investigated the performance of four Recycled Crushed Concrete (RCC) aggregates. The materials, gained by crushing concrete with compressive strength of 15 MPa (with a commercial name of AF RCC), 18.5, 49 and 75 MPa, were reconstituted to meet the grading requirements for a subbase material. Triaxial specimens were tested under repeated loading one day after compaction. It was observed that the original concrete comprehensive strength, the amount of softer material in the RCC aggregate and the flakiness index of the RCC aggregate could significantly influence the resilient modulus. To investigate the crushing and degradation of mixes under traffic loadings, 100 mm diameter specimens were confined in a rubber-lined mould and loaded under repeated stress of 550 kPa to determine the level of degradation. This test was carried out in place of the Repeated Load Triaxial Testing (RLTT) to ensure that specimen failure would not happen. Figure 2.6 shows the change in grading (from the original grading before compaction) after 50000 load repetitions. The graphs of Figure 2.6 represent the change in grading because of the combined effects of compaction and repeated loading. It was seen that in spite of the low compressive strength of AF RCC, it was relatively unaffected by the application of repeated loading. On the other hand, the 75 MPa RCC did not seem to perform that well because of its high flakiness index.

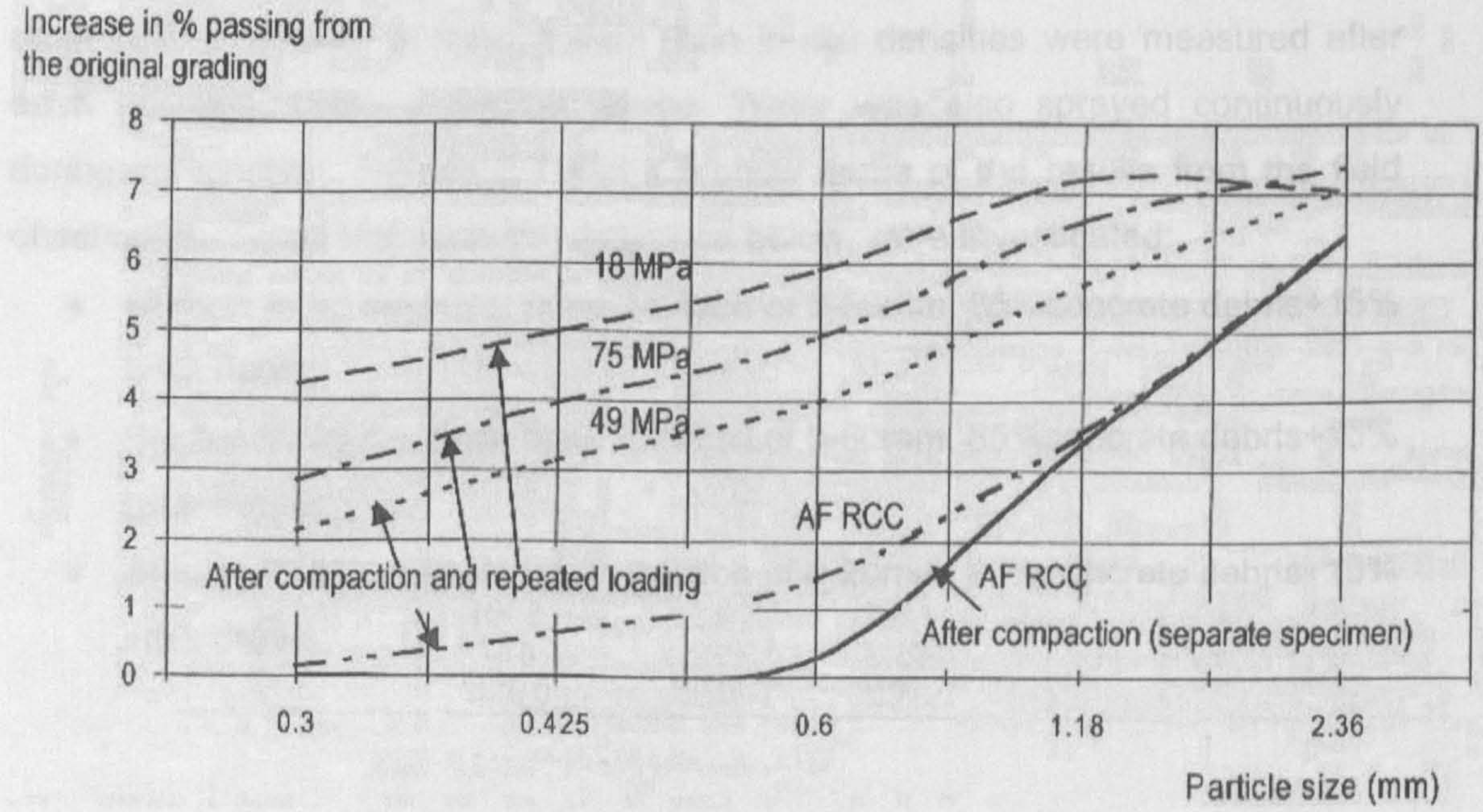


Figure 2.6 Particle crushing due to compaction and repeated loading (From Nataatmadja *et al.*, 2000)

Nataatmadja and Tan. (2000) established that, for well-graded and well-compacted RCC road aggregates, flakiness index was the most important factor affecting degradation characteristics, although the compressive strength of the concrete also had some effect. Based on this finding, it was recommended that flakiness index of RCC be limited to 10% (rather than 35% which is often specified for fresh virgin aggregates [confirmed by DMRB Vol.2 Sec.3 Part 9 BA92/07, 2007]) in order to prevent excessive permanent deformation while maximizing the resilient modulus. To investigate the effect of unhydrated cement in RCC, a number of RLTT specimens were kept for seven days after testing and then subjected to confined compressive strength tests. The results showed that the seven-day compressive strength of the specimens increased by between 1.2 and 2.3 MPa. These results suggested that the performance of RCC aggregates would be similar to that of 2% cement stabilized fresh aggregates.

A Norwegian company Franzefoss (Aurstad and Uthus, 2000) conducted a R&D project on recycling of heavy building and construction material (concrete, bricks and asphalt). The main object of this project was to evaluate the potential of these materials for use as unbound aggregates in base course layers. Laboratory investigations were carried out to characterize the materials, which were subsequently laid as lower bases in pavements and monitored for compaction. For field tests on unbound bases made of crushed concrete, two different steel rollers were applied. On each trial, one half was compacted with a small (3tons) and the other with a medium (6 tons) roller. Then in-situ densities were measured after each passage, using a nuclear gauge. Water was also sprayed continuously during compaction. Figures 2.7 and 2.8 show some of the results from the field observation. Three trial sections, described below, were investigated:

- Section A: 15 cm lower base, fraction of 0-60mm, 85%concrete debris+15% brick debris
- Section B: 25 cm lower base, fraction of 0-60mm, 85%concrete debris+15% brick debris
- Section C: 20 cm lower base, fraction of 0-20mm, 85%concrete debris+15% brick debris

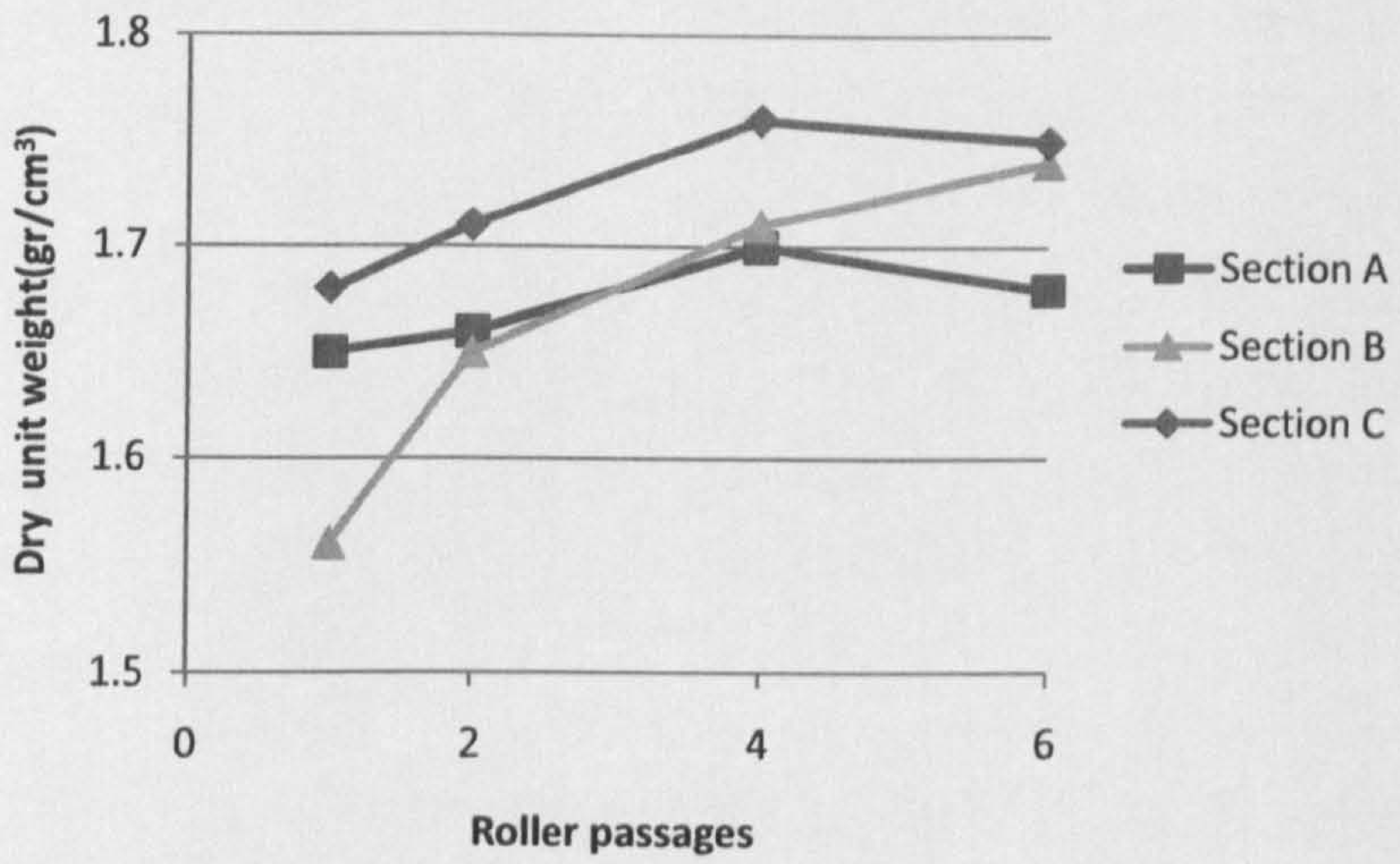


Figure 2.7 Compaction with light weight roller (3tons), lower base course of crushed concrete (After Aurstad and Uthus, 2000)

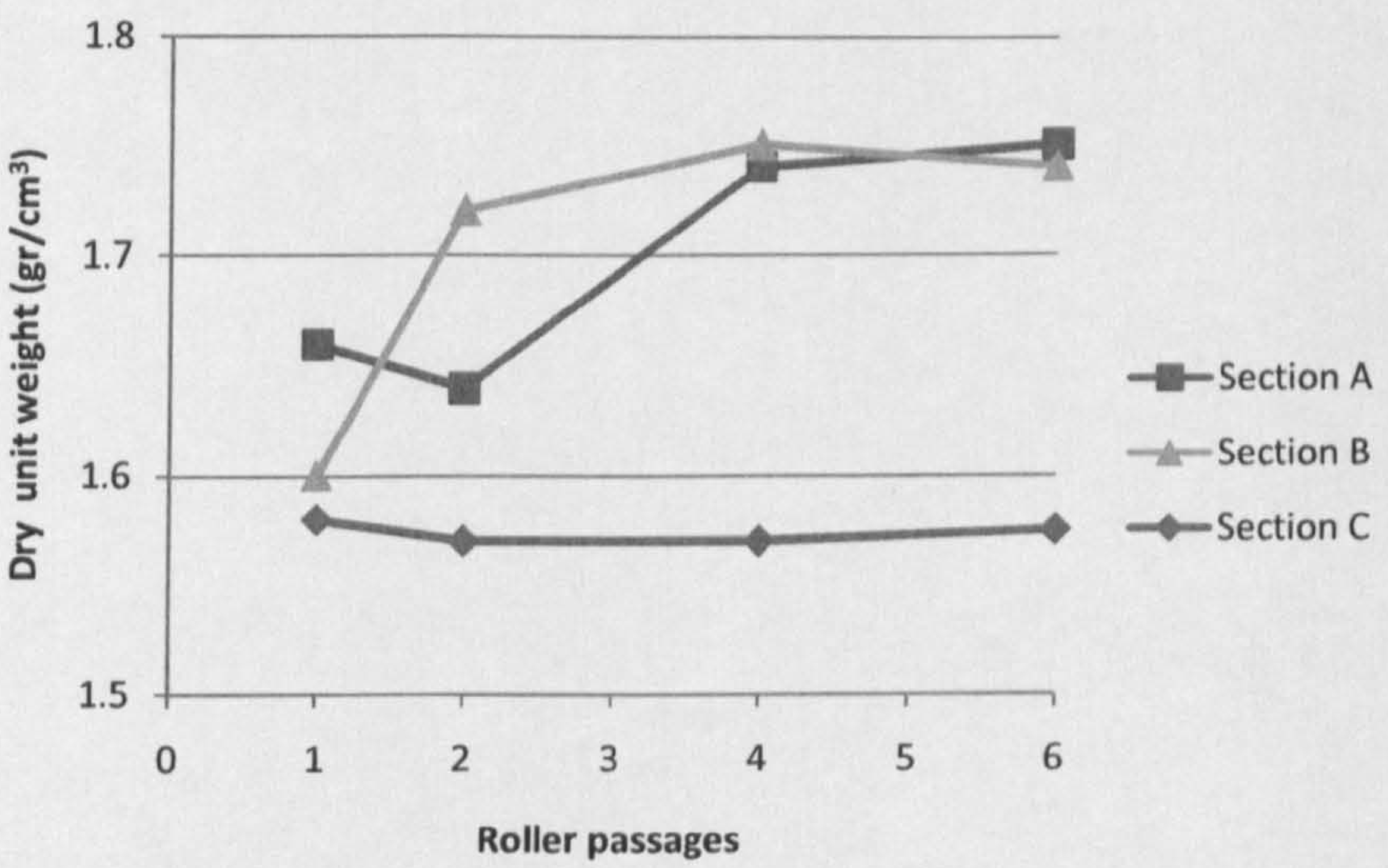


Figure 2.8 Compaction with medium weight roller (6tons), lower base course of crushed concrete (After Aurstad and Uthus, 2000)

It can be seen that section B had the highest rate of increase in dry unit weight with increasing number of roller passes up to maximum compaction point. In section C, under the 6 ton roller, the dry unit weight was unresponsive to number of passes possibly due to the high fines content (0-20 mm) in the material. In section A, under the 6 ton roller, more than 2 passes were required to cause a measurable increase in dry unit weight. On the sixth roller pass, the density of the material in section A decreased slightly and the reason for this was confirmed to be crushing of particles and losses of the material near the surface. The behavior under roller passage directs this PhD research to measure the degradation of purposed materials.

At the University of Central Florida (2002) investigations were carried out for two sections of a highway, one of which had a RCA base while the other had a limerock base. The investigations involved laboratory tests on physical and mechanical properties of RCA including: (a) gradation, (b) LBR, (c) LA abrasion loss, (d) surface soundness, (e) sand equivalent, (f) heavy metal, (g) OMC and MDD, (h) hydraulic conductivity, and (j) impurities. The test results were used to develop a set of guidelines and specifications for the Florida DOT as shown in Table 2.17 (Kuo et al., 2002).

Table 2.17 Proposed RCA specifications by FDOT (From Kuo et al., 2002)

Type of Test	Average Test Results	Proposed Specifications	FDOT Specifications
Gradation Test (FM 1-T027)	Average Gradation	Gradation Limits (90% Confidence Interval)	Section 204
Sieve No.		---	---
50mm	100.0	Min.100-Max.100	Min.100-Max.100
37.5mm	99.5	Min.98-Max.100	Min.95-Max.100
19mm	83.2	Min.65-Max.100	Min.65-Max.90
9.5mm	61.2	Min.40-Max.83	Min.45-Max.75
#4	44.8	Min.27-Max.63	Min.35-Max.65
#10	34.4	Min.20-Max.49	Min.25-Max.45
#50	15.7	Min.8-Max.24	Min.5-Max.25
#200	3.8	Min.2-Max.6	Min.0-Max.10
LBR Test (FM-515)	181.71	Min.120	100
LA Abrasion Loss (FM 1-T096)	44.02%	<48%	<45
Sodium Sulfate Test (AASHTO T-104)	52%	<50%	15%
Sand Equivalent (AASHTO T-176)	70.5%	>70%	≥28%
Heavy Metals (EPA-96)	0-12ppm	5ppm	5ppm
Asbestos (EPA-89)	Free of Asbestos	Free of Asbestos	section 112 EPA
Optimum Moisture Content (FM5-521)	11.2%-12.1%	10%-12%	Not Specified
Maximum Dry Unit Weight (FM5-521)	113.8 lb/ft^3-114.8 lb/ft^3	108 lb/ft^3-120 lb/ft^3	98% of Max. Dry Density
Permeability (FM5-513)	0.72 (ft/day)	0.10 to 1.40 (ft/day)	Not Specified
Impurities (FM 1 T-194)	1.99% by weight	<2.0% by weight	Substantially Free of Impurities
Structural Coefficient	0.34	0.30	0.15
Thickness Requirement	4 in. (10.2 cm)	Min.8.0 in. (20.4cm)	Min.8.0 in. (20.4cm)
Thickness Equivalency	0.34 (RCA)/0.213 (LR)	1.0 in. (2.54cm)	1.6 in. (4.0cm)

FM indicates Florida test method number.

Thurber Engineering Ltd. (Thurber Engineering Ltd. and Crawford, H.S., 2001) was asked by Alberta Environment (2001) to provide technical input to their Construction, Renovation and Demolition (CRD) Waste Reduction Advisory Committee (WRAC) with regard to the application of recycled aggregates within the province of Alberta. According their report the range of tests used to investigate Granular Base Course (G.B.C), asphalt concrete embankment fill and drainage material aggregates was summarised in Table 2.18.

Table 2.18 Applicable tests to assess quality of aggregate (From Thurber Engineering Ltd. and Crawford, H.S., 2001)

	Asphalt (RAP)	Portland Cement Concrete	Granular Base	Embankment Fill	Drainage Material
Moisture Content	✓	✓	✓	✓	✓
Gradation	✓	✓	✓	✓	✓
Petrographic Examination	✓	✓			
Organic Impurities	✓	✓			
Soundness (Chemical)	✓	✓	✓	✓	
Freeze-Thaw Soundness	✓	✓			
Particle Shape/Texture	✓	✓	✓		✓
Los Angeles Abrasion	✓	✓	✓		
Specific Gravity and Absorption	✓	✓			
Aggregate Expansion (Hydration)		✓	✓	✓	
Atterburg Limits (Plasticity)	✓	✓	✓	✓	
Potential Alkali-Silica Reactivity		✓			
Corrosion Testing (pH)		✓			
Mix Design	✓	✓			
Air Content/Slump		✓			
Marshal Analysis	✓				
Compressive Strength		✓			
Standard Proctor			✓	✓	✓
Permeability			✓	✓	✓
California Bearing Ratio			✓	✓	
Field Density Testing	✓		✓	✓	

At Brigham Young University (Blankenagel, 2005) a research project was carried out to investigate two recycled concrete materials (RCM) classified as poorly graded gravel from demolition waste and poorly graded sand from haul-back sources. Both aggregates were classified as A-1-a in the AASHTO soil classification system. The following parameter values were also determined for the two materials:

	Demolition gravel	Haul-back sand
7-day soaked CBR	22%	55%
7 day soaked UCS	1260 kPa	1820 kPa
7 day soaked modulus of elasticity	110 MPa	150 MPa

Significant increases in strength and stiffness were seen for both aggregates during the first 2 to 3 days after compaction. These marked increases were attributed to the reaction of previously unhydrated cement with water to form new cementitious products. Both aggregates experienced strength and stiffness losses under freeze-thaw cycling and the haul-back material was slower to reach a residual stiffness than the demolition material. Unified Compressive Strength (UCS) decrease for haul-back material after completion of the F/T cycle was considerably less than that exhibited by the demolition material. However, the haul-back material had a moisture susceptibility rating of marginal in the Tube Suction Test (TST), while the demolition aggregates were rated as good.

Field monitoring was also carried out for highway pavement constructed from the two materials described earlier. It was found that the stiffness of the road base made from the materials was quite sensitive to changes in moisture. The CBR related stiffness decreased by around 60% between the summer and spring season. This behavior was corroborated by results from Soil Stiffness Gauge (SSG) tests. From this research it was concluded that the engineering performance of the RCM compared favorably with conventional highway aggregates. With regard to the laboratory and field data developed in this research, engineers should be able to estimate the strength and durability parameters of RCM required for pavement design. Only two distinct sources of RCM were represented in this research, the properties of any RCM will depend on its source and will probably differ to some degree from the values reported in this report. Blankenagel (2005) recommended that if the material from other sources is to be used on a heavily trafficked pavement then more detailed laboratory testing should be performed to ascertain the full range of characteristics of the proposed recycled material and its source. If the material showed inadequate strength or resistance to moisture and frost damage, stabilization techniques were recommended to improve the properties of the RCM.

Materials for recycling based on final technical report from CEN/TC 154 a sub-committee of RILEM Ad hoc (Hendriks and Pietersen, 2000) for recycled aggregates and application in unbound layers have been categorized in accordance with Table 2.19.

Table 2.19 Materials for recycling and application in unbound uses (From Hendriks and Pietersen, 2000)

TC154 sub-committee	Concrete >2000kg/m^3	Masonry >1600 kg/m^3	Mixed Concrete /Masonry	Hydraulic Bound	Asphalt	Mixed Asphalt Concrete
Surface Layer	-	-	-	-	Temporary or site roads only	-
Base Layer	✓	✓	✓	✓	✓	Further information required
Sub-base Layer	✓	✓	✓	✓	✓	✓
Capping Layer	✓	✓	✓	✓	✓	✓

This sub-committee summarised the requirement for test methods in the application of recycled aggregate in highways with unbound layers according to Table 2.20.

Table 2.20 Properties required for unbound uses (From Hendriks and Pietersen, 2000)

PROPERTIES	REQUIRMENT
General description	Composition
Density	Dry particle density
Density grading	Density grading related to composition
Lump impurities / Coarse organic matter	Lump impurity/Coarse organic matter
Size distribution / grading	Grading
Shape	Particle shape
Fines content	Fines content
Strength	Resistance to fragmentation
Weathering resistance	Freeze-Thaw resistance
Abrasion(Wear) resistance	Resistance to wear
Leachate	Leaching

In Germany, the composition and physical properties of recycled aggregates, made from masonry rubble, was investigated over a ten-year period. This work, which utilized a large number of test samples, was reported by Müller (2005). It was surprisingly revealed that, when compared to recycled concrete, the particular clay brick tested had good frost resistance. It was also found that weathering of the clay bricks resulted in a mass loss of 2.6%, while the corresponding value for the recycled concrete was 7.7%. Furthermore, when a series of F/T cycles was imposed on a mix of clay brick and recycled concrete materials, percentage mass loss due to splintering was about 7.4%. To demonstrate that the recycled concrete material element had more influence on splintering than the clay brick, it was found that the brick fragments separated out from the mixture showed splintering of 1.2% by mass compared to 11% for the mortar fragments. It is worth noting that the maximum permitted F/T splintering for road construction material is 3% according to the German Standard.

In a research by Nataatmadja and Tan (2001) on four RCA materials made from concrete of different strength classes (based on Australian Standards) indicated the basic properties in Table 2.21. The results shows harder concretes will be less degraded under traffic loading.

Table 2.21 Basic properties of RCA aggregates (From Nataatmadja and Tan, 2001)

Properties	15 MPa	18.5 MPa	49 MPa	75 MPa
Los Angeles Abrasion(LAA)(B grading), %	30	22	25	21
Los Angeles Abrasion(LAA)(K grading), %	27	24	21	24
Aggregate Crushing Value (ACV), %	24	23	22	22
10% fines, kN	149	158	166	187
Flakiness index	6	12	9	144

Mills-Beale and You (2010) in a study on capability of using RCA in hot mix asphalt (HMA) for low trafficked highways indicated that virgin aggregates (VA) being replaced by the RCA can save energy of compaction effort. The construction Energy Index (CEI) of the VA-RCA mixes with 75%, 50%, 35% and 25% of RCA replaced for the VA is shown in Figure 2.9. The result of rutting failure potential indicated that use of up to 75% RCA can have satisfactory performance in permanent deformation.

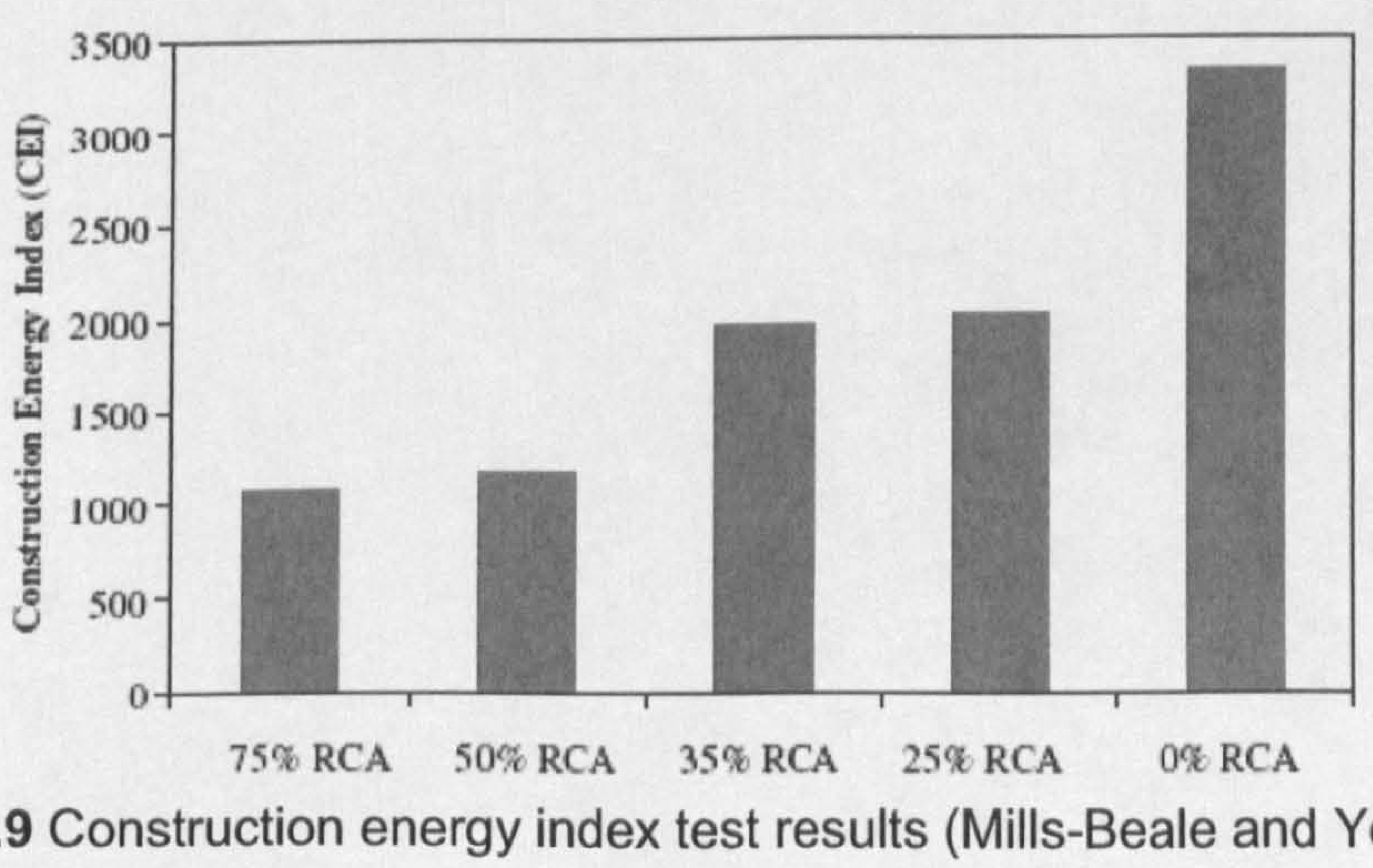


Figure 2.9 Construction energy index test results (Mills-Beale and You, 2010)

Sakanakura et al. (2009) carried out investigations into weathering factors (humidity, gas composition and temperature) considered to influence the leaching behavior of inorganic contaminants from recycled road aggregates. The following three accelerating exposure tests were carried out: (i) freezing–melting cycle test (ii) carbonation test (iii) dry–humid cycle test. The research findings demonstrated the importance of considering the freezing–melting conditions when applying recycled materials in highway pavements. It was argued that the freezing condition in a freezing–melting exposure test influences the physical properties of the material by creating enlarged micropores through freezing of pore water thereby encouraging the mobilization of the solution. This mechanism should in theory result in increased fragmentation of the samples however in this research there was not sufficient data available to support the expected behavior.

2.3.2 Studies on application of RAP and results

From a laboratory study (Huang et al., 2005) of concrete made from Portland cement and RAP, it was revealed that the asphalt present in RAP forms a thin film at the interface of the cement mortar and the aggregate. The film suppresses crack propagation through a typical aggregate particle (see Figure 2.10). Thus cracks mainly develop around rather than go through an aggregate particle, during which more energy can be dissipated. This is likely to be the one of the fundamental reasons for the improvement of strength of concrete made with aggregates coated with emulsified asphalt. This observed behavior, among other mechanisms, will form the basis of discussions of the findings of the current research in chapter 5.

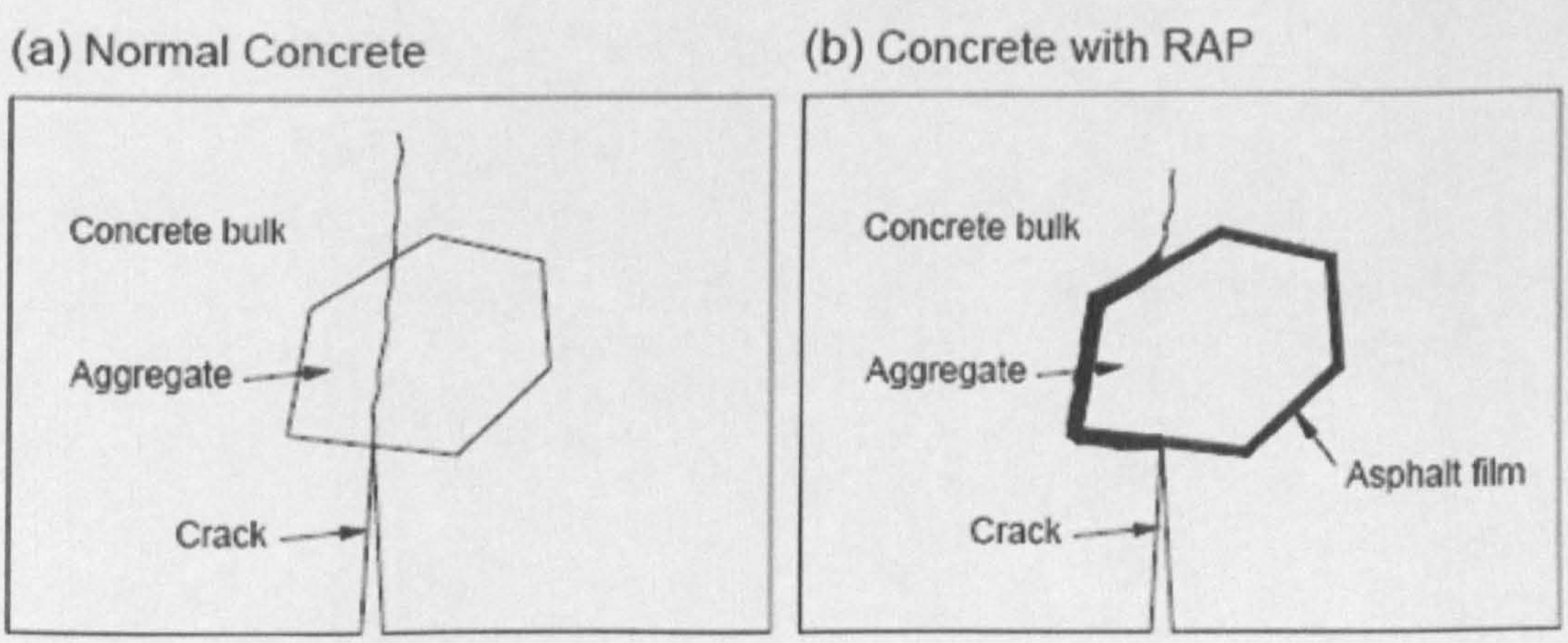


Figure 2.10 Crack propagation in concrete and concrete with RAP (From Huang et al., 2005)

Research by Taha *et al.* (1999) in the Sultanate of Oman focused on evaluation of RAP and virgin aggregate (VA) blends as base and subbase materials for highways. Laboratory tests including physical, compaction and CBR were carried out on the following RAP/VA mixes: 100/0, 80/20, 60/40, 40/60, 20/80 and 0/100 percent. Overall the most favorable results were obtained from the 60/40, 40/60, 20/80 and 0/100 percent RAP/VA mixes. It was found that an increase in the percentage of VA resulted in an increase in dry density and CBR. Taha *et al.* (2002) conducted laboratory evaluation of cement stabilized RAP and RAP/VA mixes with the following proportion: 100/0, 90/10, 80/20, 70/30, and 0/100%. For cement contents of 0%, 3%, 5%, and 7% the optimum moisture content (OMC) and maximum dry density (δ_d) values shown in Table 2.22 were found by

Table 2.22 Optimum moisture content and maximum dry density for all blends (After Taha *et al.*, 2002)

Percent cement	100% RAP		90% RAP		80% RAP		70% RAP		100% virgin aggregate	
	OMC (%)	Max δ_d (g/cm^3)	OMC (%)	Max δ_d (g/cm^3)	OMC (%)	Max δ_d (g/cm^3)	OMC (%)	Max δ_d (g/cm^3)	OMC (%)	Max δ_d (g/cm^3)
0	7.0	1.885	7.2	1.937	8.0	1.952	8.2	2.174	5.7	2.250
3	7.8	1.921	8.0	1.988	8.2	2.060	8.6	2.187	6.0	2.313
5	8.2	1.993	8.8	2.056	8.4	2.104	8.8	2.238	6.4	2.381
7	8.5	2.014	9.0	2.096	9.0	2.116	9.1	2.246	7.4	2.387

A project was carried out by the Norwegian company Franzefoss Ltd (Aurstad and Uthus, 2000) on crushed recycled asphalt intended for use in unbound base course. In this project, CBR values of gyrator compacted specimens and Modified Proctor compacted specimens were compared. For the gyrator compacted samples, CBR and dry unit weights were measured for samples subjected to different cycles as follows: 10, 40, 100 and 400. For the Modified Proctor test specimens, CBR and dry unit weights were measured for samples compacted with 10, 25, and 60 blows per layer. All the specimens were compacted with an optimum water content of 3.5%. Figure 2.11 revealed that gyratory compacted specimens produced much higher CBR values than did modified Proctor compacted specimens.

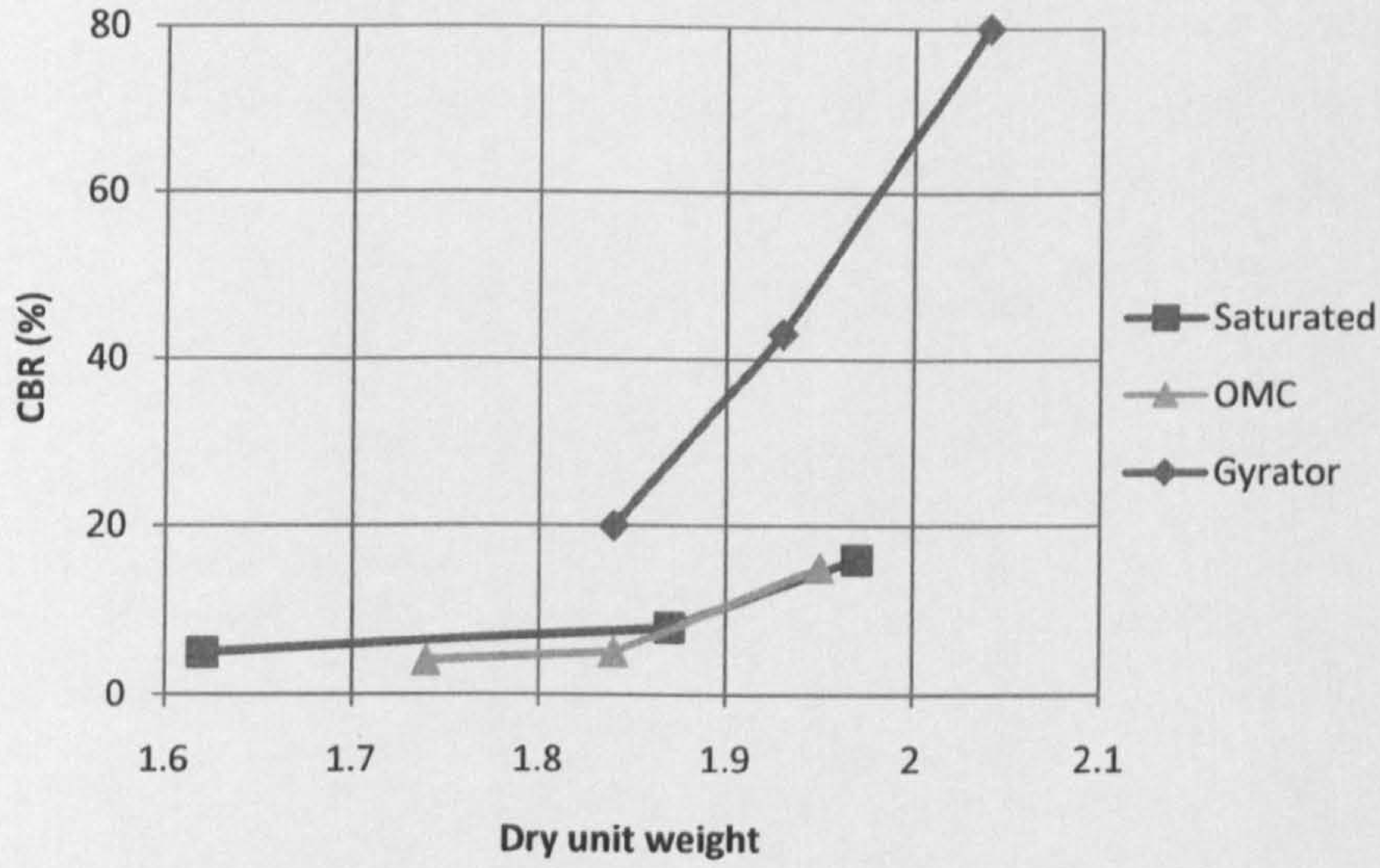


Figure 2.11 CBR curves, varying compaction effort (crushed asphalt 0-19mm) (After Aurstad and Uthus, 2000)

Through field monitoring immediately after compaction of asphalt crushed by static and heavy weight steel roller, indirect tensile testing were performed on material samples taken from the road. Material was laid in two layers, each 50 mm thick. The results of the field observation are shown in Table 2.23.

Table 2.23 Indirect tensile testing on gyratory compacted material taken from the test section of a dual carriageway (After Aurstad and Uthus, 2000)

	Density measured on road (g/cm^3)	**Gyrator density(g/cm^3)**	**Max indirect tensile stress (25 °C)**
Right carriageway	2.039	2.115	300 kPa
Left carriageway	2.053	2.126	352 kPa

The results of field observations confirmed the values found in the lab investigations.

2.3.3 Performance-related investigations and results

Research carried out at Nottingham University (2001), using performance-based specifications concluded that many alternative materials performed just as well as and in some cases even better than conventional materials. The research showed that the stiffness and strength properties of bound materials were dependent on the coarsest fraction of the material and the amount of binder. Coarse fraction of materials had a higher stiffness than fine materials and also needed less binder to gain a particular strength. Repeated load triaxial tests and indirect tensile tests were carried out on aggregates with and without stabilizers. Figure 2.12 illustrates the arrangement of the repeated load triaxial test while Figure 2.13 is a schematic representation of the indirect tensile test. It is imperative to assess the in-situ performance of a pavement material by testing representative samples at moisture contents that reflect the in-situ conditions. If laboratory tests demonstrate high strength and stiffness of the treated alternative aggregates then the design pavement thicknesses can be reduced and hence increase economy without compromising on the technical performance of the pavement (Hill *et al.* 2001).

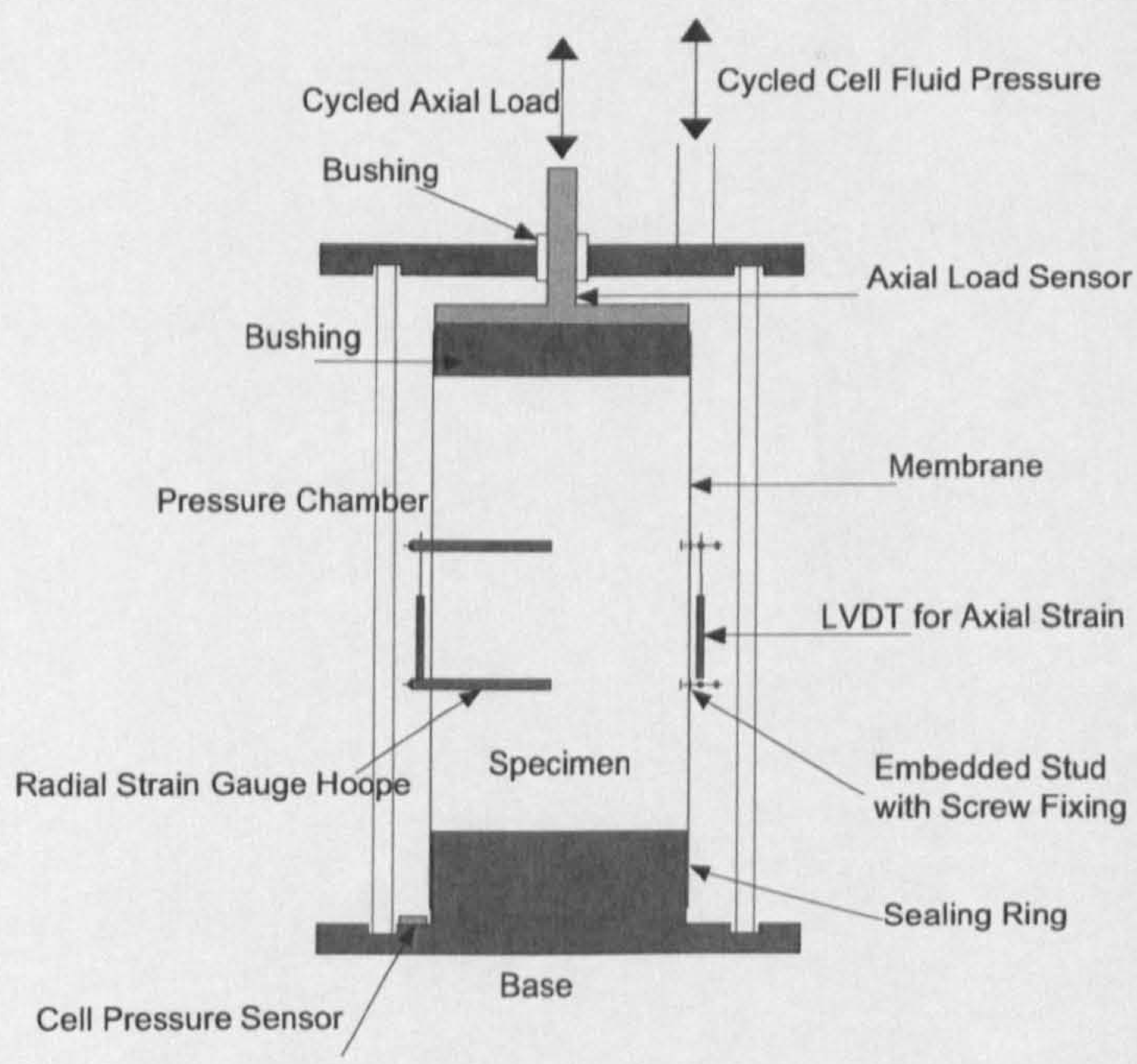


Figure 2.12 Schematic diagram of the repeated load triaxial test apparatus (RLT) (From Hill *et al.* 2001)

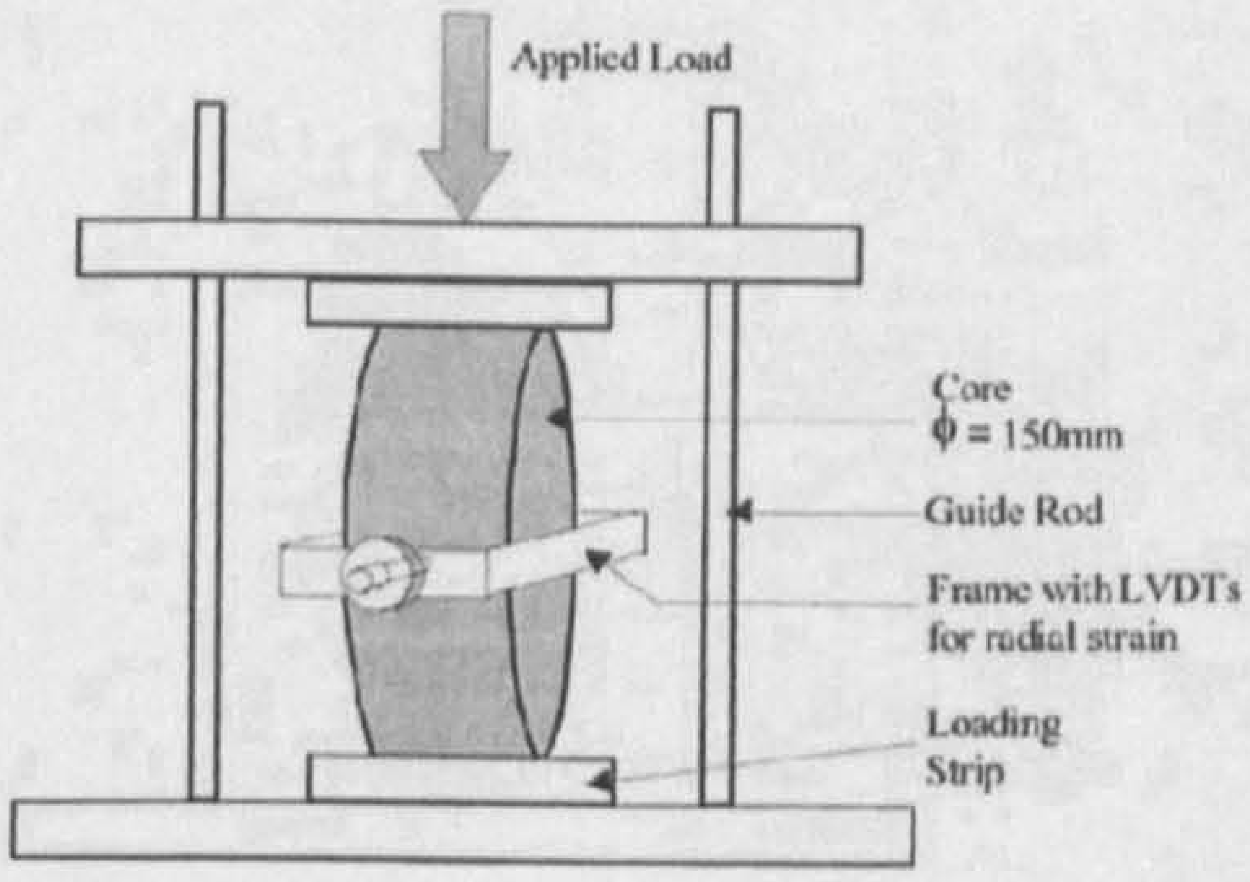


Figure 2.13 Schematic diagrams of the indirect tensile test apparatus (From Hill *et al.* 2001)

The NCHRP Report 598 (2008) sets out the procedures for performance-based testing to aid selection of recycled hot-mix asphalt (HMA) and recycled Portland cement concrete (PCC) materials as single or compound aggregates in unbound pavements. The report evaluates existing test methods for predicting pavement performance in terms of the potential application to reclaimed concrete pavement (RCP) and also describes the development of new and improved tests. Among the important conclusions made in the NCHRP Report 598 (Saeed, 2008):

1. Fatigue cracking, rutting/corrugations, depressions and frost heave in flexible pavements and cracking, pumping/ faulting, frost heave, and erosion in rigid pavements and damages are associated with poor performance of the recycled aggregates used in the unbound layers of these pavements.
2. The properties of recycled aggregates that have an influence on pavement performance include: shear strength, stiffness, toughness, durability, frost susceptibility, and permeability. Shear strength and stiffness (resilient modulus) have a much greater effect on the performance of an unbound aggregate layer than the other properties.
3. For unbound pavement layers, the following tests relating to the performance of recycled aggregates are the most important (Table 2.24):
    - Screening tests for sieve analysis and the moisture-density relationship,
    - The Micro-Deval test for toughness,
    - Resilient modulus for stiffness,
    - Static triaxial and repeated load at OMC and saturated for shear strength, and
    - The tube suction test for frost susceptibility.

Table 2.24 Tests selected for the laboratory test program (Saeed, 2008)

Aggregate property	Test method	Test reference	Test parameter
Screening Tests	Sieve Analysis	AASHTO T 27 and T 11	Particle size distribution
	Moisture-Density Relationship	AASHTO T 180	Maximum dry density and optimum moisture content
Shear Strength	Static Triaxial Shear	AASHTO T 296	c, F, shear strength
	Repeated Load Triaxial	*NCHRP Report 453 (1)*	
Permeability	Saturated Repeated Load Triaxial	*NCHRP Report 453 (1)*	
Stiffness	Resilient Modulus	*NCHRP Report 453 (1)*	
Frost Susceptibility	Tube Suction Test	*NCHRP Report 453 (1)*	
	Index Method	U.S. Army Corps of Engineers, F categories	
Toughness Tests	Micro-Deval	AASHTO TP 58	
Durability	Canadian Freeze-Thaw	MTO LS-614	

In a research carried out in continuation of laboratory tests performed by the University of Central Florida (2002), one trial pavement section made of RCA and another made of lime rock were constructed at UCF-CATT. The two trial sections were used for accelerated performance tests for the base courses layers. A total of 362,198 load repetitions were applied to the two trial sections, both of which were designed to have a minimum design life of 36 years based on identical traffic conditions. Along both trial sections, pavement damages due to deep rutting, cracking, and surface rutting were measured. No cracks were observed in the RCA trial section, but many transverse cracks and one longitudinal crack occurred on the lime-rock section. Prior to the accelerated performance tests, a falling weight deflectometer (FWD) test was performed on the test sections. The in in-situ moduli of the RCA, lime-rock and subgrade were back-analyzed based on the data from FWD test and modeling using KENLAYER^{TM} (Huang, Yang H.,2004) computer software. Subsequently, the tensile strain at the bottom of the surface course layer and the compressive strain at the top of the subgrade were calculated, and the allowable load repetitions to the fatigue and rutting failures of the pavement system were determined. The allowable load repetitions for RCA test sections were observed to be much higher than those of lime-rock test sections. None of the trial sections showed fatigue or rutting related failure. The results of this particular study showed that pavement sections built with RCA base demonstrated a better performance than the pavement section built with the limerock base (Kuo et al., 2002).

CHAPTER 3 METHODOLOGY

3.0 Preamble

This chapter presents a brief overview of the research methodologies and goals to identify the most suitable methods to adopt in the project. The proposed methods are discussed including the research issues pertinent to this project. The application of each method is justified and the research philosophy is described.

3.1 Review of Assessment Methodology

A study by Potter (1996) evaluated the application of recycled materials and secondary aggregates and developed design guides which were published in 1996. The work led to the development of the guidelines summarized by Biczysko (1999). This guide included the flow chart shown in Figure 2.14 which, although designed for hunch repairs and gave useful general advice on the assessment of alternative materials for use in road construction (Sherwood, 2001).

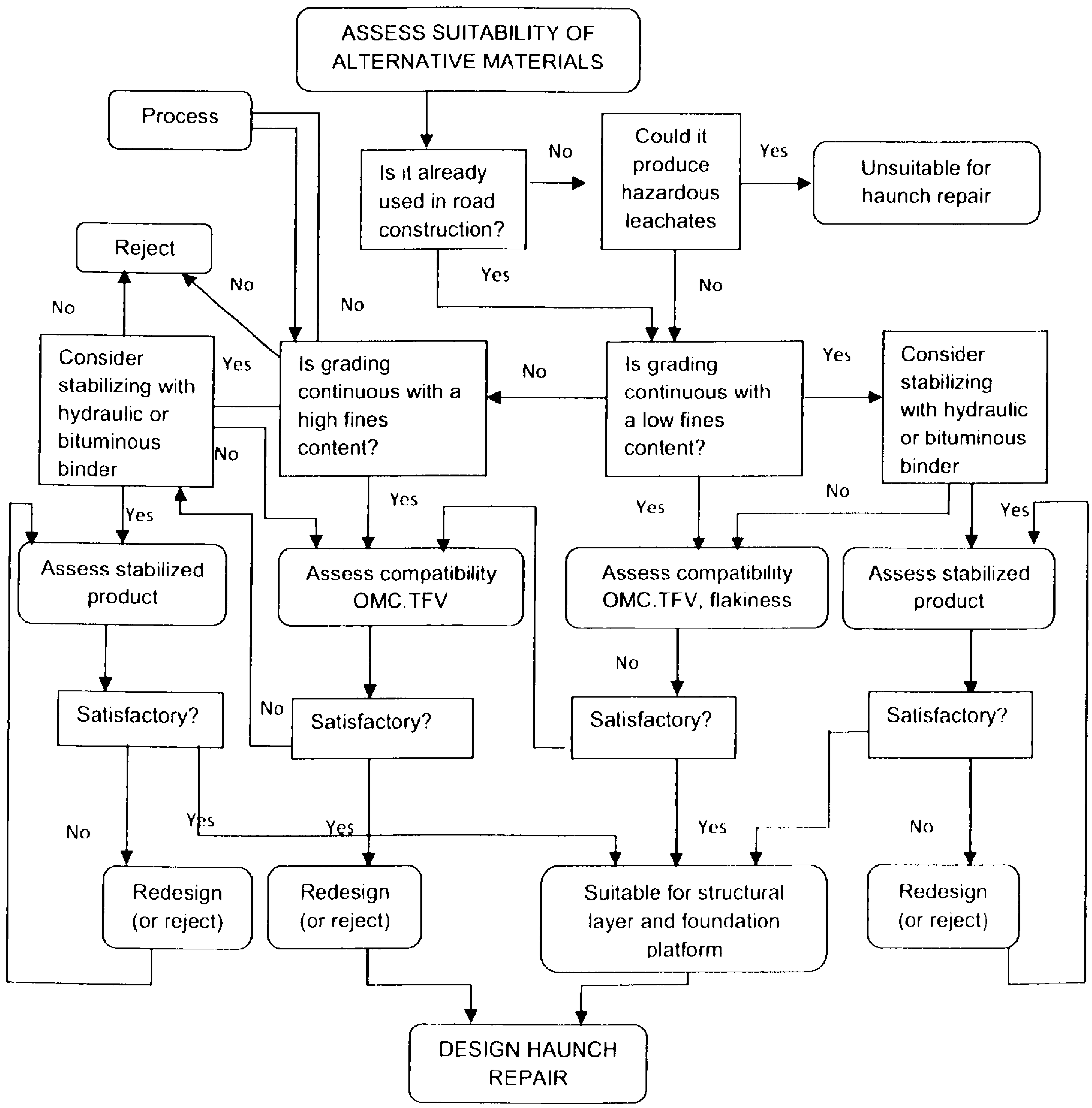


Figure 3.1 Decision chart for assessing suitability of alternative materials (From Sherwood, 2001)

BS 6543 (1985) has optimized use of by-products and waste materials in road construction according to Figures 2.15, 2.16 and 2.17. According to this flow diagram the early views and guidance in the choice of materials has been provided.

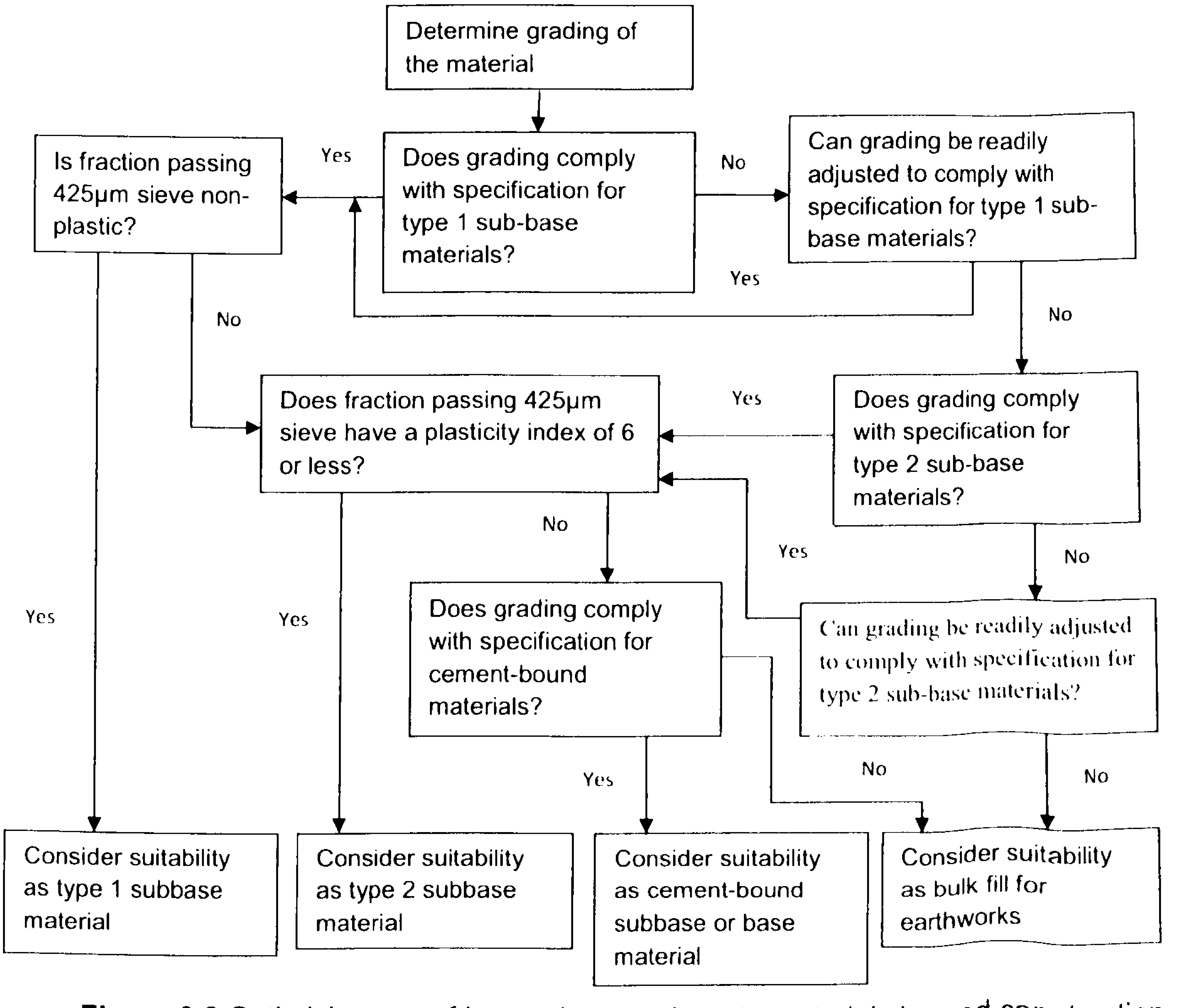


Figure 3.2 Optimizing use of by-products and waste materials in road construction (After BS 6543, 1985)

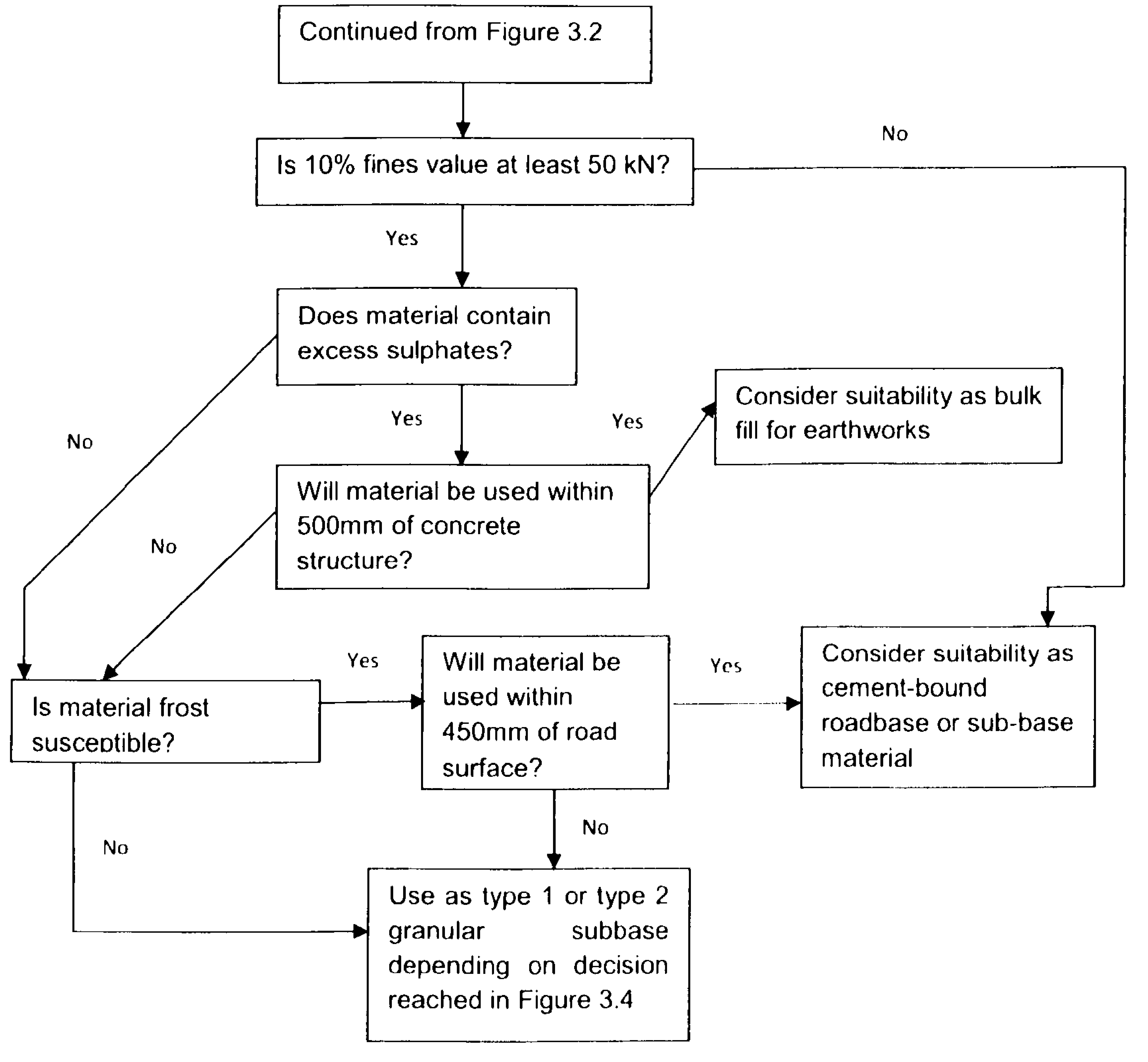


Figure 3.3 Determining suitability as type 1 and type 2 granular subbase (After BS 6543, 1985)

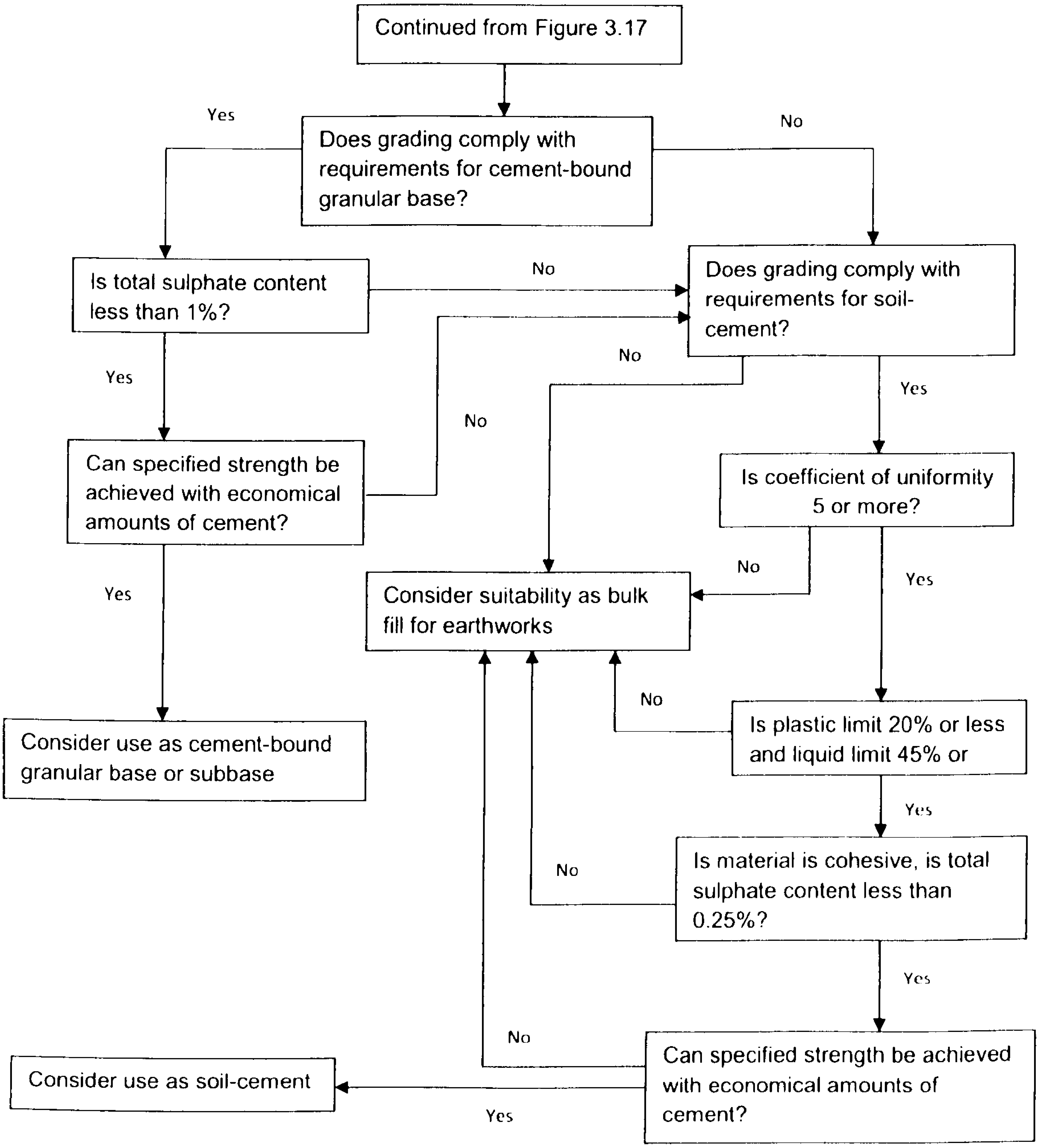


Figure 3.4 Determining suitability as cement-bound road base or subbase material (After BS 6543, 1985)

3.2 Research Methodologies

Research methodology has divided into two types: quantitative and qualitative research methods. The choice of methodology depends on the nature and aims of the research project, and the type of information available. A brief description of these two types of methodologies is discussed with a view to justifying the methodology adopted within this research project (Lambert, 2007).

Quantitative research is deductive and it tests existing theories with the aim of proving or disproving them. It can test theories composed of variables, numerical measurements and can be analysed statistically in order to validate the theories. The methods work well with large quantities of data and allow examination of a wide range of variables. Quantitative research is applied for achieving facts about a concept, a question, or a characteristic where factual evidence is collected and compared against theory. In quantitative research, the methods such as structured surveys, experimentation, research of secondary data and numerical methods could be used (Lambert, 2007).

Qualitative research is subjective and their methods aim to produce new theories and ideas. Information gathered can be categorised as exploratory or attitudinal. Explorative research is used when limited information is available. It is useful for (i) diagnosing a situation, (ii) screening alternative options, and (iii) discovering new ideas. Attitudinal research subjectively evaluates the subject such as opinions, views and perspectives of participants to a specific topic such as a variable or a question. After gathering the information, they will be quantified, but the approach used to gain the information tends to value the information as qualitative. Methods commonly used include (i) unstructured surveys (interviews and questionnaires) and (ii) action research (Lambert, 2007).

It is possible to combine the methodologies, which is termed triangulation research projects will tend to have a support to one of the two methodologies or use methods from both.

3.3 Adopted Research Methodology

Following a careful review of published literature, it became clear that different sources of information from various codes and standards give different recommendations as regards material properties. In this research, two different sets of design codes/Standards were selected from UK and Iranian practice as the bases of material testing. Various tests were then performed in accordance with the requirements of (i) HA for the UK Standards and (ii) AASHTO for the Iranian Standards. Subsequently, a system was designed for comparing the vast results of the tests obtained from the experimental process. Next, a scheme of tests was drawn up for assessing the performance of these aggregates and their suitability for application in highway subbases. Since the standard equipment required for performance related tests were expensive and therefore unavailable, the research focused on the use of experimental data complemented by a widely used qualitative model for evaluating performance of materials. Amongst the variables considered in the qualitative model were: (a) moisture condition (b) traffic loading and (c) weather conditions. Then in the next stage, computer modelling was carried out for hypothetical pavement layers made from the materials being investigated. As the main focus of computer analysis was on prediction of the loaded behaviour of subbases, the properties of all other layers were kept constant while those of the subbase were systematically varied. The results of the computer modelling were then compared, for the various mixes examined, in terms of the Mohr-Coulomb failure criterion. The schematic diagram in Figure 3.5 shows the sequence and interconnections between the activities involved in the whole research project, as a flow chart.

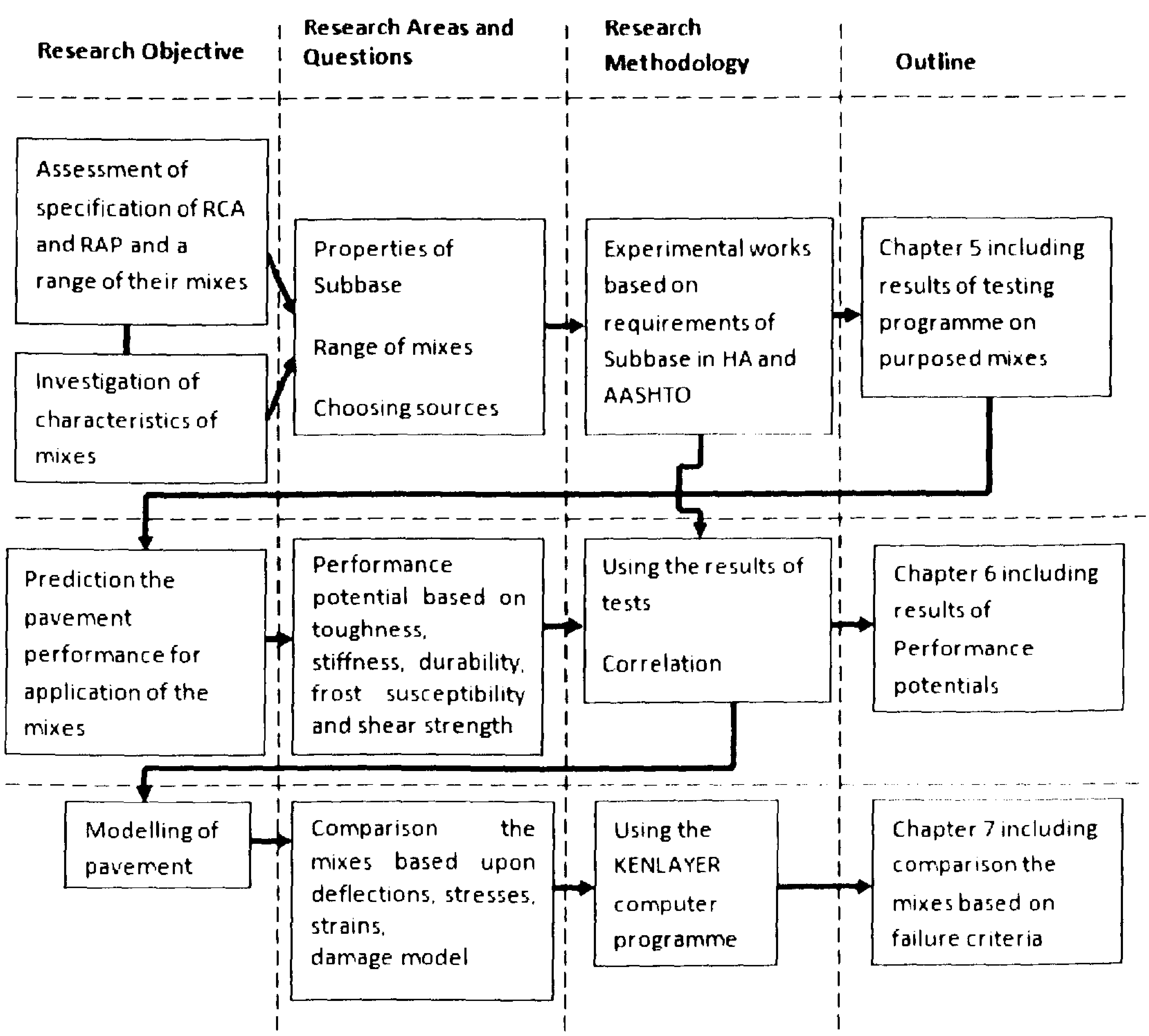


Figure 3.5 Schematic map describing the research context and process

CHAPTER 4 ROAD MATERIAL, DESIGN AND CONSTRUCTION

4.1 Road Construction

Roads are typically constructed with different layers of compacted materials in such a way that there is a general increase in material quality upwards from the lowermost to the uppermost layer (Figure 4.1). The surface layer is usually asphalt (aggregate with bitumen binder), although concrete can be used in this layer. The surface layer can be made with two or more layers depending on design specifications. The subsurface layers, of which there are usually two, are made from compacted materials. The materials used in each layer of the pavement are subject to design and standards specifications. Traditionally, these specifications cover many properties of the material including, for example, grading, particle strength, toughness and durability. The stringent specifications generally increase for the higher quality materials used in the upper pavement layers, which are designed to distribute high localized stresses through the layers. The upper layers are also affected by greater influence from other external factors such as temperature, weathering conditions and associated maintenance (Hill *et al.* 2001).

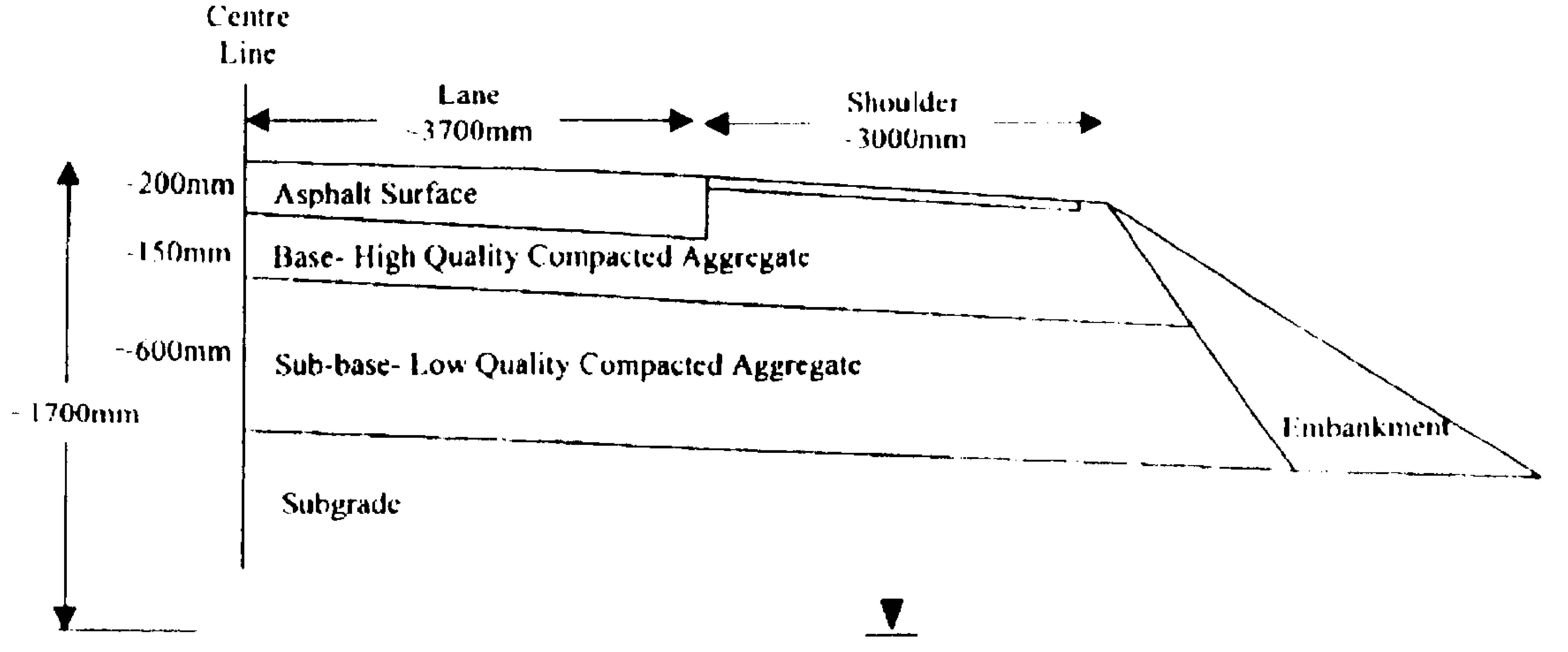


Figure 4.1 Schematic diagram of a typical road construction cross-section (not to scale-approximate dimensions) (From Hill *et al.* 2001).

4.2 The Role of the Pavement Foundation

A highway pavement is defined as a layered structure of selected materials placed and compacted on top of a foundation (HD25, 1994). The pavement foundation comprises the subbase with/without an underlying capping layer, as shown in Figure 4.2, on top of the subgrade (natural ground or constructed earthworks). This structure has to perform as both a short-term construction platform and also as a long-term durable structure for the overlying pavement. The requirements of the pavement foundation can be subdivided into four roles (HD25/94):

1) Resistance to the increase in permanent deformation within each layer over their design life.
2) Ability to disperse traffic loads (including construction traffic) to an amount that can be supported by underlying layers without failure.
3) Provision of a sufficiently stiff base on which the overlying layers can be compacted.
4) Provision of a sufficiently durable, tough and stiff base to support any overlying layers in the long term during in-service conditions including any environmental changes and weathering actions.

Pavement foundations must fulfill their four main roles without excessive cracking (for bound foundation), permanent deformation (rutting), or deterioration due to environmental factors including temperature (freeze/thaw and thermal expansion) and water (HD25/94 and IAN 73/06 Rev 1).

Interim Advice Note 73/06 in its first revision (2009) has divided the roles of foundation during construction from in-service roles below (IAN 73/06 Rev 1):

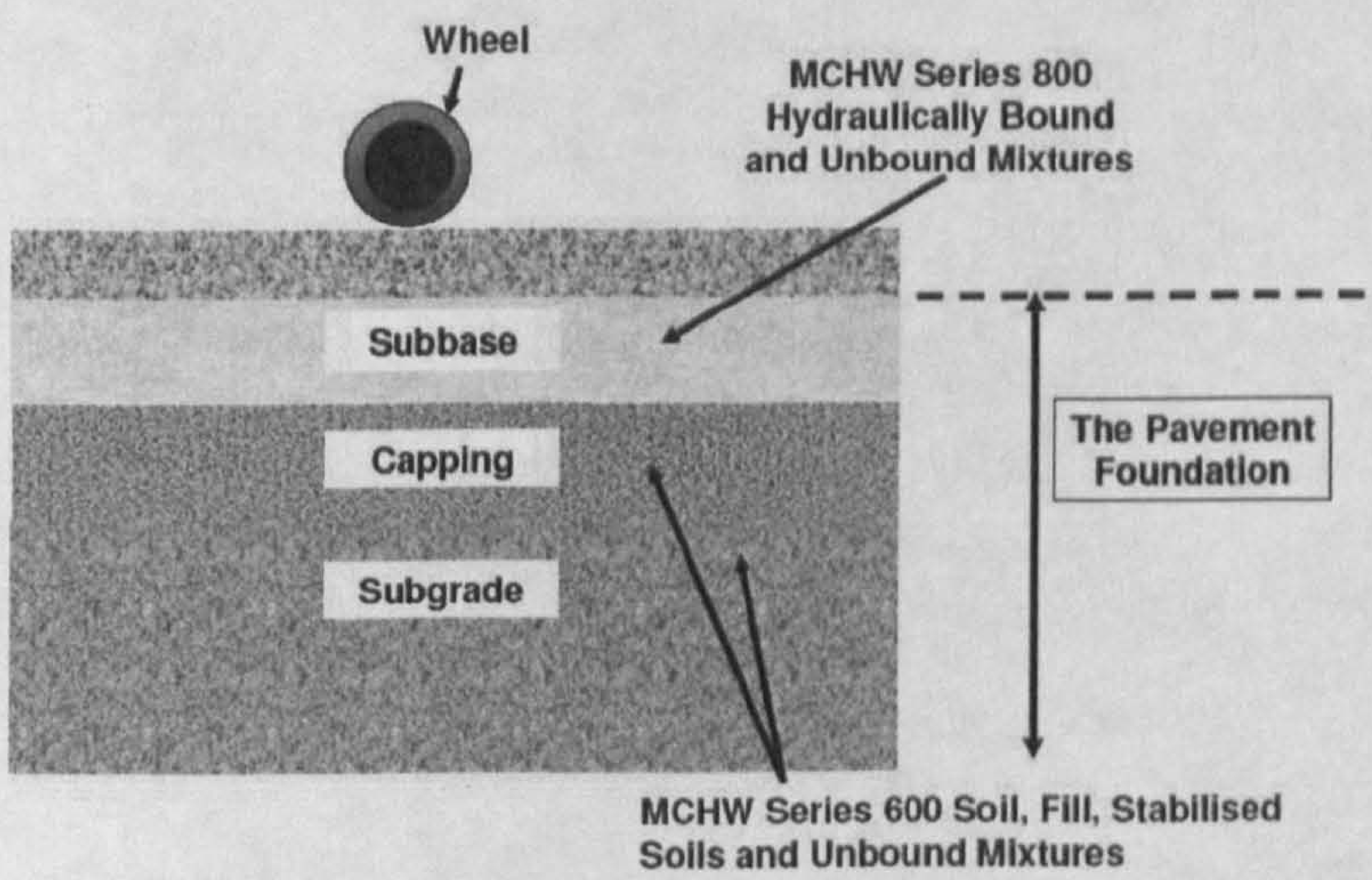


Figure 4.2 Definition of the pavement foundation and material options (From Edwards, 2007)

4.2.1 During construction

The requirements for a pavement during the construction stage and in the in-service life period are different. Below are important factors that should be taken into account for the construction stage (HD25/94):

a) The stresses in the foundation are relatively high, although the number of stress repetitions from construction traffic is relatively low and traffic is not as channelised as during the in-service life of the pavement.
b) Loads are introduced to the foundation by machineries such as delivery vehicles, pavers and other construction plant. At any level where such loading is applied, the strength and aggregates thickness should be adequate to resist the load without causing damage that might affect the future performance of the pavement.
c) Foundation layers should have adequate quality to resist environmental effects (e.g. precipitation, frost, high temperatures etc) without damage that could affect the long term performance of the pavement. Alternatively the foundation layers should be protected from such environmental effects.
d) Damage may happen in the form of rutting or other uneven deformation, cracking in hydraulically bound mixtures (including stabilised soils), or different forms of aggregates degradation.
e) The designs given in Interim Advice Note 73/06, besides the tests and material limitations given in the Highway Agency Specification, are intended to ensure that, such damage is avoided, under normal construction conditions
f) The foundation also should have adequate stiffness for the overlying pavement layers to be placed and sufficiently compacted.

4.2.2 In-service

Factors that should be taken into account for the service life of the pavement are (HD25/94):

a) The foundation should be able to resist large numbers of repeated traffic loads. If unprotected, pavement layers underneath the surfacing can be weakened by ingress of water from surface run-off or ground water. Deterioration of upper water-proofing layers will encourage ingress of water in to the lower layers.

b) It is essential that the foundation surface modulus assumed in the design, and relating to the choice of foundation class, is kept in the whole life of the pavement. Otherwise, deterioration of the upper pavement layers would typically occur more sooner than assumed.
c) It is also essential that excessive deformation does not accumulate within the foundation under repeated traffic loading, since this is a potential source of wheelpath rutting at the pavement surface.
d) The performance of the foundation will also depend on the design, construction and maintenance of the earthworks and their drainage system. It is vital that the drainage system ensures that there is no gathering of water in the pavement and foundation layers and that all excess moisture is allowed to exit from the foundation body.

4.3 Foundation Layers and Materials

The materials of pavement foundations are the natural ground or placed fill (subgrade) and unbound and/or bound layers (capping and/or subbase) (Edwards, 2007).

Up to the beginning of 2006, a limited number of materials in a restricted range of design options were the only choice according to the UK pavement design codes and standards used on Highways Agency (HA) projects(Edwards, 2007 and Nunn, 2004). Proven application of materials, for example "near concrete quality" aggregates within cement bound materials (Williams, 1986) or the designation of hard crushed rock for Type 1 unbound subbase materials, assessed via index tests, formed the mainstay of pavement foundation materials testing. Other materials could be used, but these had to be justified following a departure from standard practice. Research carried out by Edwards (2007) and Lambert (2007) on laboratory characterization of pavement foundation materials introduced laboratory apparatus and material guidance including the potential to use recycled and secondary materials which were incorporated into HA specification and associated documents.

The specifications and properties for the layers of a pavement foundation are in the following sections.

4.3.1 Subgrade

The subgrade is the bottom layer of the pavement foundation and generally consists of natural soil, which can either be natural or made ground (engineered fill). Subgrade classification tests have traditionally relied on empirical index-based tests such as CBR which compares the competence of a subgrade to that of a standard material. More recently, methodologies which show departure from the high empiricism of the CBR continue to be developed and used by various authorities. The new methodologies are mainly based on measurements of permanent deformation, suction and stiffness however they have generally been used as field methods so far (Edwards, 2007).

4.3.2 Capping Layer

Capping layers are generally either an unbound site aggregate (sourced from a borrow pit or from cutting), a stabilised soil of the HBM family or an imported materials. If required for the purpose of protection of a weak subgrade or to provide a riding surface for construction traffic, the capping layer (or improved soil) is laid between the subgrade and subbase. The aim is to increase the stiffness modulus and strength of the formation, on which the subbase will be placed. Capping materials with a laboratory CBR of at least 15% should make a sufficient platform for construction of the subbase when compacted to the appropriate and designed thickness (HD 25/94).

This layer in conjunction with subbase materials have a low thermal conductivity and therefore placement of these materials on the subgrade will insulate it, to some level, from damage by frost. Capping layer as a part of foundation materials may also protect the subgrade from the effects of rainfall (Chaddock and Roberts, 2006). The specification for these materials is given in MCHW Series 600 (2009). The permitted constituents and material properties of capping materials are based upon British Standards based on physical, chemical and geometrical properties (grouped under the term index properties) of the aggregates to meet performance requirements over a range of subgrade CBR values used in HD25/94 and IAN 73/06. The assumed in situ performance is defined by a CBR measurement.

4.3.3 Subbase Layer

The subbase layer generally consists of aggregates, used in either an unbound form or as a HBM. This layer in conjunction with capping materials have a low thermal conductivity and therefore placement of these materials on the subgrade will insulate it, to some level, from damage by frost. Subbase layer as a part of foundation materials may also protect the subgrade from the effects of rainfall (Chaddock and Roberts, 2006).

Granular and cemented subbases are permitted for use in flexible and flexible composite pavements but only cemented subbases are permitted to use in rigid and rigid composite pavements. The grading for unbound granular subbase is designed to prepare a dense layer of relatively high stiffness modulus, which is reasonably impermeable and will thus shed rain water during construction, given sufficient fall. It is not necessarily free draining and may exit through suction, and thus increase in moisture content. Granular subbase with a laboratory CBR value of at least 30% should provide an adequate platform for construction of the upper layers of pavement when compacted to the appropriate and designed thickness (HD25/94).

4.4 Pavement Design and Maintenance

4.4.1 Background

Current UK practice in pavement design and maintenance has developed from a combination of practical experience and laboratory research as well as field trials. For a number of decades the Transport Research Laboratory (TRL) has been at the forefront of research and development of highway materials, testing and standardization. Details of UK research findings related to the design and performance of flexible pavements are available in TRRL Report LR 1132 (1984). Significant developments have also happened in other countries, particularly connected with the analytical or mechanistic approach to design. This is based on the traditional structural design philosophy, which requires an understanding of material behavior under traffic loading using a sound engineering and scientific basis (HD23/99).American Association of State Highway and Transportation Officials (AASHTO) in association with National Cooperative Highway Research Program (NCHRP) have published mechanistic-empirical design procedures for new and reconstructed flexible pavements. The overall design process for asphalt pavements is illustrated in Figure 4.3 (NCHRP, 2004).

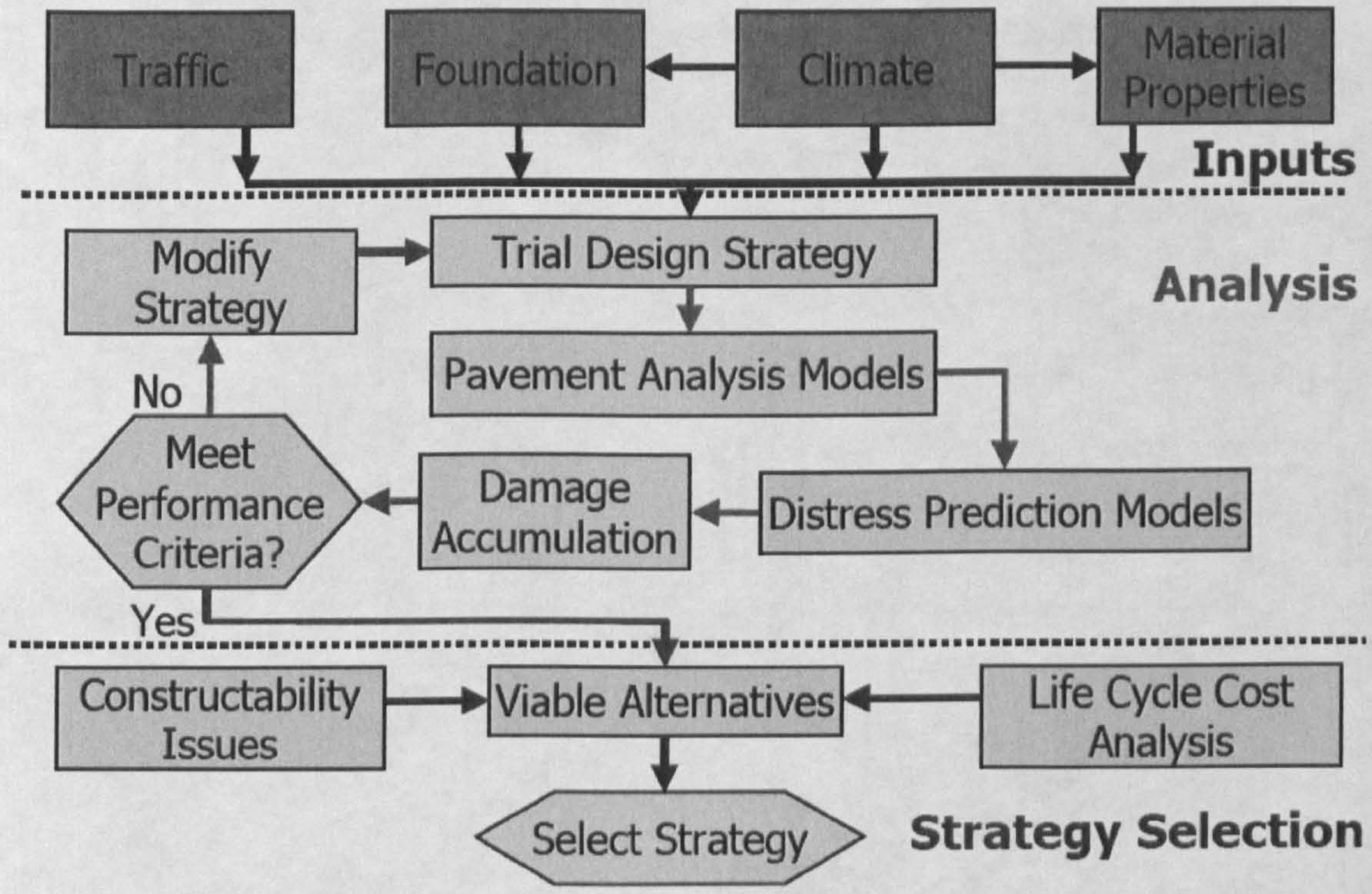


Figure 4.3 Overall design process for flexible pavements (From NCHRP, 2004)

4.4.2 Pavement Components

Figure 4.4 illustrates two typical cross-sections of road pavements in the U.K. The underlying sub-grade soil (cut or fill), capping (if used) and subbase are the layers of the foundation, the platform upon which the more expensive and structurally important layers are constructed. This platform is designed to be of a certain minimum standard quality whatever the underlying soil conditions. It is not a drainage layer although it does itself require to be sufficiently drained since it is never completely impermeable (HD23/99).

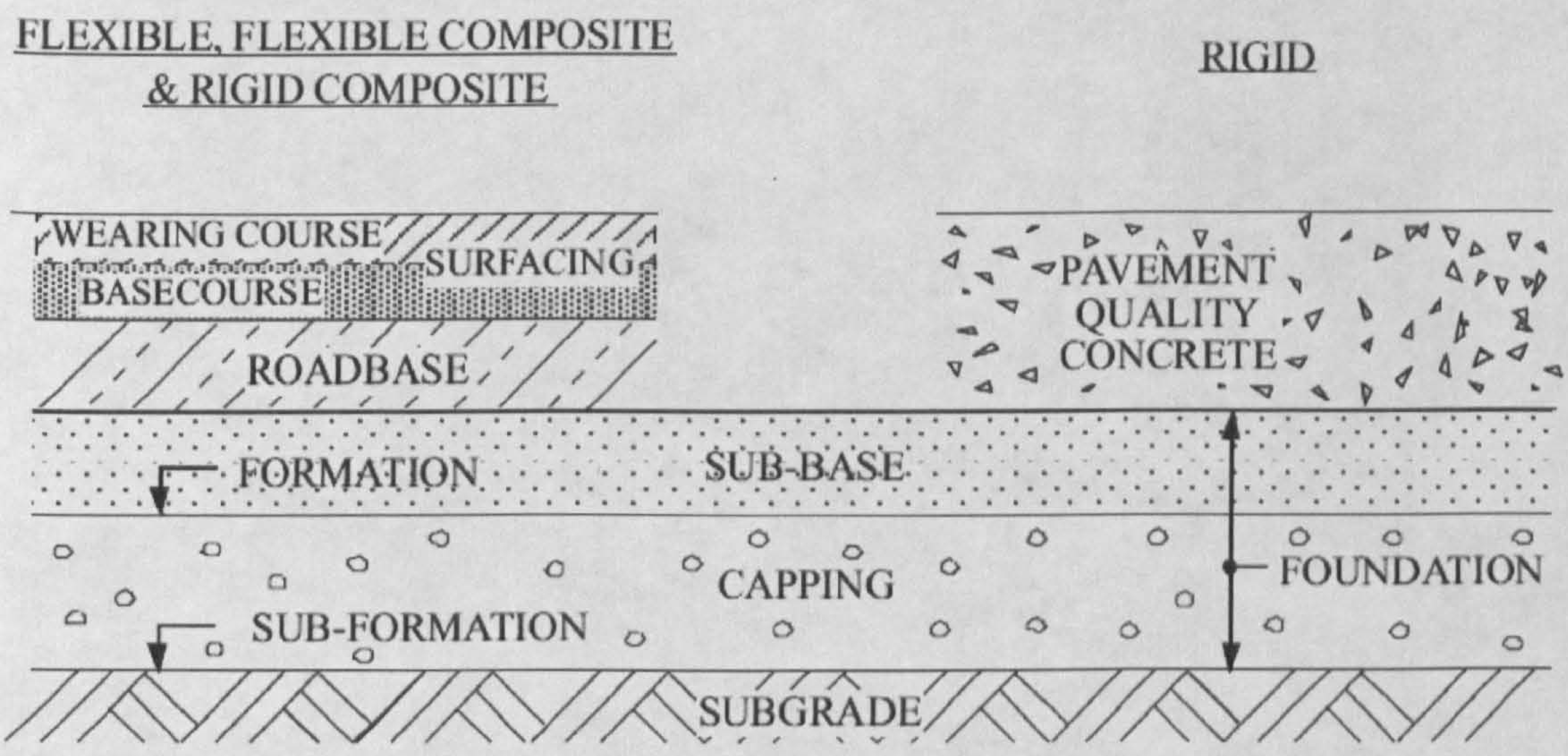


Figure 4.4 Typical pavements (From HD23/99)

The Road-base is the main structural layer of the pavement, needed to spread the applied traffic loading so that the underlying materials do not become overstressed. It must be able to resist the stresses and strains generated within itself without greater or rapid deterioration of any kind.

The function of the Surfacing is to produce a weatherproof and safe riding surface with appropriate resistance to skidding and to a variety of defects such as cracking, squeezing, flowing etc. For this reason, proper strength, toughness and durability under traffic are essential. In the case of concrete roads, the surfacing and road-base may be installed as a combined layer (HD23/99).

4.4.3 Pavement types

Four different types of pavements are defined in HD23/99:

a) Flexible: The surfacing and road-base materials are bound with bituminous binder.

b) Flexible Composite: The surfacing and upper road-base (if used) are bound with bituminous binder on a road-base or lower road-base of cement bound material.

c) Rigid Pavement quality concrete is used for the combined surfacing and road-base. The concrete can be:

- Jointed reinforced (JRC)
- Continuously reinforced (CRCP)

d) Rigid Composite: Continuously reinforced concrete road-base (CRCR) with bituminous surfacing.

The design procedure by AASHTO and NCHRP offers the capability to consider six structural sections for asphalt pavement system as illustrated in Figure 4.5 (NCHRP, 2004):

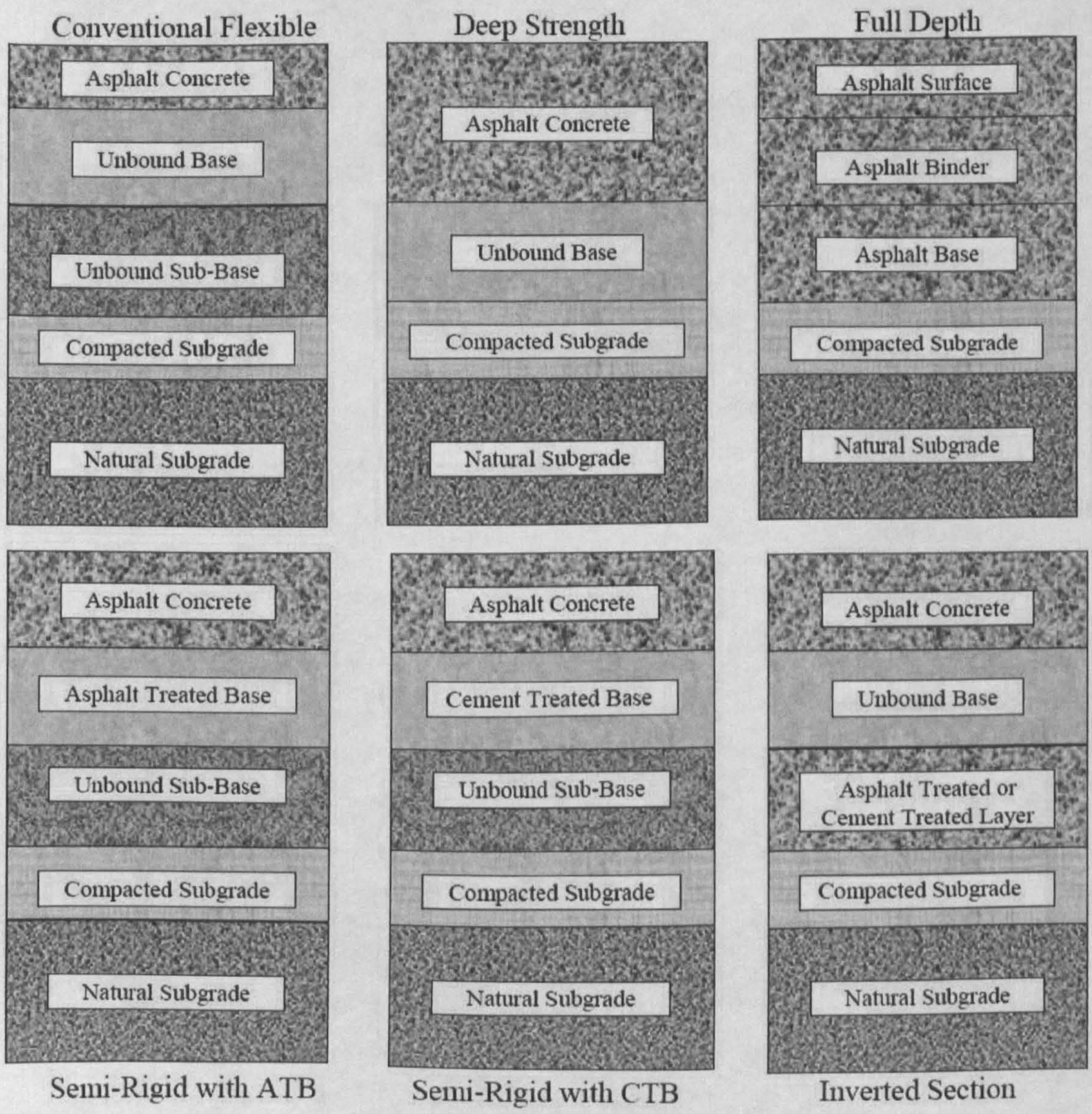


Figure 4.5 Illustration of possible asphalt pavement layered systems (From NCHRP, 2004).

4.4.4 Pavement performance

Pavements usually fail gradually rather than suddenly because of accumulative weakening under traffic loading. This deterioration, unless stemmed, may continue until such a state whereby the serviceability of the pavement has degraded to such an extent that failure is deemed to have occurred. The pace of deterioration often accelerates as failure is approached. This will appear by an unacceptable rate of rutting, general unevenness, cracking, crazing etc. In the case of the surfacing layer, loss of skid resistance can be equivalent to failure. Generally long life pavements have thicker bound layers constructed on a good foundation, sustain their strength or become stronger with time, instead of weakening gradually under traffic loads. The strength of such types of pavements is intentionally designed to be above a threshold value in order to give a very long structural service life. As a result, any distresses or damages in such pavements would only appear in the form of surface cracks and ruts, which can easily be treated before affecting the structural integrity of the road.

It is good practice to ensure when carrying out major maintenance or strengthening at a location in a pavement, the structural integrity of other layers is preserved.

To monitor the performance of a pavement, it is essential to use of a number of assessment machines and methods. These include the (i) High Speed Road Monitor, (ii) the Deflectograph, (iii) FWD, (iv) Sideway Force Coefficient Routine Investigation Machine (SCRIM), (v) PRIMA (Portable FWD), and (vi) Visual Condition Surveys (VCS) . In this way the appropriate time table should be chosen for the various necessary maintenance processes which all pavements eventually require HD23/99.

4.4.5 Maintenance

Clearly any of the three main components of a pavement (Foundation, Road-base, and Surfacing) can deteriorate, leading to a reduction in the quality of performance of the overall structure. Depending on the mode of deterioration, inferred from the various pavement assessment processes, maintenance measures ranging from surface treatment through to total reconstruction may be necessary.

Whichever type of maintenance measure is to be adopted, it is always the most important that the correct methods are followed to make the repair or strengthening as effective and long-lasting as possible HD23/99.

4.5 Performance Foundation Designs in the UK

The four proposed foundation classes for flexible and flexible composite pavements were specified in Appendix A of TRL615 as illustrated in Figure 4.6:

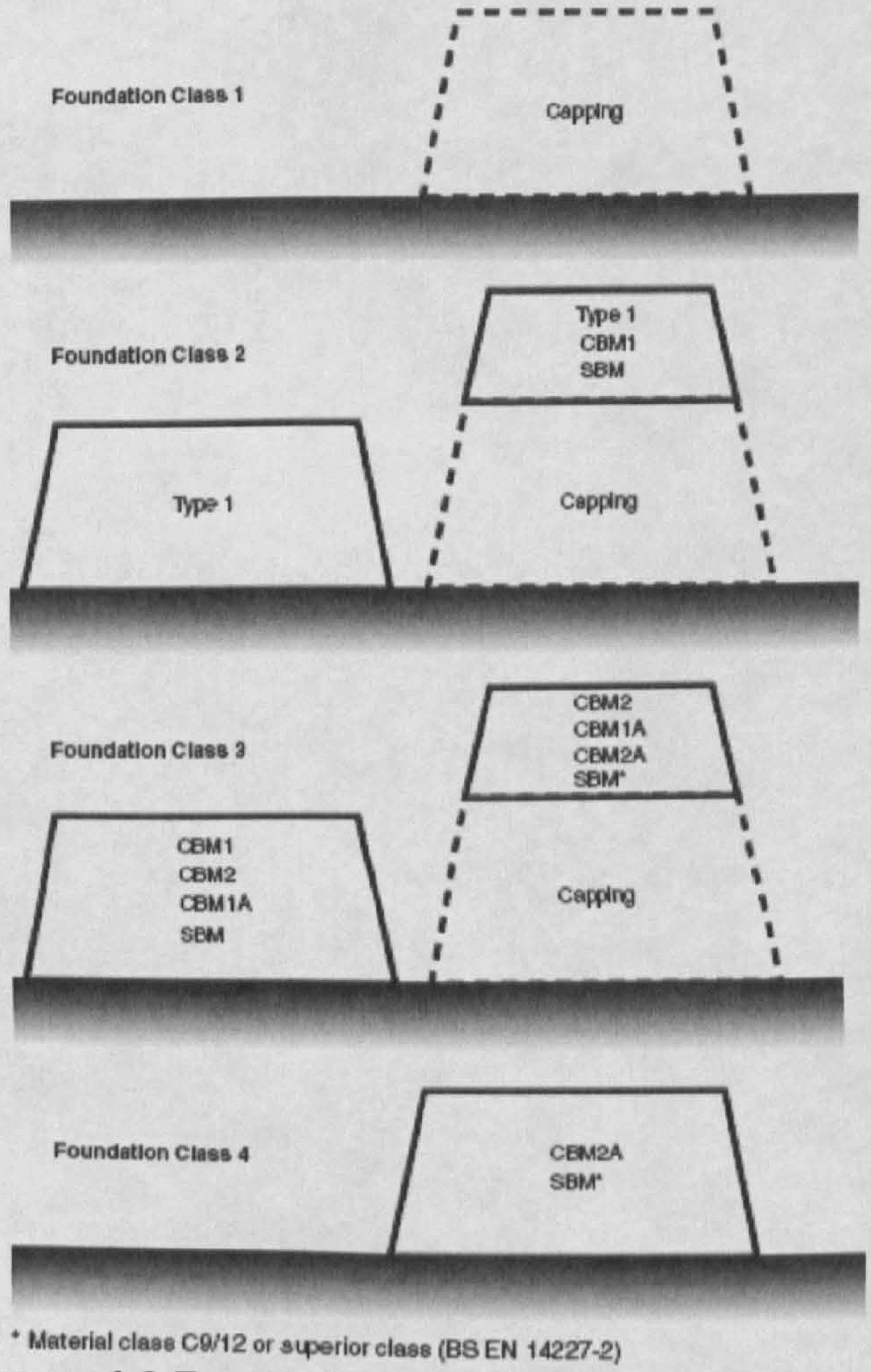


Figure 4.6 Foundation classes (From Nunn, 2004)

For all traffic categories, Foundation Class 2 (FC2) is a subbase only or subbase on capping design that is considered as equivalent to the standards of unbound granular foundation (Nunn, 2004). The Foundation Surface Modulus values for Foundation Classes have been defined below (IAN 73/06 Rev 1):

- Class 1 ≥ 50MPa
- Class 2 ≥ 100MPa
- Class 3 ≥ 200MPa
- Class 4 ≥ 400MPa

Figure 4.7 indicates different Modulus Definition required to design:

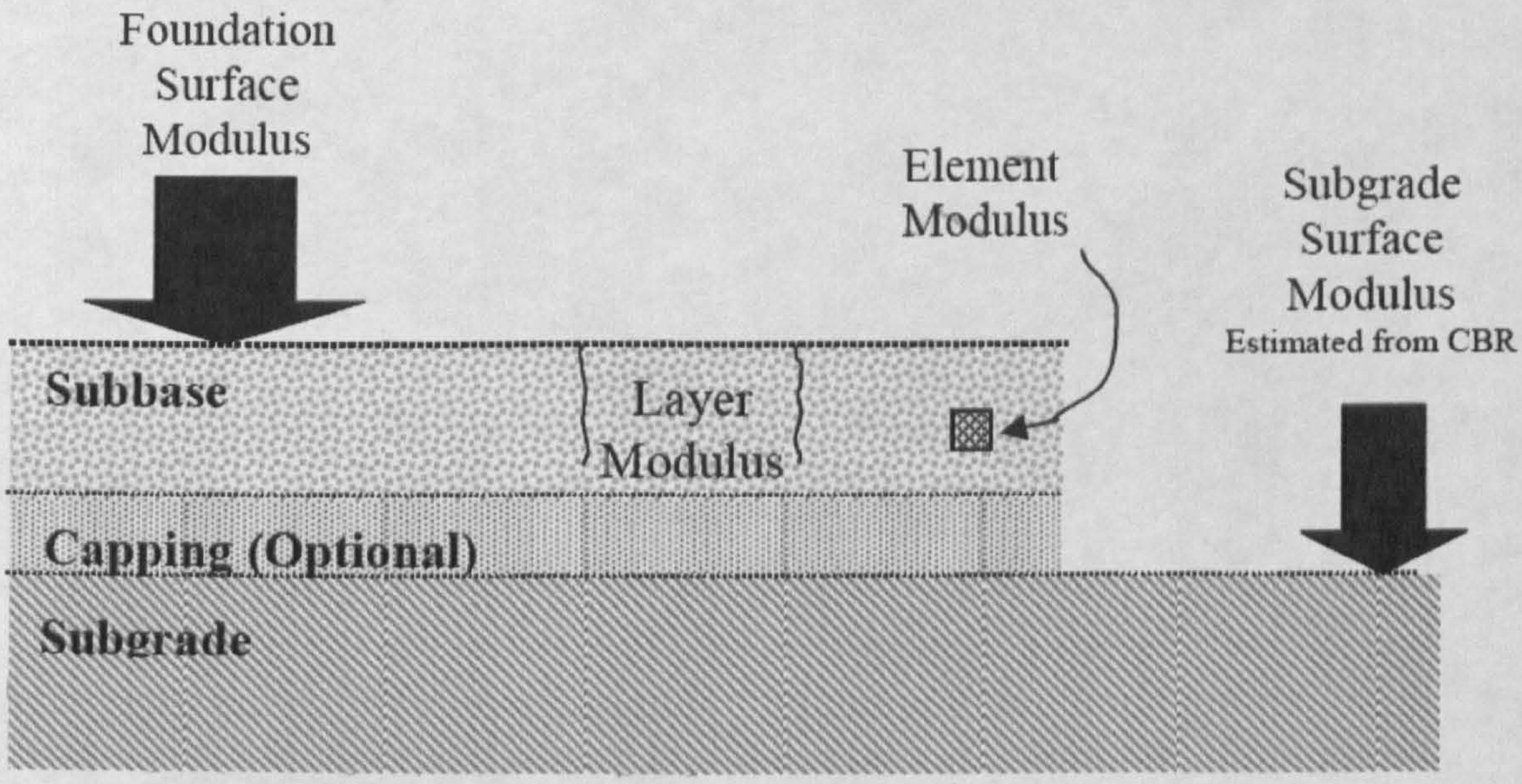


Figure 4.7 Modulus definitions (From IAN 73/06 Rev. 1)

The principle objectives for developing Performance Designs for foundations and the Performance Related Specification for foundations are (IAN 73/06 Rev. 1):

- Facilitating the efficient application of a vast range of resources, incorporating natural, secondary and recycled materials as both binders and aggregates.
- Providing some assurance to achieve the material performance assumption made during the design process.
- Recognising the structural contribution of improved foundation performance and hence permitting the adjustment in thickness of the upper layers of pavement.

The philosophy of Performance Design is based upon performance testing to confirm the physical properties that are critical to the design process and will discuss in Chapter 5.

The materials used in pavement foundations have a wide range of properties that impact on performance (e.g. particle size, strength, elastic stiffness, stress dependency, curing rates). However, it is not practical to perform testing for all of these properties and a single foundation surface modulus performance test will be a practical solution.

The process of designing, constructing and testing a Performance Related Foundation according to Interim Advice Note 73/06 Revision 1 (2009) is summarised in Figure 4.8.

Performance Foundation Design should only be used in association with the Performance Related Specification for Foundations. This Specification needs a series of information to be collected during construction of the pavement foundation. The main tests and measurements are (IAN 73/06 Rev 1):

- Strength measurement (CBR value) at the top of the exposed subgrade, immediately before construction of the foundation upper layers, throughout the works;
- Material density and the actual thickness for each phase of foundation construction, throughout the works;
- Compliance with the relevant material specifications from Specification Series in MCHW1 Series 600 and 800 for each phase of foundation construction, throughout the works;
- The measurement of the Foundation Surface Modulus at the top of the foundation level, throughout the works should be performed immediately before the construction of the upper pavement layer.

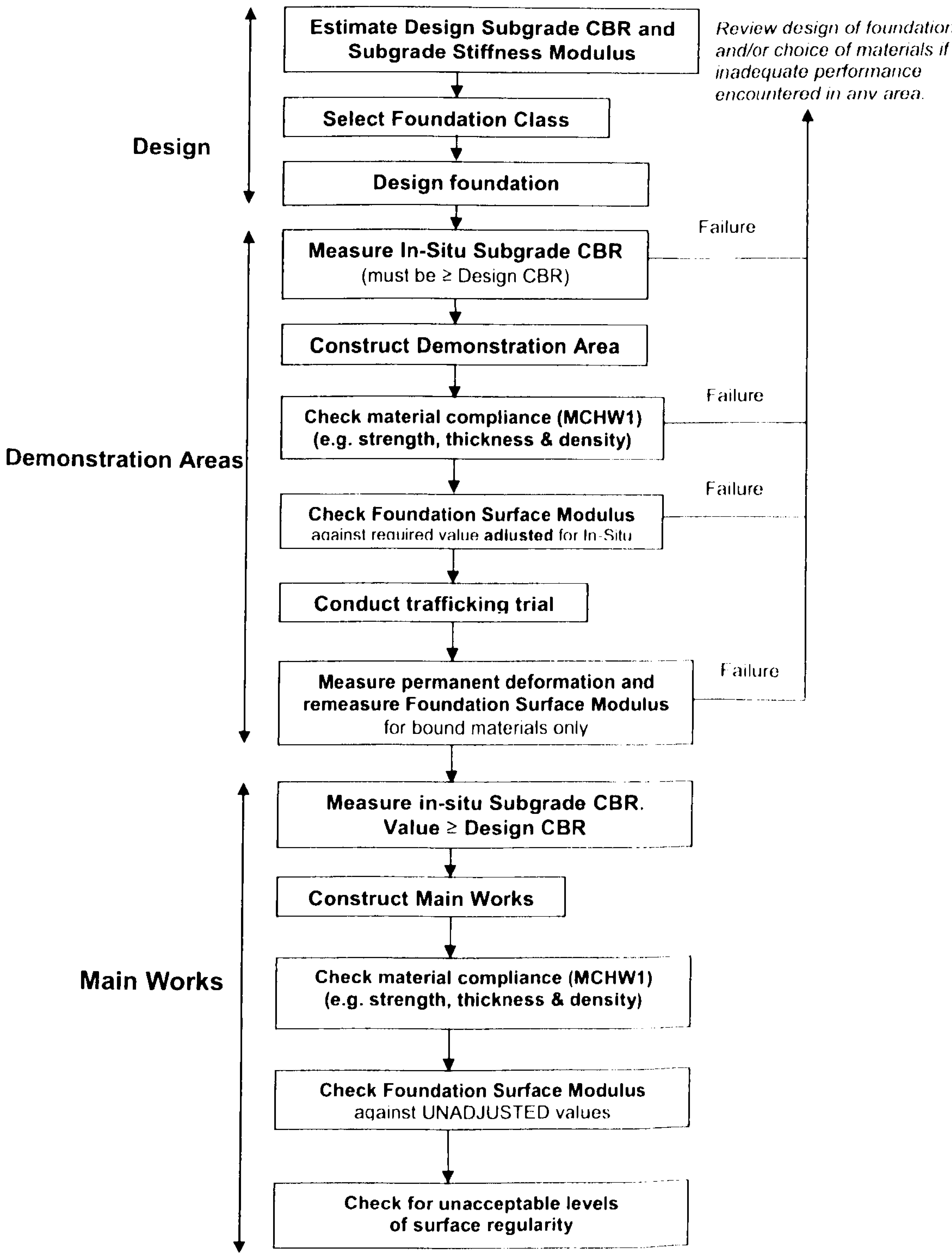


Figure 4.8 Summary Flowchart for Performance Related Foundations (From IAN 73/06 Rev. 1)

4.6 AASHTO Performance Related Tests for Unbound Materials

A research was performed by NCHRP (NCHRP 453, 2006) in conjunction with AASHTO on performance-related tests of aggregates for use in unbound pavement layers in this project. Amongst the salient outcomes of the research were:

- Fatigue cracking, rutting, corrugations, depressions, and frost heave are distresses connected with poor performance of unbound granular layers in flexible pavements.
- Distresses are results of poor performance of unbound granular layers in rigid pavements including cracking, pumping/faulting, and frost heave.
- Properties of unbound granular aggregates used as base and subbase layers that affect pavement performance are shear strength, toughness, abrasion, stiffness, durability, frost susceptibility, and permeability.
- Shear strength may be one of the most influential factors controlling the performance of an unbound aggregate pavement layer. Because stiffness is directly connected to shear strength, it has a similarly large influence on performance.
- The performance of aggregates in unbound pavement layers can be investigated using the tests listed in Table 4.1:

Table 4.1 Performance-related tests (From NCHRP 453, 2006)

Aggregate Property	Test Methods
Screening Tests	(a) Particle size analysis (b) Consistency limits (c) Moisture–Density relationship (d) Flat and elongated particles (e) Uncompacted voids
Durability	(a) Magnesium Sulfate Soundness
Shear Strength Tests	(a) Triaxial tests: (i) saturated and (ii) dry (b) California Bearing Ratio
Stiffness	(a) Resilient Modulus: (i) saturated and (ii) dry
Toughness and abrasion resistance	(a) Micro-Deval Test
Frost susceptibility	(a) Tube Suction Test

- The potential performance of an aggregate can be predicted taking into account specific traffic and climatic conditions. The method, which has so far been tested on limited data, involves selecting design parameters based on the traffic and climate conditions given in Tables 4.2 and 4.3.

Table 4.2 Traffic levels (From NCHRP 453, 2006)

Traffic Level	Number of ESALs/year
Low traffic	<100,000 ESALs/year
Medium traffic	100,000–1,000,000 ESALs/year
High traffic	>1,000,000 ESALs/year

ESAL in Table 4.2 is the acronym for Equivalent Single Axle Load.

The climatic conditions of moisture and freezing were chosen based upon the AASHTO definitions. Table 4.3 shows the significance levels of traffic, moisture, and climate combinations on a scale of 1 to 4, where 4 is the most significant and 1 is the least significant on potential of aggregate performance.

Table 4.3 Significance levels of traffic, moisture and temperature combinations on aggregate performance potential (From NCHRP 453, 2006)

Temperature Condition	Moisture Condition	Traffic		
		High	Medium	Low
Freezing	High	4	4	3
	Low	4	3	2
Non Freezing	High	3	2	2
	Low	3	2	1

Recommended parameter values from prescribed tests for use in assessing the potential performance of virgin aggregates under different combinations of traffic (high, medium or low), moisture (high or low) and temperature conditions (freezing or non-freezing) are listed in Table 4.4.

Table 4.4 Recommended tests and test parameters for assessment of aggregate performance potential (From NCHRP 453, 2006)

TESTS					
Traffic	H, H, M	M, H, H, L	M	M, L, L	L
Moisture	H, L, H	L, H, L, H	H	L, L, H	L
Temperature	F, F, F	F, NF, NF, F	NF	NF, F, NF	NF
Screening Tests					
Gradation, Cu	≥6	≥6	≥2		≥2
Max. Aggregate Size	≥3/4"	≥3/4"	≥3/4"		≥3/4"
Minus #20	≤ 5	≤ 8	≤ 10		0 ≤ # ≤ 12
Atterberg Limits	Nonplastic	Nonplastic	Nonplastic		Nonplastic
Uncompacted Voids, Uc	<35	<45	<55		Uc < 65
Flat and elongated 5:1 m	<0.10	<0.10	<0.32		<0.32
Toughness/Abrasion					
Micro-Deval, MD	≤ 5	≤ 15	≤ 30		≤ 45
Durability					
Sulfate Soundness, S	≤ 13	≤ 30	≤ 30		≤ 45
Frost Susceptibility					
Tube Suction Test, DCV	≤ 7	≤ 10	≤ 15		≤10
F-Category	F-1	≥F-2	≥F-2		≥F-3
Shear Strength					
Static Triaxial Test					
Max σ_d at OMC, σ_C=5 psi	≥ 100	≥ 60	≥ 40	≥ 25	
Max σ_d at Sat,σ_C=15 psi	≥ 180	≥ 135	≥ 90	≥ 60	
Reapeted Load Test					Not Required
Max σ_d at OMC, σ_C=15 psi	≥ 180	≥ 160	≥ 130	≥ 90	
Max σ_d at OMC, σ_C=15 psi	≥ 180	≥ 160	≥ 125	≥ 60	
Stiffness					
Resilient Modulus, M_R	≥ 60 ksi	≥ 40 ksi	≥ 32 ksi	≥ 25 ksi	

For recycled aggregates in unbound pavement layers, NCHRP Report 598 (2008) proposed a different set of recommendations, alternative to Table 4.4 above, for defining the performance-related tests. The objective of the research in the NCHRP Report 598 was to develop a process for selecting recycled hot-mix asphalt (HMA) and Portland cement concrete (PCC) materials and performing performance-related tests on these materials before use in unbound layers.

The NCHRP Report 598 describes various types of distress, along with relevant contributing factors, in flexible pavements as summarized in Table 4.5. The cross-relationships between the performance parameters, aggregate properties and relevant laboratory test methods are listed in Table 4.6, for flexible and rigid pavements.

Table 4.5 Flexible pavement performance parameters and contributing factors (From NCHRP Report 598, 2008)

Distress	Description of distress	Unbound layer failure mechanism	Contributing factors
Fatigue Cracking	Fatigue cracking first appears as fine, longitudinal hairline cracks running parallel to one another in the wheel path and in the direction of traffic; as the distress progresses the cracks interconnect, forming many-sided, sharp angled pieces; eventually cracks become wider and, in later stages, some spalling occurs with loose pieces prevalent. Fatigue cracking occurs only in areas subjected to repeated loadings	Lack of base stiffness causes high deflection/strain in the HMA surface under repeated wheel loads, resulting in fatigue cracking of the HMA surface. High flexibility in the base allows excessive bending strains in the HMA surface. The same result can also be due to inadequate base thickness. Changes in base properties (e.g., moisture induced) with time can render the base inadequate to support loads.	Low modulus of the base layer Low density of the base layer Improper gradation High fines content High moisture level Lack of adequate particle angularity and surface texture Degradation under repeated loads and freeze-thaw cycling
Rutting/Corrugation	Rutting appears as a longitudinal surface depression in the wheel path and may not be noticeable except during and following rains. Pavement uplift may occur along the sides of the rut. Rutting results from a permanent deformation in one or more pavement layers or subgrade, usually caused by consolidation and/or lateral movement of the materials due to load.	Inadequate shear strength in the base allows lateral displacement of particles with applications of wheel loads and results in a decrease in the base layer thickness in the wheel path. Rutting may also result from densification of the base due to inadequate initial density. Changes in base (mainly degradation producing fines) can result in rutting. The base can also lose shear strength from moisture-induced damage, which will cause rutting.	Low shear strength Low base material density Improper gradation High fines content High moisture level Lack of particle angularity and surface texture Degradation under repeated loads and freeze-thaw cycling High moisture content coupled with traffic can contribute to stripping
Depressions	Depressions are localized low areas in the pavement surface caused by settlement of the foundation soil or consolidation in the subgrade or base/subbase layers due to improper compaction. Depressions contribute to roughness and cause hydroplaning when filled with water.	Inadequate initial compaction or nonuniform material conditions result in additional reduction in volume with load applications. Changes in material conditions due to poor durability or frost effects may also result in localized densification with eventual fatigue failure.	Low density of base material Low shear strength of the base material combined with inadequate surface thickness
Frost Heave	Frost heave appears as an upward bulge in the pavement surface and may be accompanied by surface cracking, including alligator cracking with resulting potholes. Freezing of underlying layers resulting in an increased volume of material cause the upheaval. An advanced stage of the distortion mode of distress resulting from differential heave is surface cracking with random orientation and spacing	Ice lenses are created within the base/subbase during freezing temperatures, particularly when freezing occurs slowly, as moisture is pulled from below by capillary action. During spring thaw large quantities of water are released from the frozen zone, which can include all unbound materials.	Freezing temperatures Source of water Permeability of material high enough to allow free moisture movement to the freezing zone, but low enough to also allow suction or capillary action to occur

Table 4.6 Links between aggregate properties and performance (From NCHRP Report 598, 2008)

Pavement type	Performance parameter	Related aggregate property	Test measures
Flexible	Fatigue Cracking	Stiffness	Resilient modulus, Poisson's ratio, gradation, fines content, particle angularity and surface texture, frost susceptibility degradation of particles, density
	Rutting Corrugations	Shear Strength	Failure stress, angle of internal friction, cohesion, gradation, fines content, particle geometrics (texture, shape, angularity), density, moisture effects
	Fatigue Cracking, Rutting, Corrugations	Toughness	Particle strength, particle degradation, particle size, gradation, high fines
		Durability	Particle deterioration, strength loss
		Frost susceptibility	Permeability, gradation, percent minus 0.02 mm size, density, nature of fines
		Permeability	Gradation, fines content, density
Rigid	Cracking, Pumping, Faulting	Shear Strength	Failure stress, angle of internal friction, cohesion, gradation, fines content, particle geometrics (texture, shape, angularity), density, moisture effects
		Stiffness	Resilient modulus, Poisson's ratio
		Toughness	Particle strength, particle degradation, gradation
		Durability	Particle deterioration, strength loss
		Permeability	Gradation, fines content, density
	Cracking, Pumping, Faulting, Roughness	Frost Susceptibility	Permeability, gradation, percent minus 0.02 mm size, density, nature of fines

To ensure specification compliance, and to investigate the strength and durability properties, laboratory tests should be carried out to establish the characteristics of aggregates as construction materials. NCHRP developed a series of laboratory tests, mostly along empirical lines, to estimate performance and to identify potentially poor performers. The proportions and properties of RAP and RCP in the mixes of unbound pavement layers have a significant impact on the performance of the layer. Because of the particulate nature of unbound aggregate layers, their mechanical properties also depend on the stress state and environmental conditions. For pavement applications, tests have been developed to measure four categories of aggregate properties and characteristics: (a) stiffness or modulus, (b) shear strength, (c) permanent deformation and (d) durability.

Table 4.7 lists the potential test methods and their comparative rankings in applicability to the evaluation of recycled aggregates.

Table 4.7 Rating of potential test methods for evaluating recycled aggregates (From NCHRP Report 598, 2008)

Property measured	Test	Performance predictability	Accuracy	Practicality	Complexity	Precision	Cost	Composite
Shear Strength	Static Triaxial Shear	F	G	H	FS	G	M	H
	Repeated Load Triaxial	G	G	H	C	G	M	H
	Texas Triaxial	F-G	G	M	FS	F	M	M
	Illinois Rapid Shear	F-G	G	M	FS	G	M	M-H
	Confined Compression	F	F	M	S	F	L	M
	Direct Shear	F	F	L	FS	F	M	M
	Gyratory Shear	F	F	M	C	F	M	M
	k-Mould	G	G	M	C	F	M	M
	CBR	F	F	M	S	F	L	M
	Hveem Stabilometer	F	F	M	S	F	L	M
	Hollow Cylinder	G	G	L	VC	L	H	L
	Dynamic Cone Penetrometer	F	F	M	S	F	L	M
	Lab Rut-Tester	G	F	L	C	F	H	M
Stiffness	Resilient Modulus	G	G	H	C	C	M	H
	Var. Conf. Pres. Modulus	F	F	L	VC	F	H	L
Frost Susceptibility	Frost Susceptibility Test	F	F	L	C	P	H	L
	Tube Suction Test	G	G	M	FS	G	M	H
	Index Tests	F	G	H	S	F	L	H
Permeability	Constant Head	F	F	M	FS	F	L	M
	Falling Head	F	F	H	FS	F	L	M
	Pressure Chamber	F	F	H	FS	F	M	M
	Horizontal Permeameter	F	F	H	FS	G	M	M
Toughness	LA Abrasion	F	F	M	S	F	L	M
	Aggregate Impact Value	F	F	F	S	F	L	M
	Aggregate Crushing Value	F	F	F	S	F	L	M
	Aggregate Abrasion Value	P	P	P	FS	P	L	L
	Micro-Deval	G	F	M	S	F	L	M
	Durability Mill	P	P	P	FS	F	M	L
	Gyratory Test	P	P	P	FS	F	M	L
Durability	Tube Suction Test	G	G	M	FS	G	M	H
	Sulfate Soundness	P	P	P	F	F	L	L
	Freezing and Thawing	P	P	P	FS	F	M	L
	Canadian Freeze-Thaw	G	G	M	FS	F	L	H
	Aggregate Durability Index	F	F	H	FS	F	L	M
	Unconfined Freeze Thaw	F	F	H	FS	F	M	M
Particle Geometric Properties	Shape/ Surface Texture Index	F	F	M	S	F	L	M
	Flat and Elongated Particles	P	P	L	C	P	L	L
	Percent Fractured Particles	P	P	L	C	P	L	L
	Uncompacted Void Content	P	P	L	C	P	L	L
	Digital Image Analysis	P	P	L	C	F	H	L
	Atterberg Limits	F	F	M	S	F	L	M

Rating scales in Table 4.7 are:

- Performance Predictability: G = good, F = fair, P = poor
- Accuracy: G = good, F = fair, P = poor
- Practicality: H = high, M = medium, L = low, F = fair, P = poor

- Complexity Levels: S = simple, FS = fairly simple, C = complex, VC = very complex
- Precision: G = good, F = fair, P = poor, L = low
- Cost: H = high, M = medium, L = low
- Composite: H = high, M = medium, L = low (based on relative ratings of other factors)

The selected test parameters and their proposed ranges which relate to performance have shown in Table 4.8. Different selection based on any property can be made. In this research, traffic levels and its combination with climate condition are according to Tables 4.2 and 4.3, respectively.

Table 4.8 Recommended tests and test parameters for levels of intended use in recycled aggregates (After NCHRP Report 598, 2008)

TESTS												
Traffic	**H**		**M**		**H**		**L**	**M**		**L**		
Moisture	**H**	**L**	**H**	**L**	**H**	**L**	**H**	**H**	**L**	**L**	**H**	**L**
Temperature	**F**	**F**	**F**	**F**	**NF**	**NF**	**F**	**NF**	**NF**	**F**	**NF**	
Toughness/Abrasion Micro-Deval, MD	< 5 percent			< 15 percent				< 30 percent				< 45 percent
Frost Susceptibility Tube Suction Test (dielectric constant)	≤ 7			≤ 10				≤ 15				≤20
Shear Strength *Static Triaxial Test* Max σ_d at OMC, σ_C=5 psi Max σ_d at Sat,σ_C=15 psi	≥ 100psi ≥ 180psi			≥ 60psi ≥ 135psi				≥ 25psi ≥ 60psi				Not Required
Reapeted Load Test Max σ_d at OMC, σ_C=15 psi Max σ_d at OMC, σ_C=15 psi	≥ 180 ≥ 180			≥ 160 ≥ 160				≥ 90psi ≥ 60psi				
Stiffness Test Resilient Modulus, M_R	≥ 60 ksi			≥ 40 ksi				≥ 25 ksi				

4.7 Feedstock Source of Recycled Aggregates

The materials used as recycled aggregates are products of site operations, construction processes, demolition and excavation from the local area (Figure 4.9).

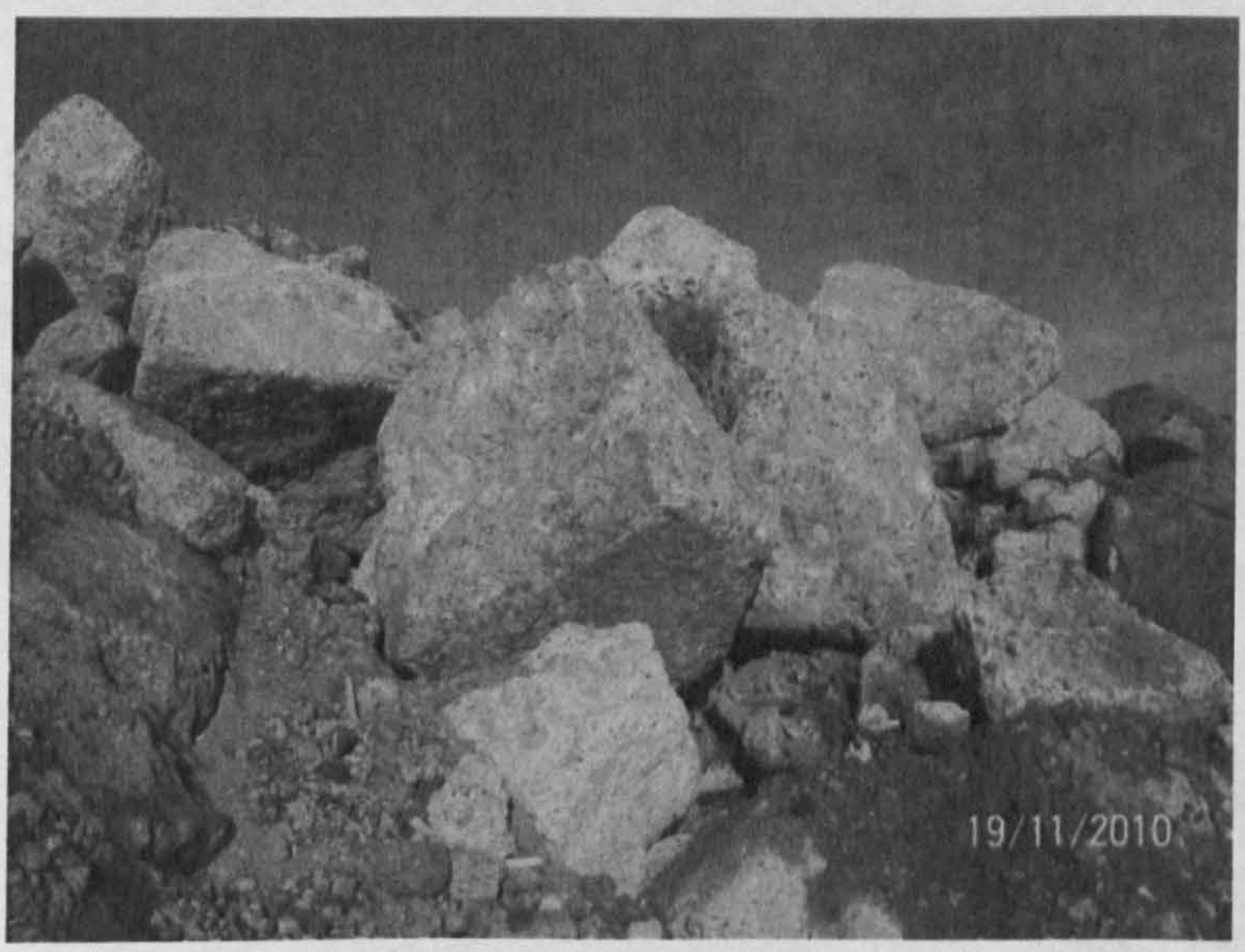


Figure 4.9 A crushed concrete slab at a typical feedstock site (photograph courtesy of Day Group Ltd)

According to Day Group website recycled aggregates are considered as 'waste materials' unless they have been produced by a recycler whose procedures comply with the WRAP Quality Protocol. Such materials may be used as waste in construction if the receiving site holds an appropriate exemption from Environmental Permitting Regulations. In the lack of either such an exemption or a suitable supplier the Environment Agency (EA) prosecutes both producing and receiving companies for the illegal disposal of waste (Day Group Ltd, 2011). The supplier of the materials for this research was Day Group Ltd. Figure 4.10 shows a copy of a completed checklist for Day Aggregates' production facilities.

Checklist and Summary Guidance[i]	YES	NO
Waste management requirements (QP ref* 3.4.1, 3.4.4, 3.6.1 and 3.7.1) Does your recycling operation have the required environmental permit/waste management licensing/exemptions and is the Duty of Care applied? *NOTE: You must demonstrate that you meet the statutory and regulatory requirements, including use of registered waste carriers and Waste Transfer Notes (WTNs). Please consult the Guidance Notes for further details.*	☑	☐
Acceptance of incoming waste (QP ref 3.4.1 to 3.4.4 and App C)		
Do you have site/location specific Acceptance Criteria procedures for the incoming waste?	☑	☐
Do your Acceptance Criteria include a description of the types of waste accepted and a description of the method of acceptance? *NOTE: List Of Waste Regulations/ European Waste Code for consistency with the WTNs must be used. You must demonstrate that only inert waste is accepted for production of aggregates to the Quality Protocol. Inspection at receipt and at tipping must be carried out.*	☑	☐
Are material input records kept? *NOTE: A record of each load received and accepted must be kept.*	☑	☐
Do you have a procedure for non-compliant waste? *NOTE: You must demonstrate how you are dealing with non-conforming incoming waste. Please consult the Guidance Notes for further details.*	☑	☐
Production and Standards/Specifications requirements (QP ref 3.1 to 3.3 and 3.5)		
Have you set up a Factory Production Control (FPC) system, which includes a Method Statement of Production (MSP), describing the waste recovery process and the range of products? *NOTE: FPC is mandatory for production of aggregates to BS EN Standards and common industry specifications and it is a requirement of the Quality Protocol. The MSP may be represented by a flow chart. All materials produced must be listed. Implementation of the FPC must be demonstrated using the detailed list of requirements within the guidance notes.*	☑	☐
Do you produce to established specifications and/or standards? *NOTE: Aggregates must be produced to be fully compliant to established specifications and/or standards.*	☑	☐

Figure 4.10 checklists for Day Aggregates (Day Group Ltd, 2011)

4.8 Facility and Processing Methodology

The main processes and material flows are illustrated in Figure 4.11. Input (1) in this flow chart shows an example of required stages of recycling the concrete and asphalt arisings from roads and footways. Input (2) shows an example of required stages of recycling the gully wastes arisings from drainage networks.

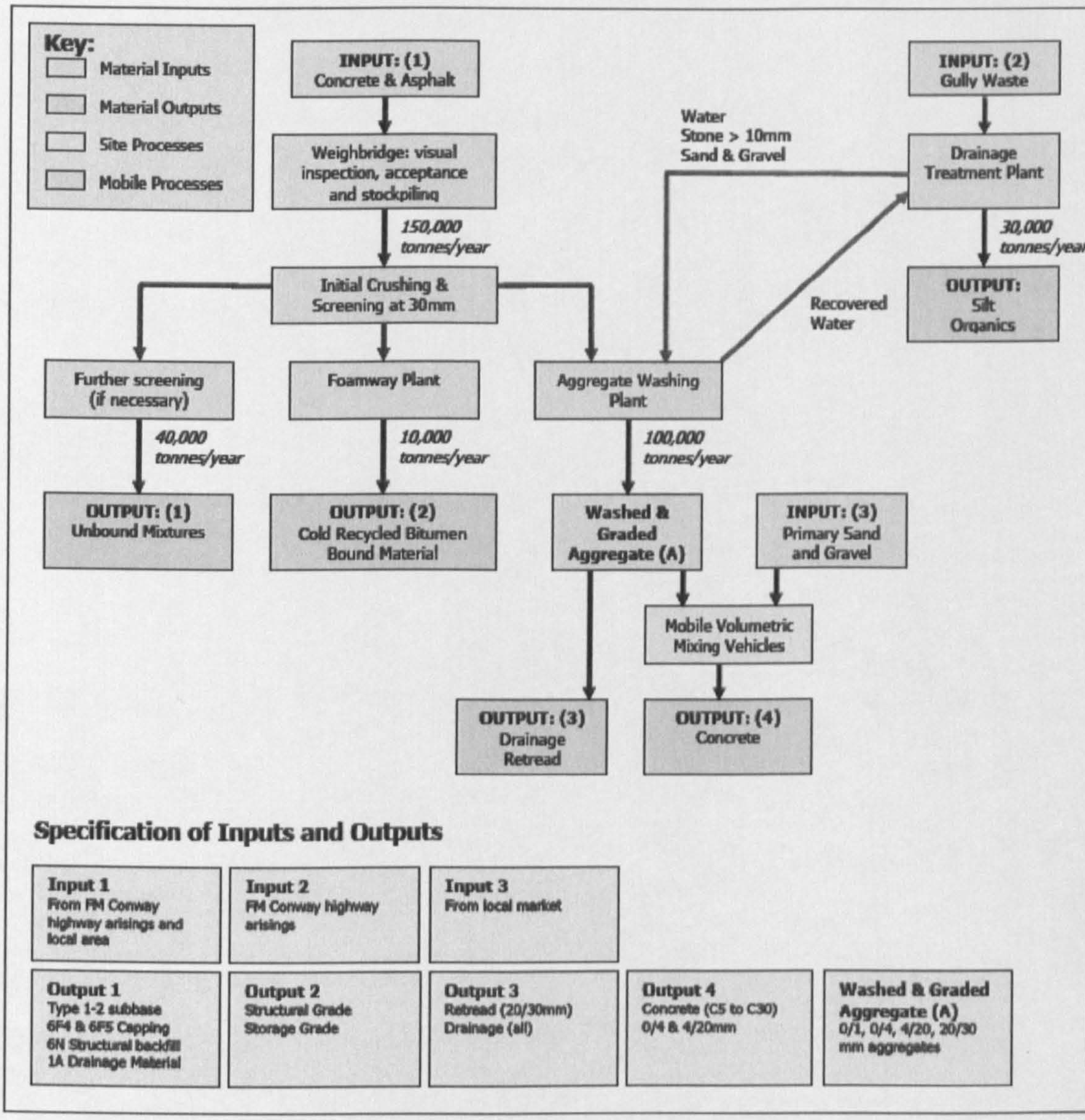


Figure 4.11 Example of recycling flows (WRAP, 2004)

4.9 Aggregate Reprocessing

Reprocessing of aggregates from feedstock to useful material as product according to objectives of this research includes reprocessing of concrete, bricks and masonry, asphalt as well as unbound granular material into a product such as Type-1 unbound subbase aggregates. The processes involved are discussed in the following sections:

4.9.1 Concrete into subbase material

The simple process reduces the size of the concrete feedstock using a crushing attachment on an excavator and a crusher, and then removes material which is still too large by passing the feedstock through a screening process. The complex process has some equipment to pre-screen the concrete and to decrease the size of the feedstock before introducing it to the crusher. It also includes an additional stage to crush any feedstock retained on the screens and return it into the recycling process, in order to minimize the amount of oversize by-products from the process (Figures 4.12 and 4.13) (WRAP website, 2011).

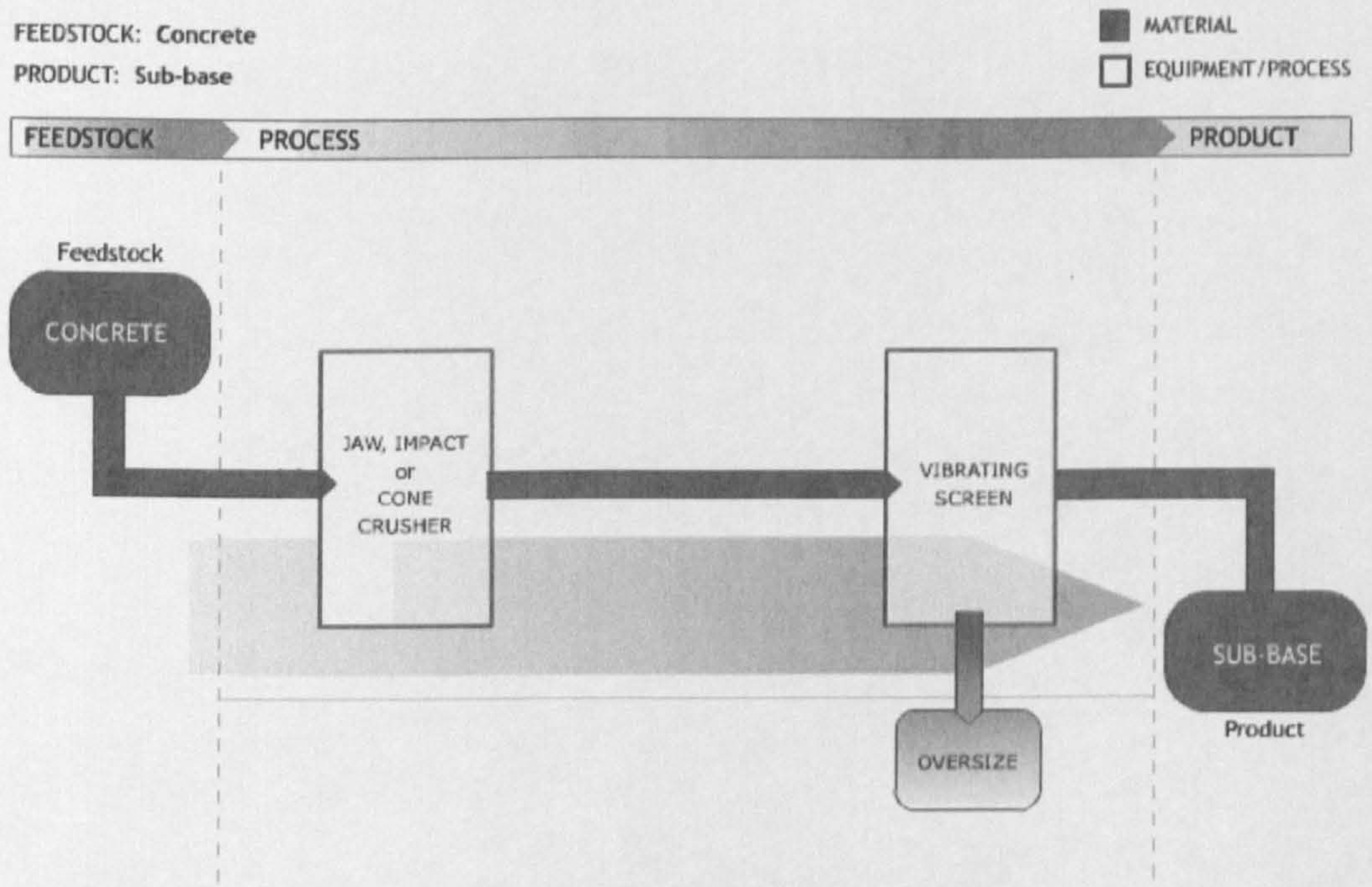


Figure 4.12 Simple process diagram of concrete into subbase (WRAP website, 2011)

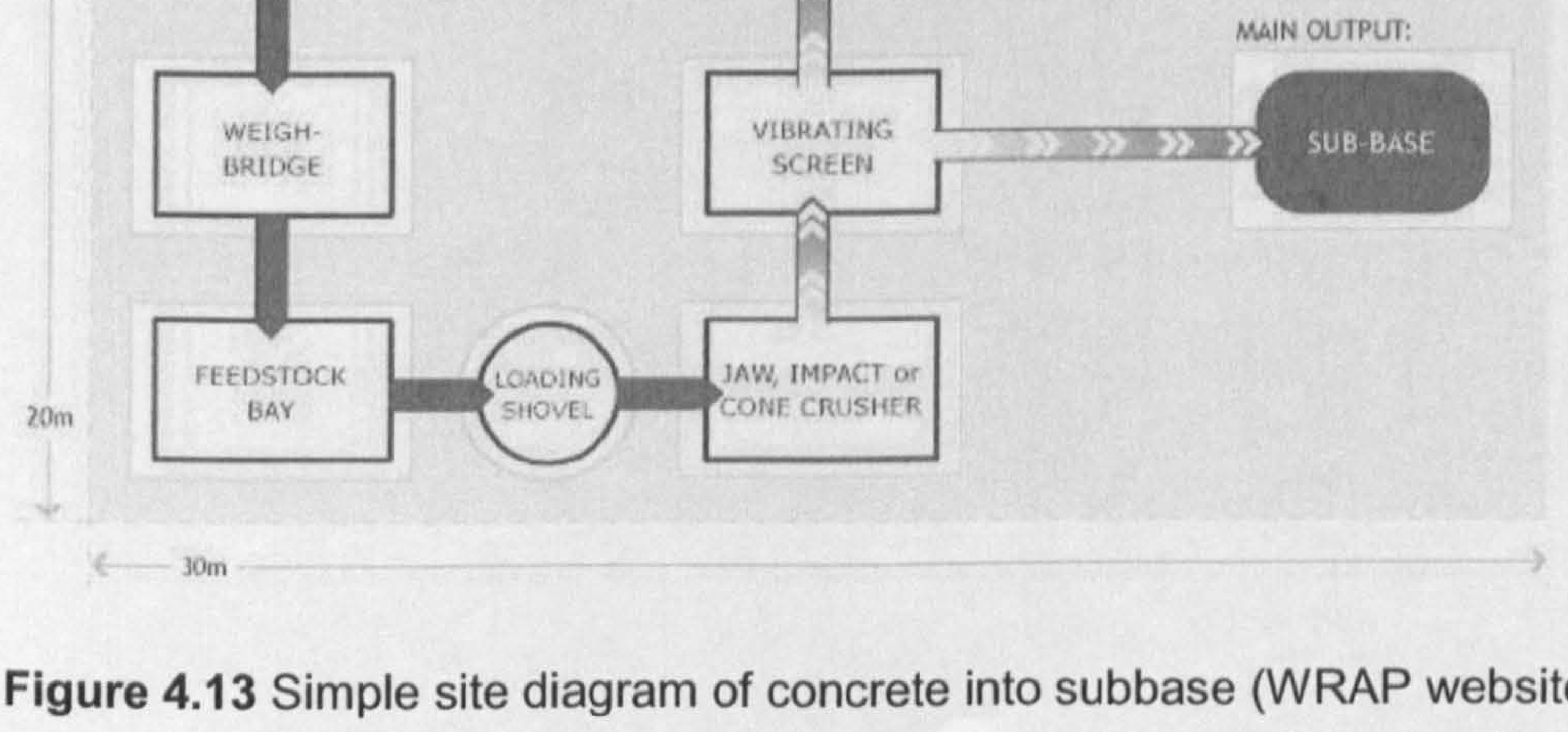


Figure 4.13 Simple site diagram of concrete into subbase (WRAP website, 2011)

Amongst the equipment required in these processes are (WRAP website, 2011): (i) vibrating screens, (ii) cone, impact and jaw crushers, (iii) belt conveyors, (iv) mobile bulk handling equipment such as loading shovels, excavator and ancillary equipment. Figure 4.14 shows a view of Day Group plant including different equipment for aggregate reprocessing.

Figure 4.14 Day Group plant (photograph courtesy of Day Group Ltd).

4.9.2 Bricks and masonry into subbase material

The simple process shown in Figures 4.15 and 4.16 reduces the size of the bricks and masonry feedstock using a crushing attachment on an excavator, and a crusher, and then removes material which is still too large by passing the feedstock through a screening process. The simple process is suitable for small feedstock, for example, bricks rejected by a manufacturer of construction materials. Larger sections of brick and masonry from, for example, the demolition of buildings, may need a more complex processing system. The complex process includes equipment to prescreen the bricks and masonry, to reduce the size of the feedstock before introducing to the crusher. It also has some more stages to crush any feedstock retained on the screens and return it to the recycling process, in order to keep quantity of the oversize by-products to a minimum (WRAP website, 2011).

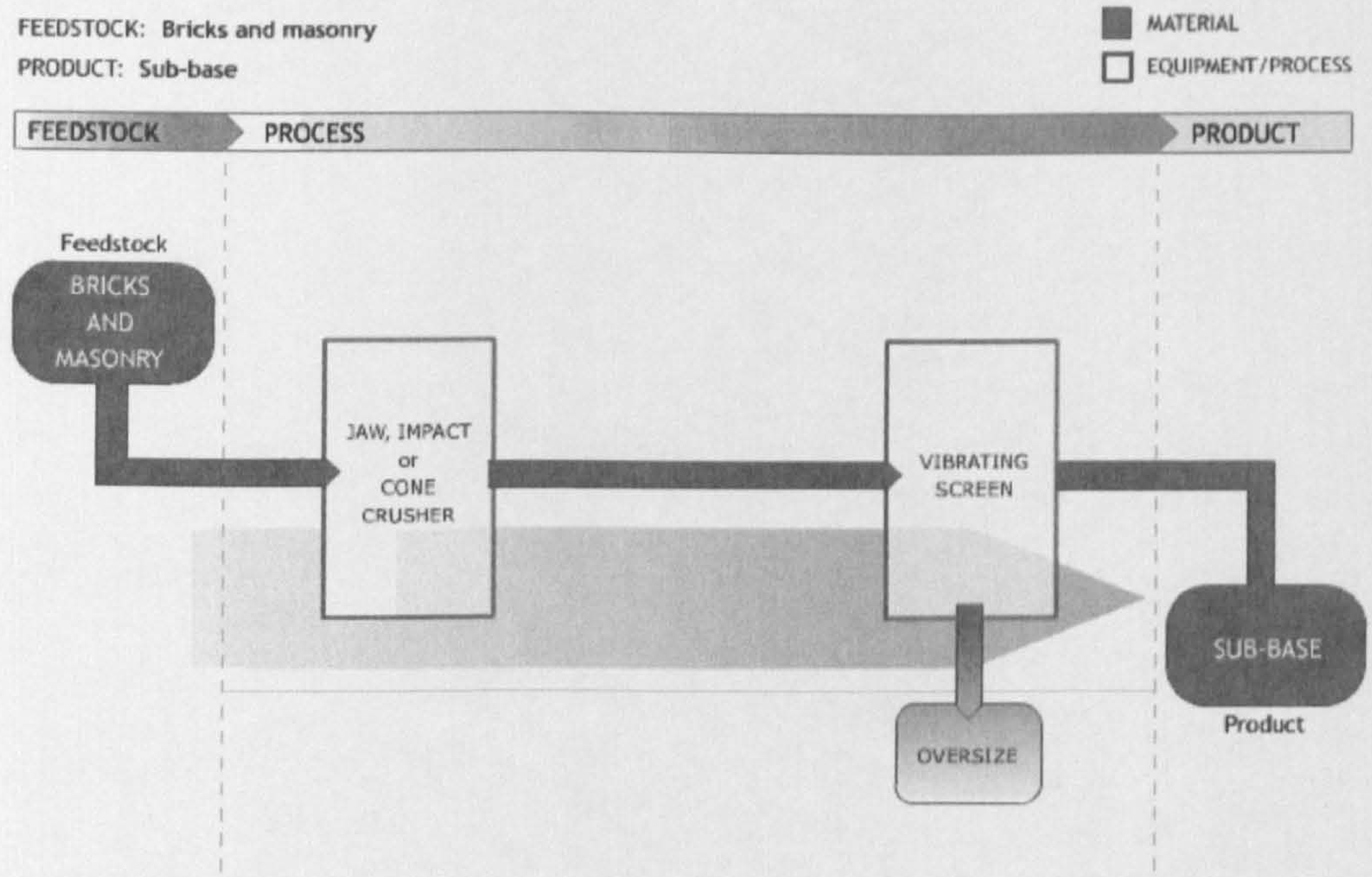


Figure 4.15 Simple process diagram of bricks and masonry into subbase (WRAP website, 2011)

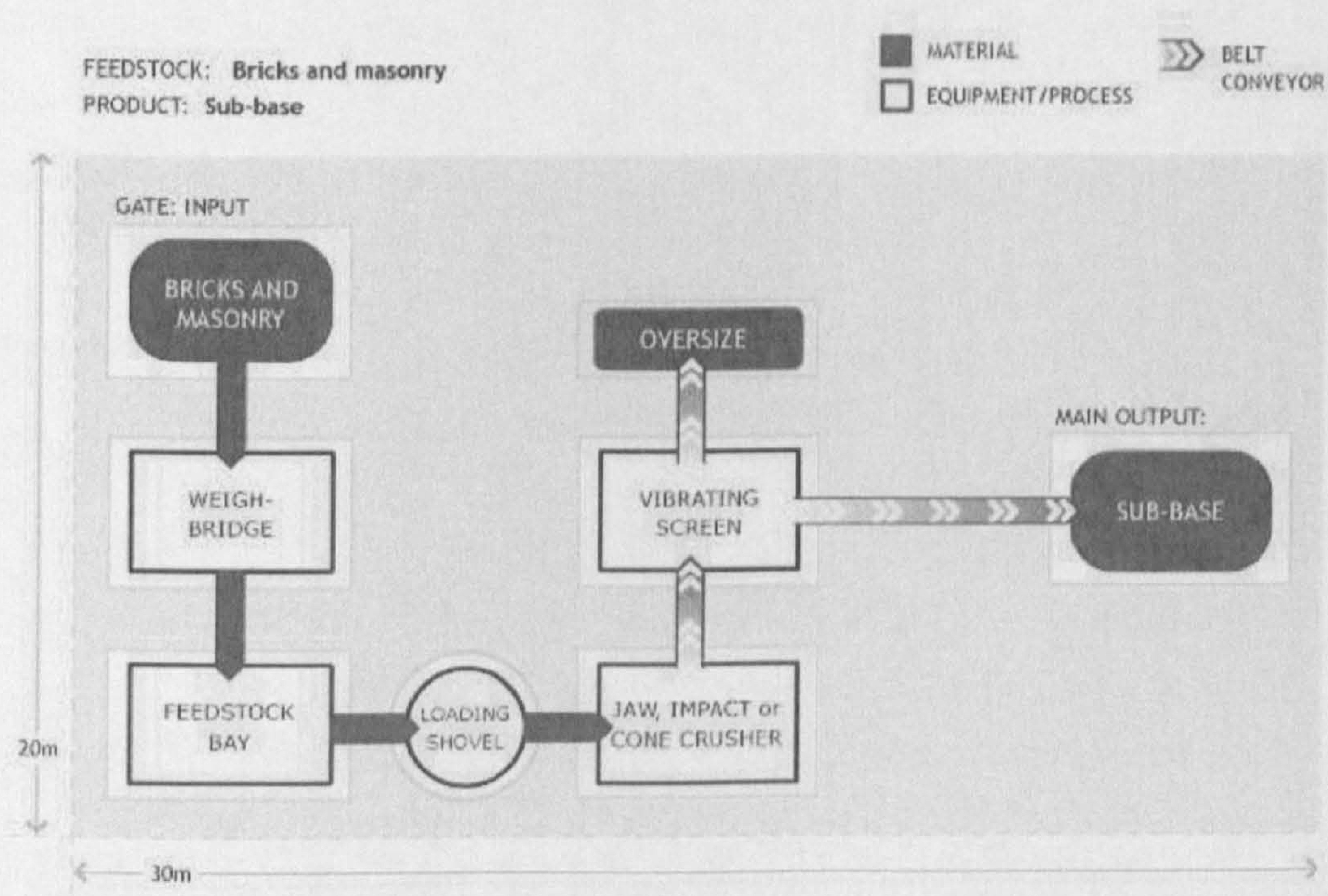


Figure 4.16 Simple site diagram of bricks and masonry into subbase (WRAP website, 2011)

Equipment of reprocessing of concrete to subbase mentioned in 4.9.1 can be used in this case as well (Figure 4.17).

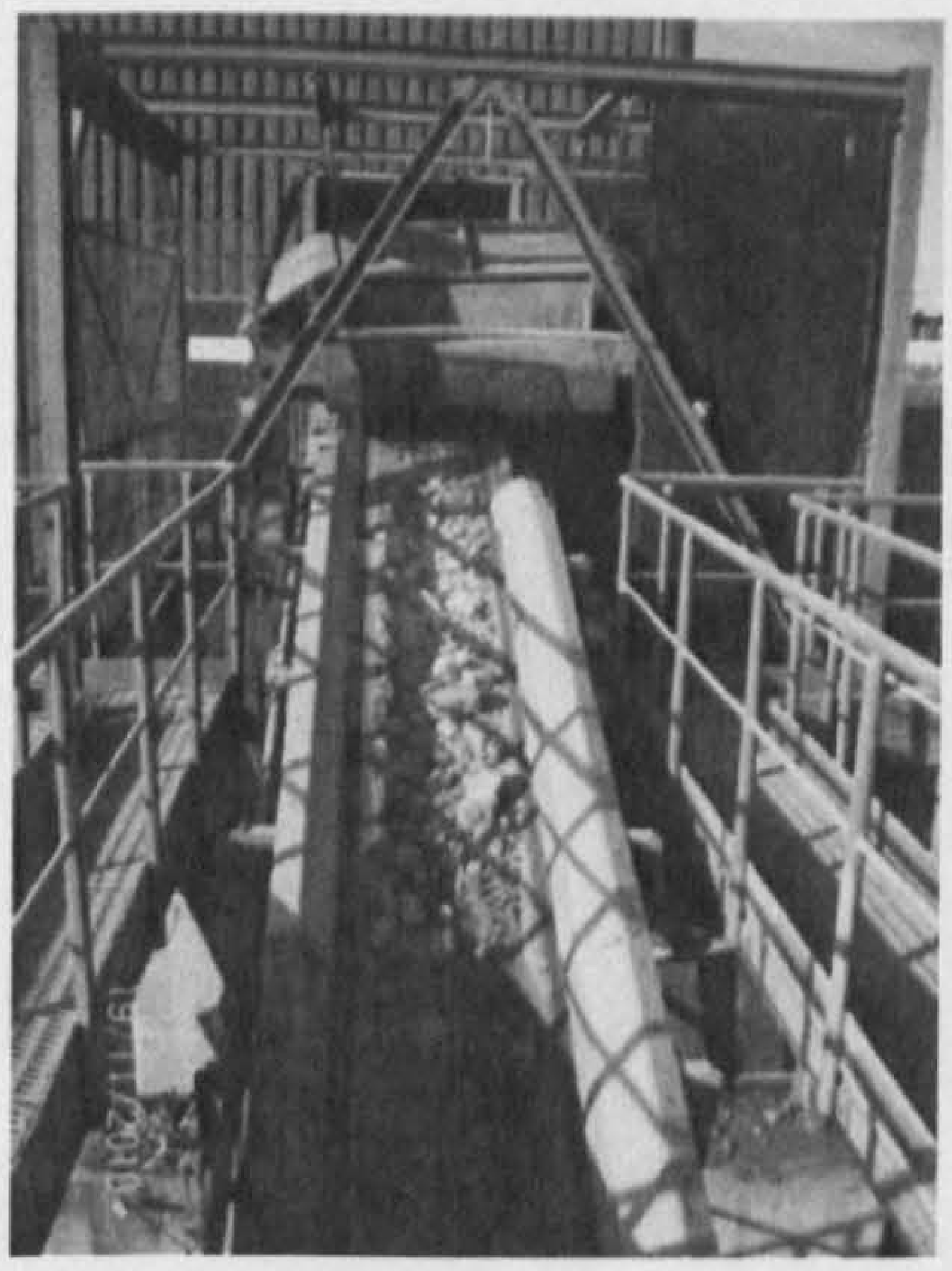


Figure 4.17 A belt conveyor in Day Group plant conveys the aggregate into screens (photograph courtesy of Day Group Ltd)

4.9.3 Planed asphalt into subbase material

The simple process reduces the size of the asphalt feedstock by applying a crushing attachment on an excavator, and a crusher, and then removes material which is still too large by passing the feedstock through a screening process (Figure 4.18). The feedstock, if produced from road planings, may be sufficiently small to not require further crushing.

Figure 4.18 Final RAP products for Type 1 subbase (photograph courtesy of Day Group Ltd)

The complex process includes equipment to pre-screen the asphalt and to decreasing the size of the feedstock before introduction to the crusher. It also has an additional stage to crush any feedstock retained on the screens and return it to the recycling process, because the oversize by-products from the process should be minimise (Figures 4.19 and 4.20). Equipments of reprocessing of concrete to subbase mentioned in 4.9.1 can be used in this case as well (WRAP website, 2011).

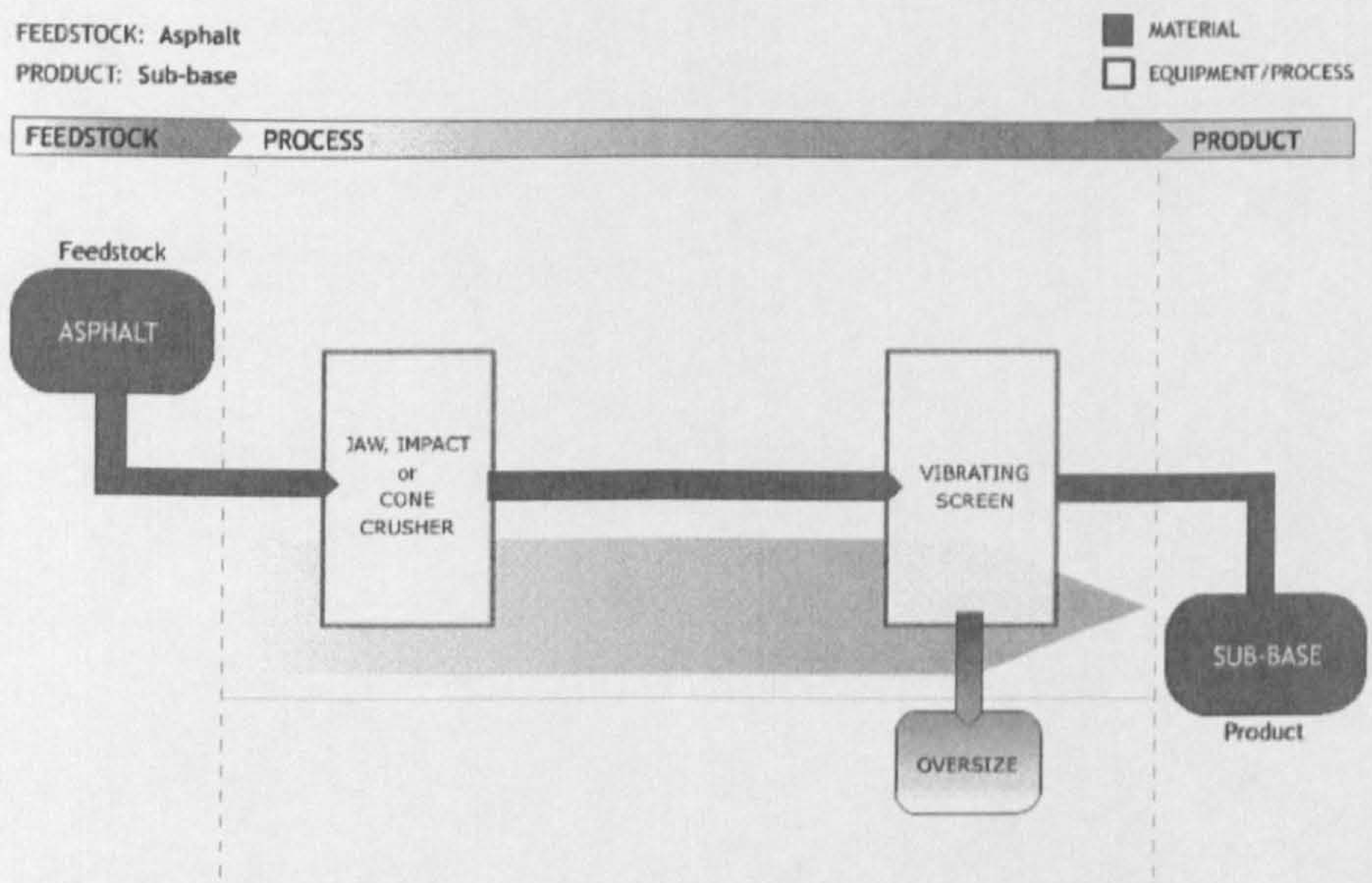


Figure 4.19 Simple process diagram of asphalt into subbase (WRAP website, 2011)

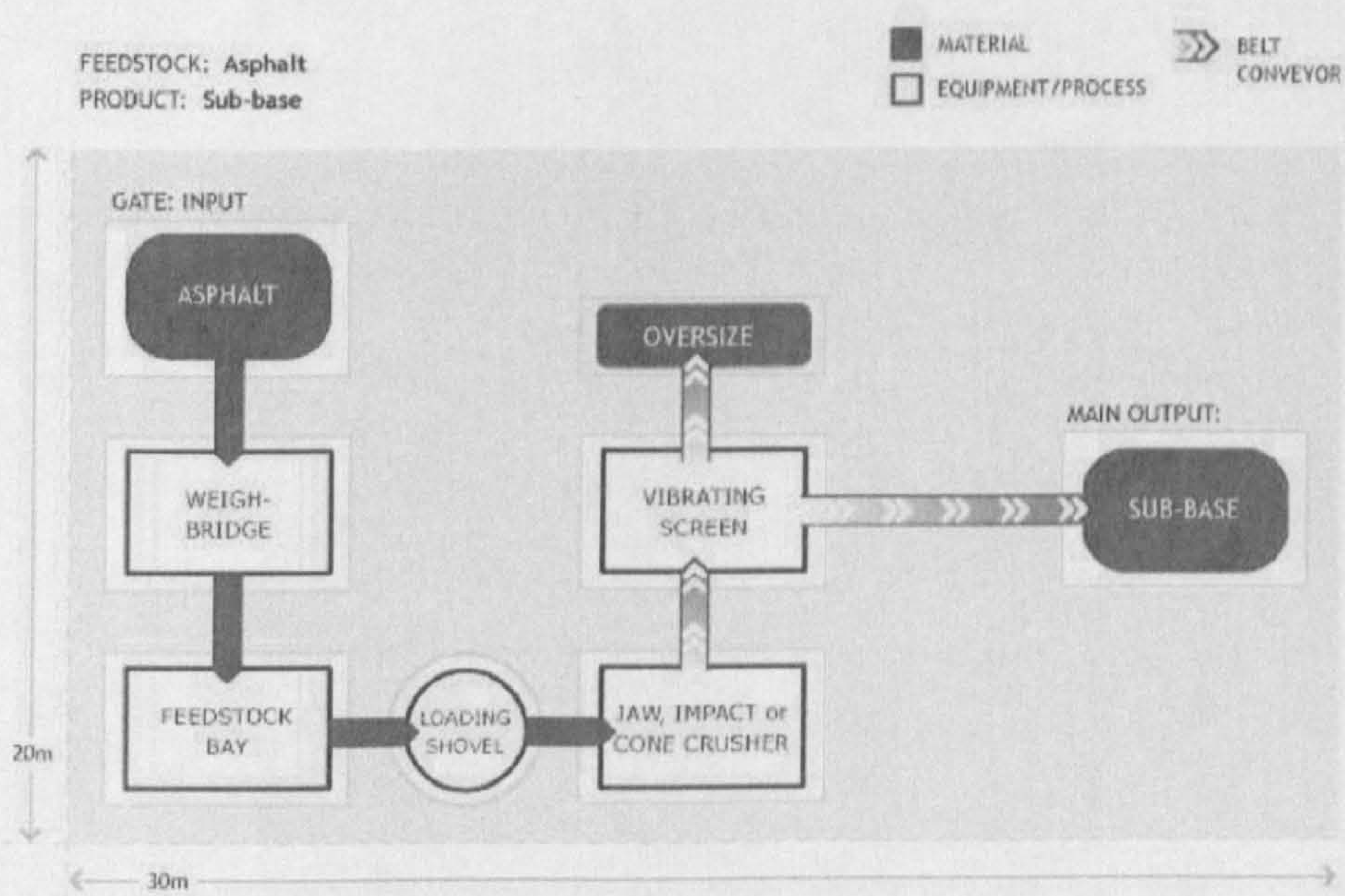


Figure 4.20 Simple site diagram of asphalt into subbase (WRAP website, 2011)

4.9.4 Unbound granular material into subbase material

This procedure consists of two processes: a simple and a complex process. In the simple process, the size of the feedstock is reduced by use of an excavator attachment and crusher, followed by screening of the feedstock to produce the final subbase type material. In the complex process, a finger screen or "trammel" is used to remove small contamination, such as soils. Soils can also be removed from the feedstock through a washing process. In addition, the complex process includes a further stage whereby any feedstock retained on the screens is crushed and fed back into the recycling process. (Figures 4.21 and 4.22) Equipment for reprocessing of concrete to subbase mentioned in 4.9.1 can be used in this case as well (WRAP website, 2011).

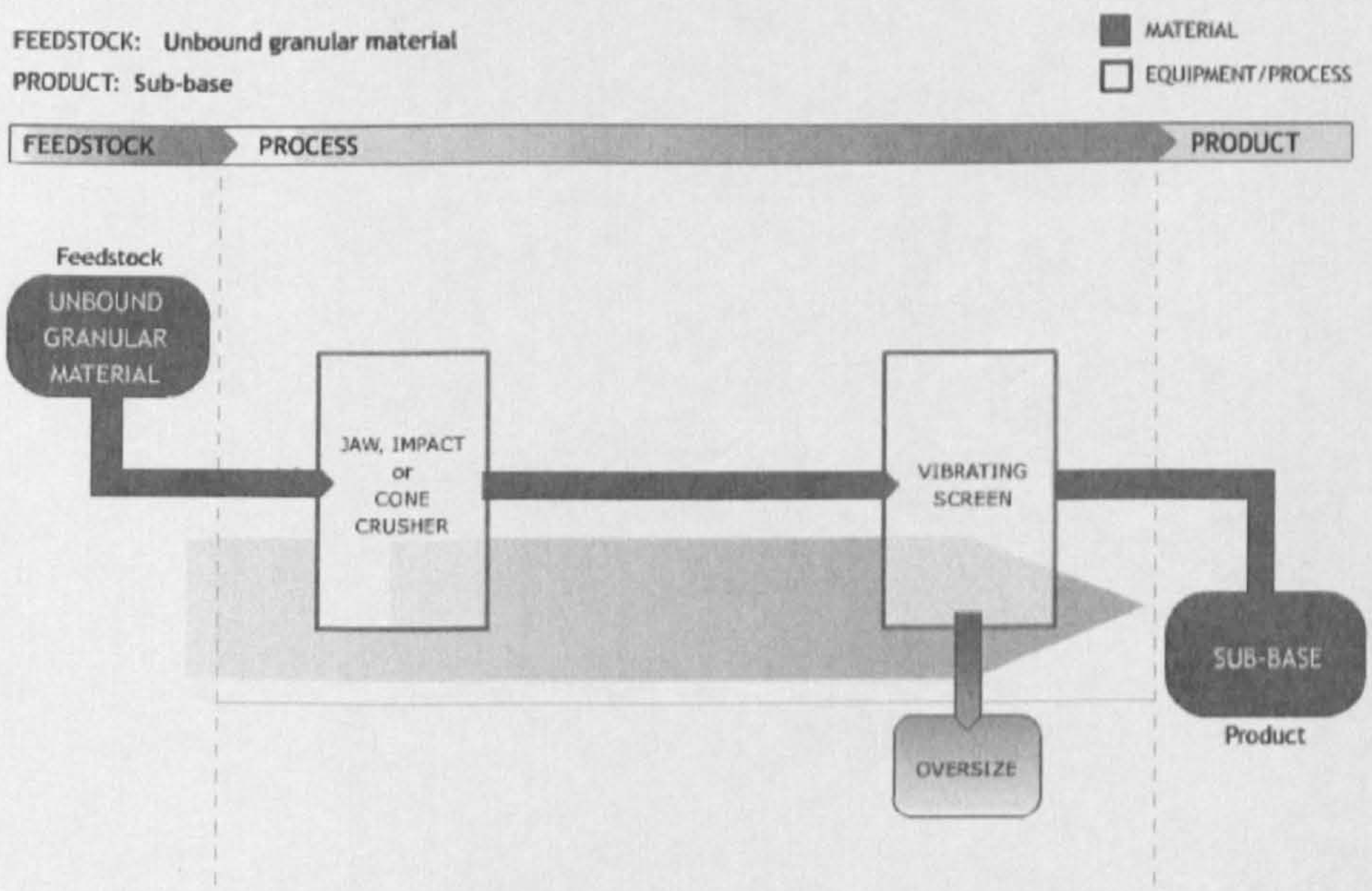


Figure 4.21 Simple process diagram of unbound granular material into subbase (WRAP website, 2011)

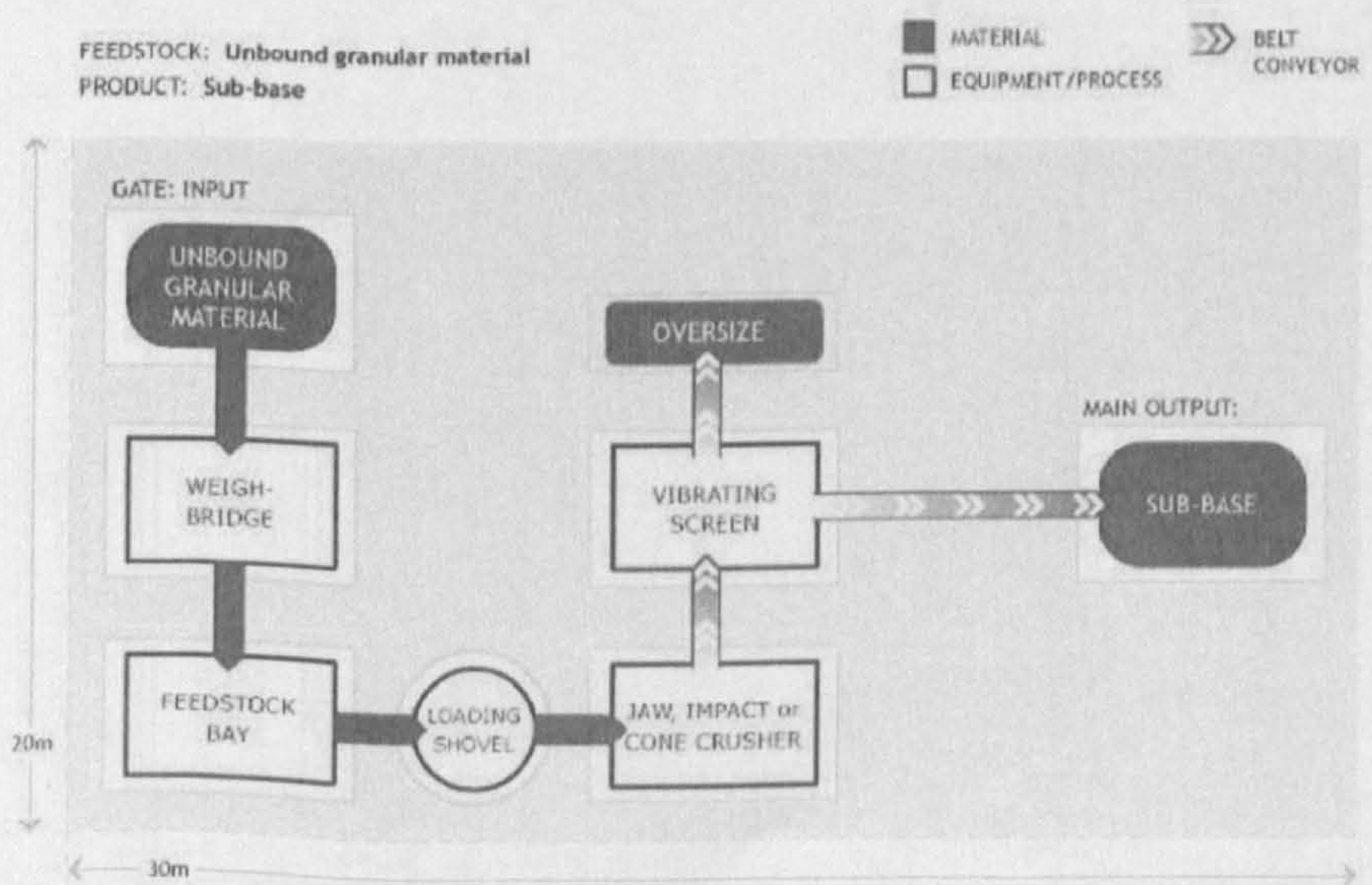


Figure 4.22 Simple site diagram of unbound granular material into subbase (WRAP website, 2011)

4.10 Selection and Description of the Recycled Materials Used in This Research

A series of tests were carried out by the author at UK and Iranian institutions over a period of nearly 2 years. The tests, which were carried out on different constituent aggregates provided a range of comparable characteristics and performance, as summarized in Table 4.9.

Table 4.9 Selected aggregates

Aggregates tested in the UK	Aggregates tested in Iran
100%NA	100%NA
	80%NA+20%RCA
	50%NA+50%RCA
100%RCA	20%NA+80%RCA
	100%RCA
50%RCA+50%RAP	80%RAP+20%RCA
	50%RAP+50%RCA
	20%RAP+80%RCA
	100%RAP

4.10.1 Selection and description of the recycled aggregates tested in the UK

The main focus of the research was on investigating the suitability of three kinds of aggregates which all are Type 1 unbound mixtures for subbases (MCHW Series 800):

- 100% Recycled Concrete Aggregate (RCA) which is indicated with the notation T-1.

These aggregates were supplied by Day Group Ltd from depots located in Brentford to where materials were brought from various resources in South East London. The commercial name of this mix is White Recycled (Figure 4.23).

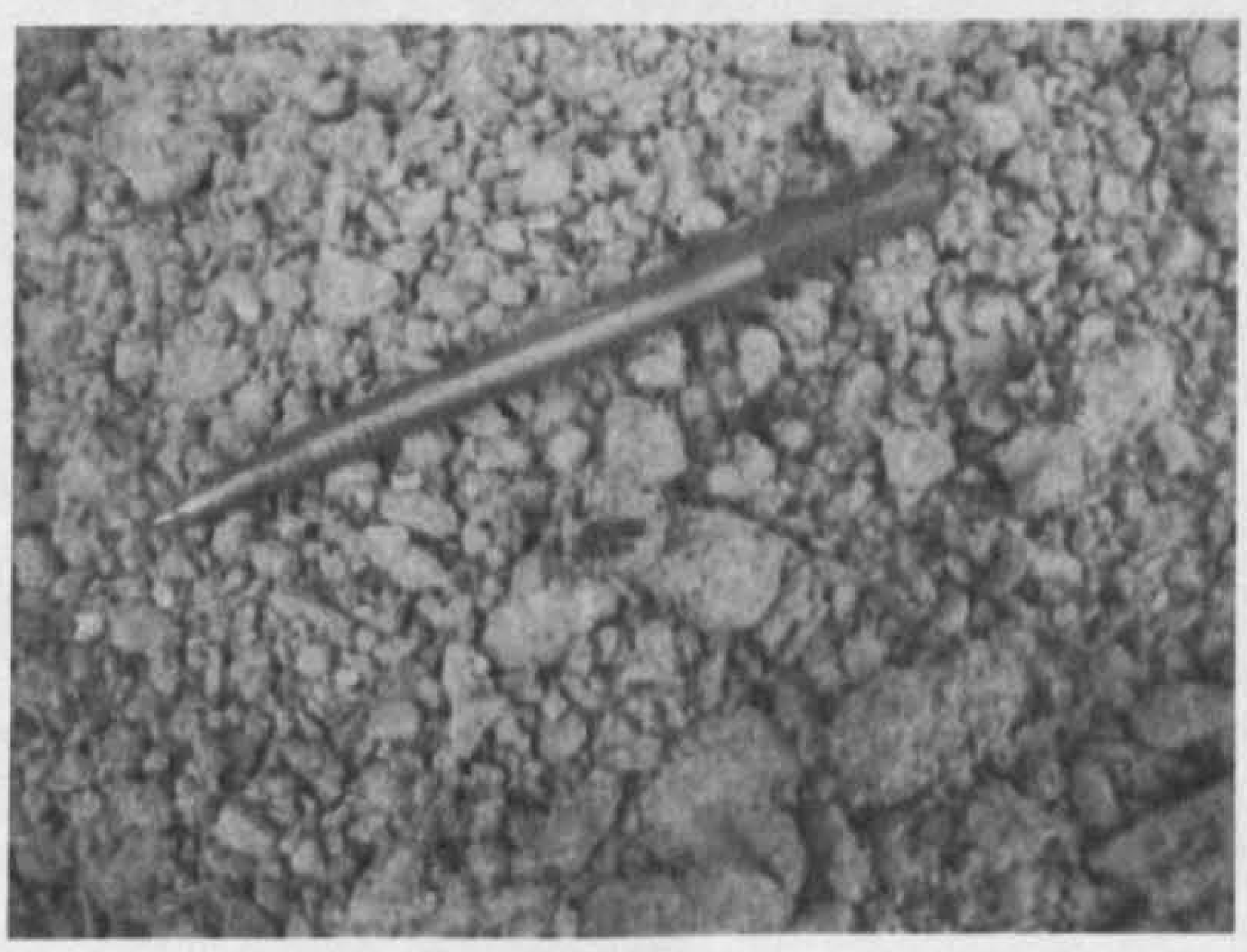


Figure 4.23 Sample of T-1 used in laboratory testing

- 100% Lime stone as the natural aggregate (NA) which is indicated with the notation T-2.

The limestone aggregates were also supplied by Day Group Ltd. from Torr Works Quarry located in Somerset (Figure 4.24).

Figure 4.24 Sample of T-2 used in laboratory testing

- 50% Recycled Concrete Aggregate (RCA) + 50% Reclaimed Asphalt Pavement (RAP) which is indicated with the notation T-3.
- These aggregates were supplied by Day Group Ltd. from their depots located in Purley which was being feed through a various resources in South East London. The commercial name of this mix is Black/White Recycled. (Figure 4.25)

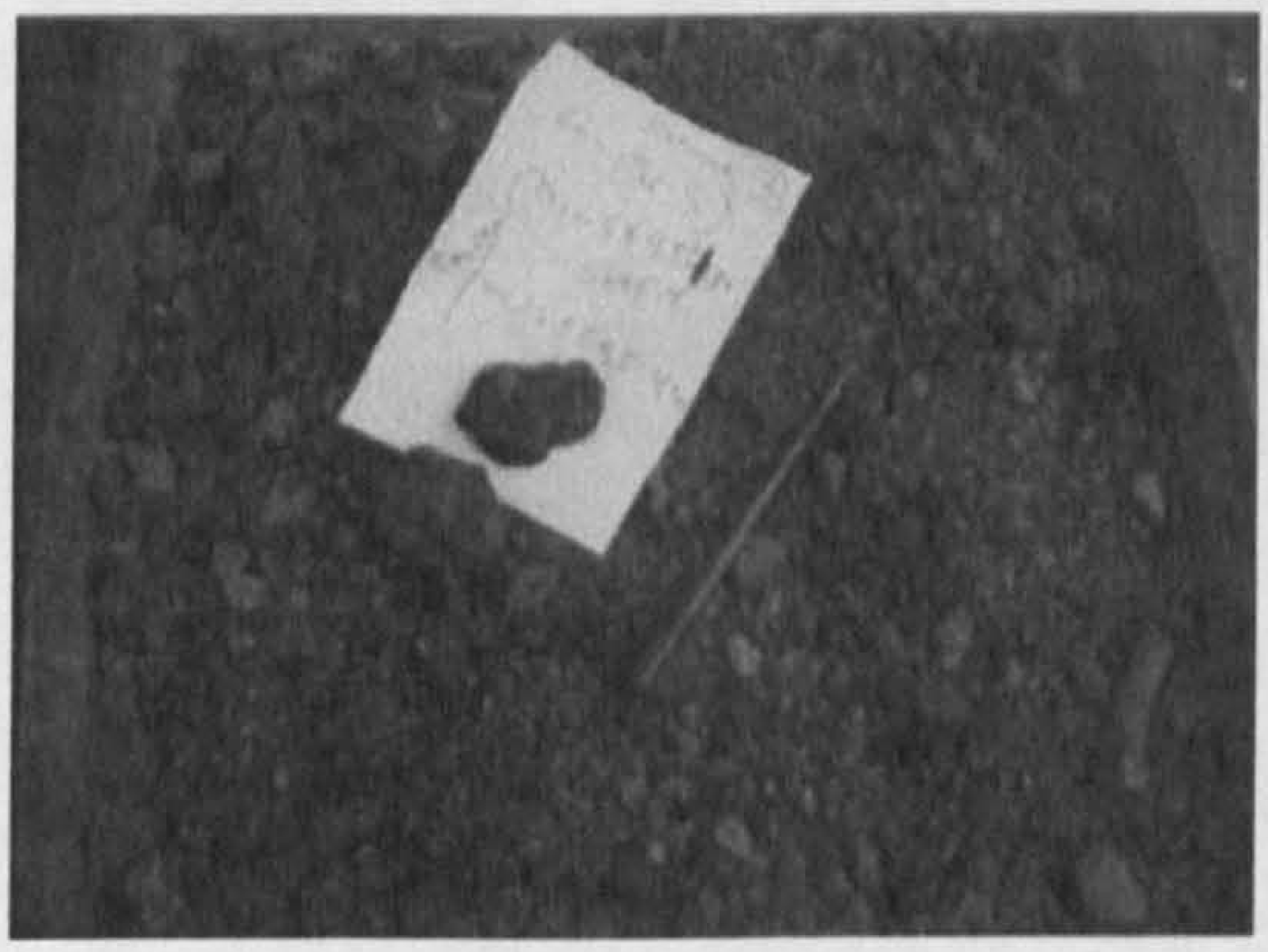


Figure 4.25 Sample of T-3 used in laboratory testing

4.10.2 Selection and description of the recycled aggregates tested in Iran

4.10.2.1 Source of RCA

Recycled concrete aggregate (RCA) was produced from demolishing of walls and piles of Amir Kabir tunnel site located in central Tehran which had been redesigned. The contract of operation of Amir Kabir Tunnel and passage way was signed between The Municipality Organization of Tehran and Nimrokh – Arsa Joint

Venture in September 2009. The project includes the construction of a series of tunnels (cut and cover & NATM) with nearly 2,200 m in length. The primary role of the project was to solve some of Tehran's traffic problems. The contract also includes the demolition of old walls, piles and ramps as well as the construction of an underground parking facility and a shopping centre. The estimated cost of the Project was 100 million USD (Figure 4.26) (Nimrokh website).

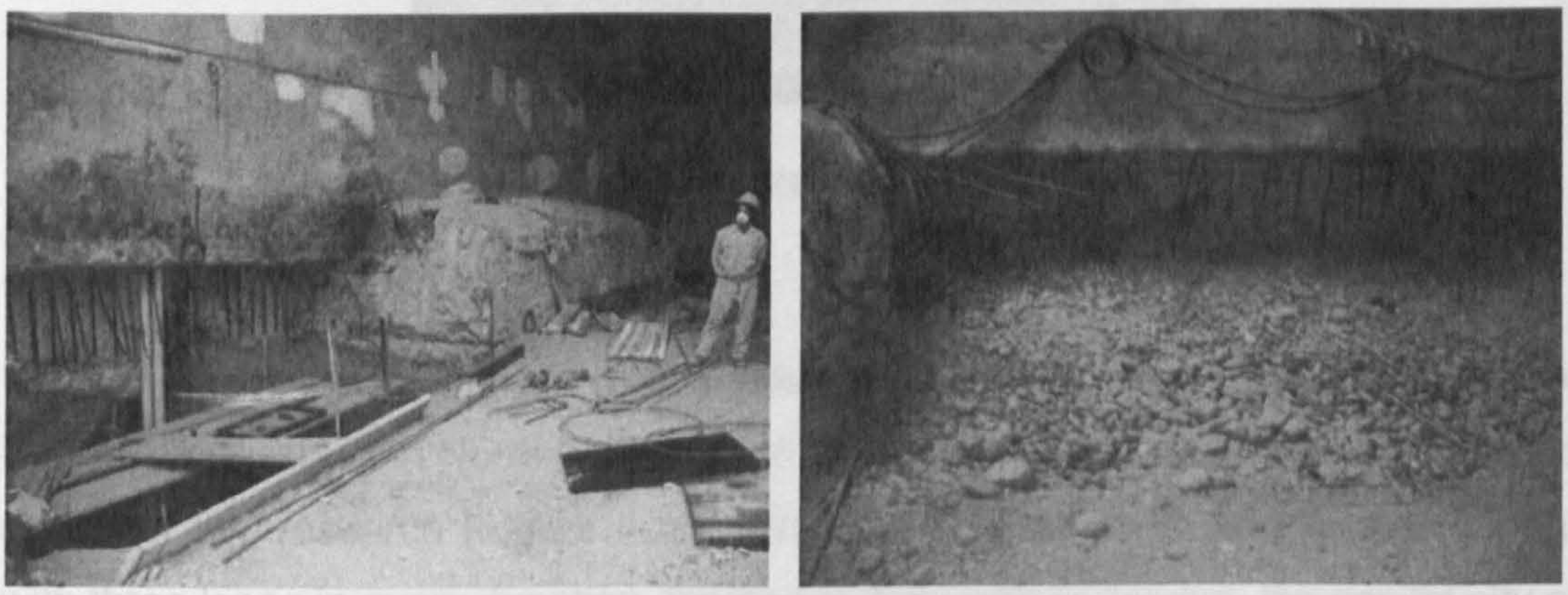


Figure 4.26 Demolished walls and piles

As most of the concrete demolished in the initial stages was oversize (Figure 4.27), it was crushed manually using pneumatic hammers to produce both coarse and fine aggregate ranges in subbase limits.

Figure 4.27 Demolished concrete

4.10.2.2 Source of NA

Natural aggregates (NA) were obtained from Kan River located in North West Tehran. These aggregates were mostly rounded corner and the oversize particles were separated manually (Figure 4.28).

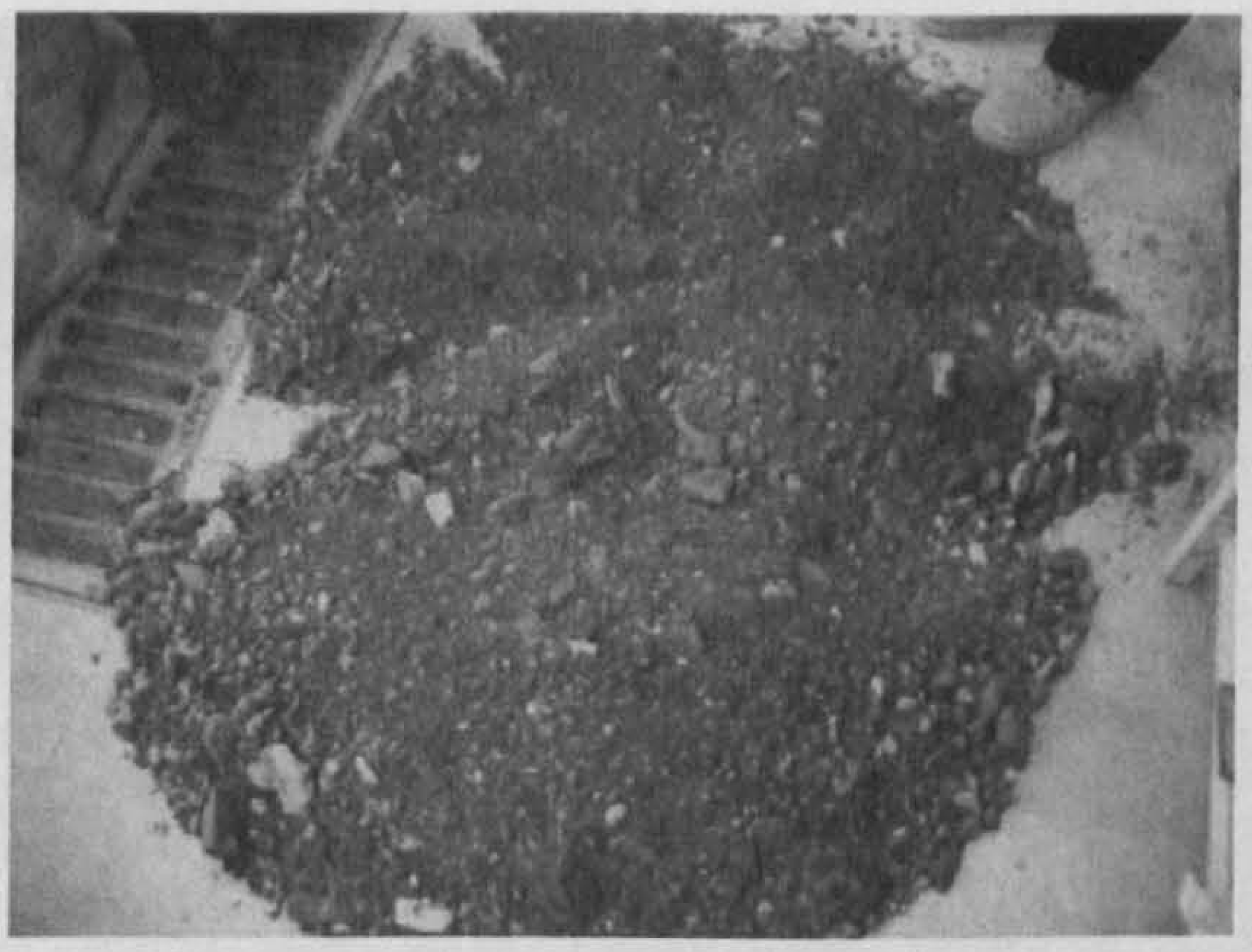


Figure 4.28 Natural river aggregates

4.10.2.3 Source of RAP

Reclaimed asphalt pavement (RAP) was obtained from milling the pavement of Babaie expressway in North East Tehran which was being rehabilitated during the period of this research (Figure 4.32). Babaei expressway is 16 kilometers long, with three lanes in every direction, located to the north of Tehran, the capital city of Iran. Investigation had showed that the pavement had deteriorated to unacceptable levels. There were longitudinal and transverse cracks in the pavement, described as "alligator cracks" (Figure 4.29). Observations showed that the lower layers of asphalt were defective and there was a large proportion of clay in these layers (Figure 4.30). Under these circumstances, it was decided to mill the whole asphalt layer and to improve the lower layers, replacing the cold recycled asphalt with cement and foamed bitumen.

Figure 4.29 Asphalt distresses

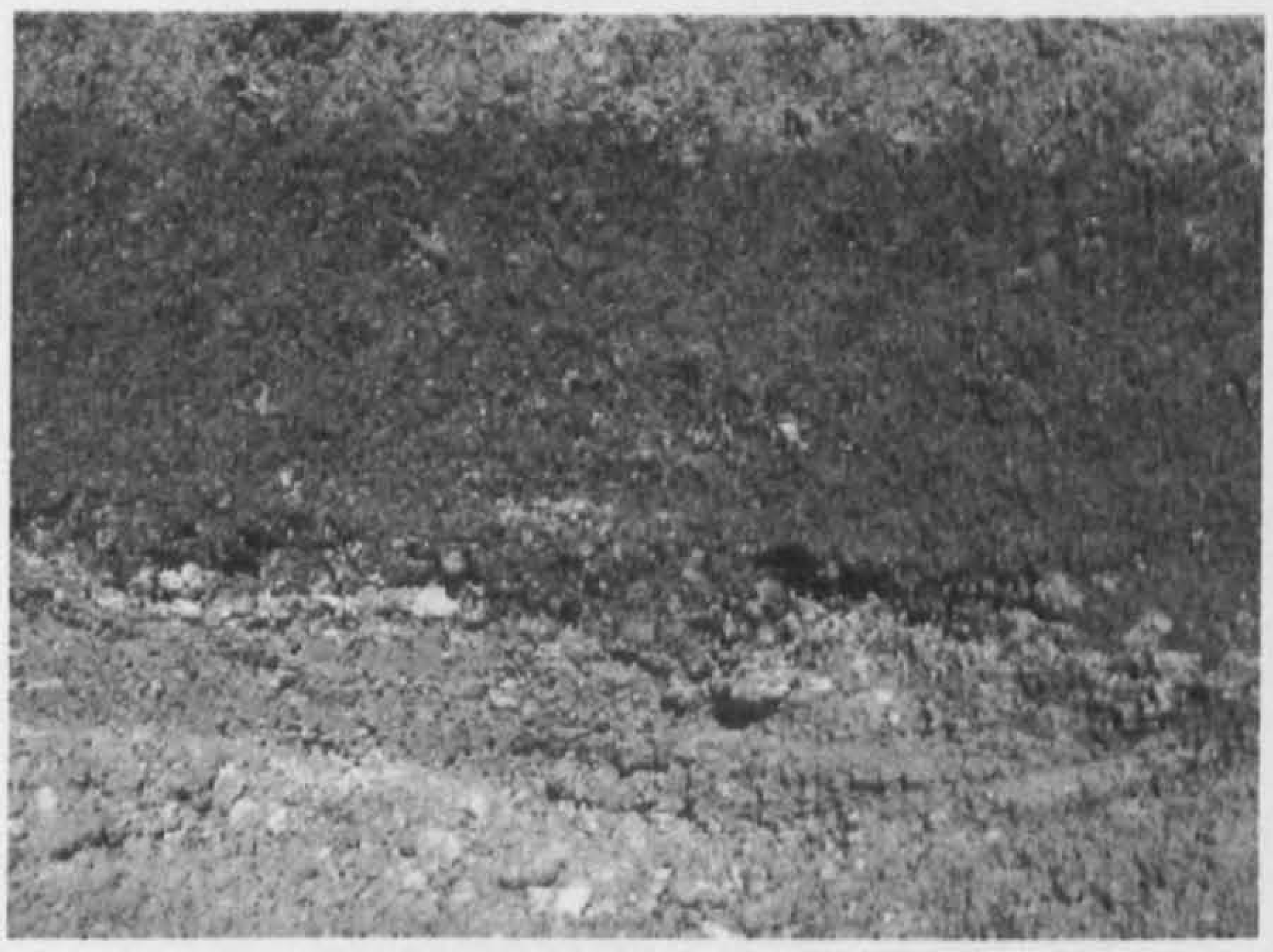


Figure 4.30 Pavement after milling

The first stage of in-plant asphalt recycling in Babaei expressway rehabilitation project involved producing the RAP in the size of subbase aggregates. This procedure is illustrated in Figure 4.31.

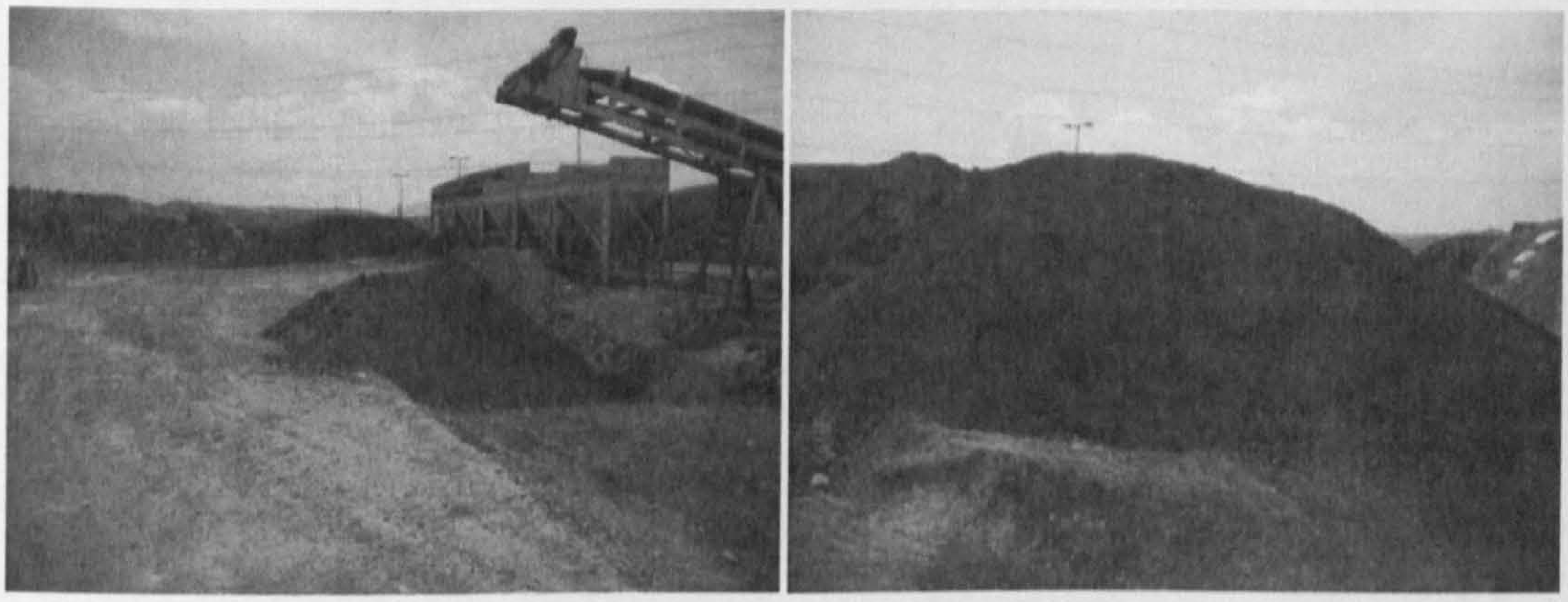


Figure 4.31 In-plant asphalt recycling produces RAP

Figure 4.32 RAP aggregates from stockpile of milled bituminous concrete

4.10.2.4 Preparing the mixes

After planning for the source of required aggregates, some samples were taken from these resources. The visual and sieving tests showed that RAP and NA after separating some oversize fractions would meet the requirements of codes and Standards for subbase material sizes. Nevertheless RCA did not comply with the subbase grading limits as shown in Figure 4.33. For this reason, the coarse particles were crushed manually to modification the grading curves and bring that inside the limits.

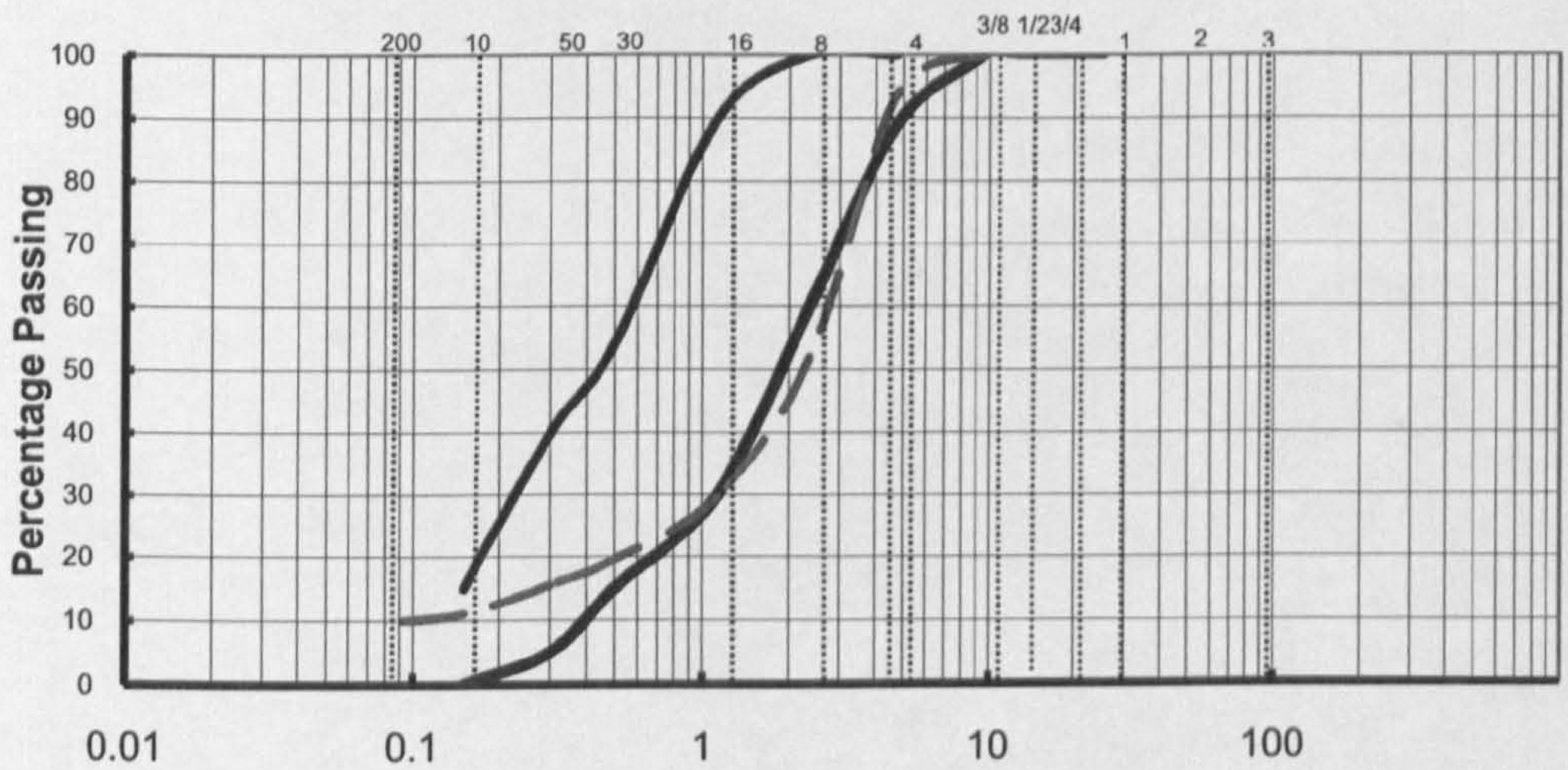


Figure 4.33 Initial grading curves of RCA

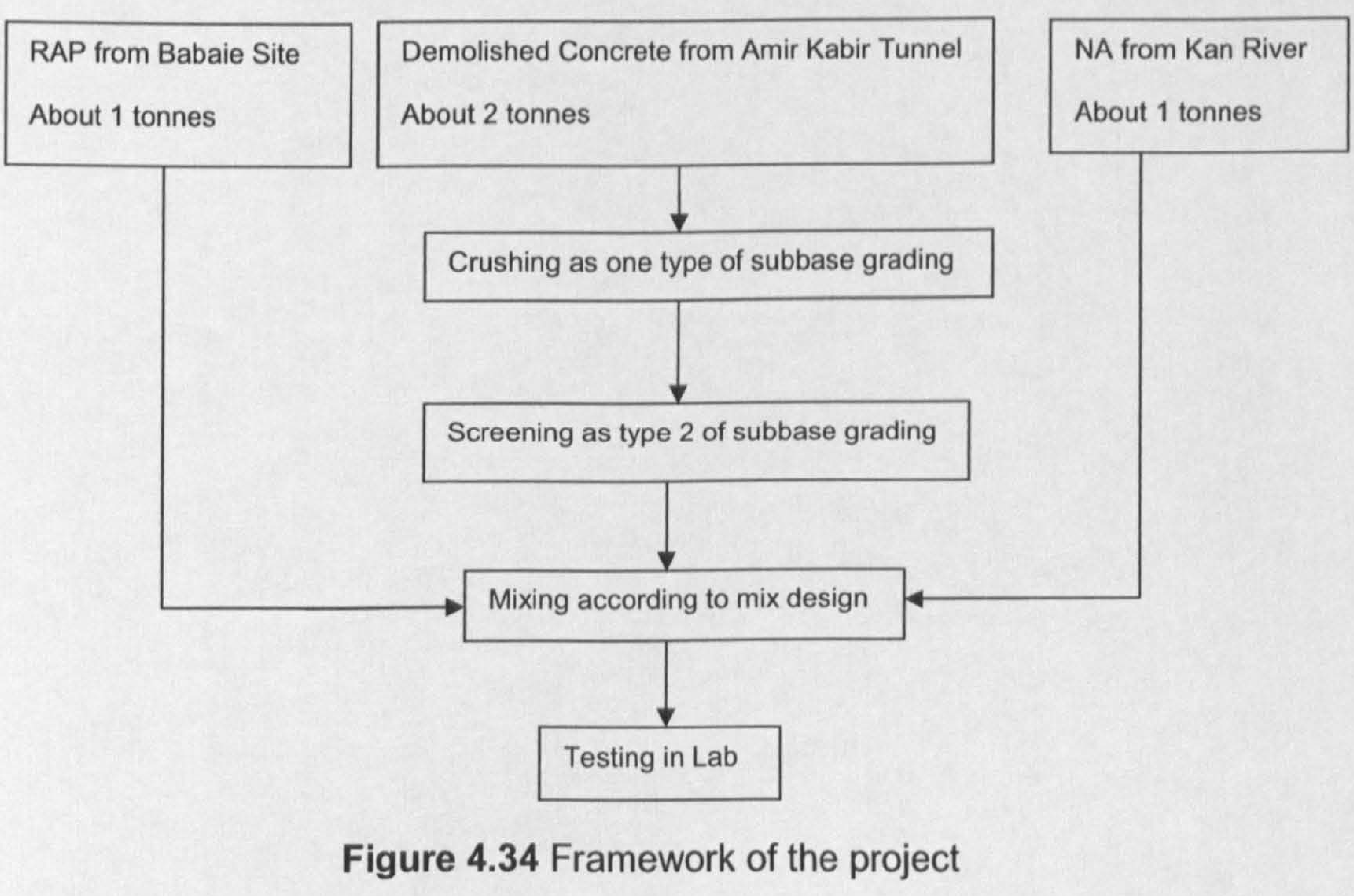


Figure 4.34 Framework of the project

The framework illustrated in Figure 4.34 shows the stages of preparing the mixes right from the locating the source to transporting to the laboratory. A part of Amir Kabir tunnel site was chosen to stockpiling the recycled materials. The site was covered, drained and cleaned to prevent contamination of the aggregates. The stored aggregates were separated with cement blocks to prevent them from accidental mixing (Figures 4.35, 4.36).

Figure 4.35 View of the covered site

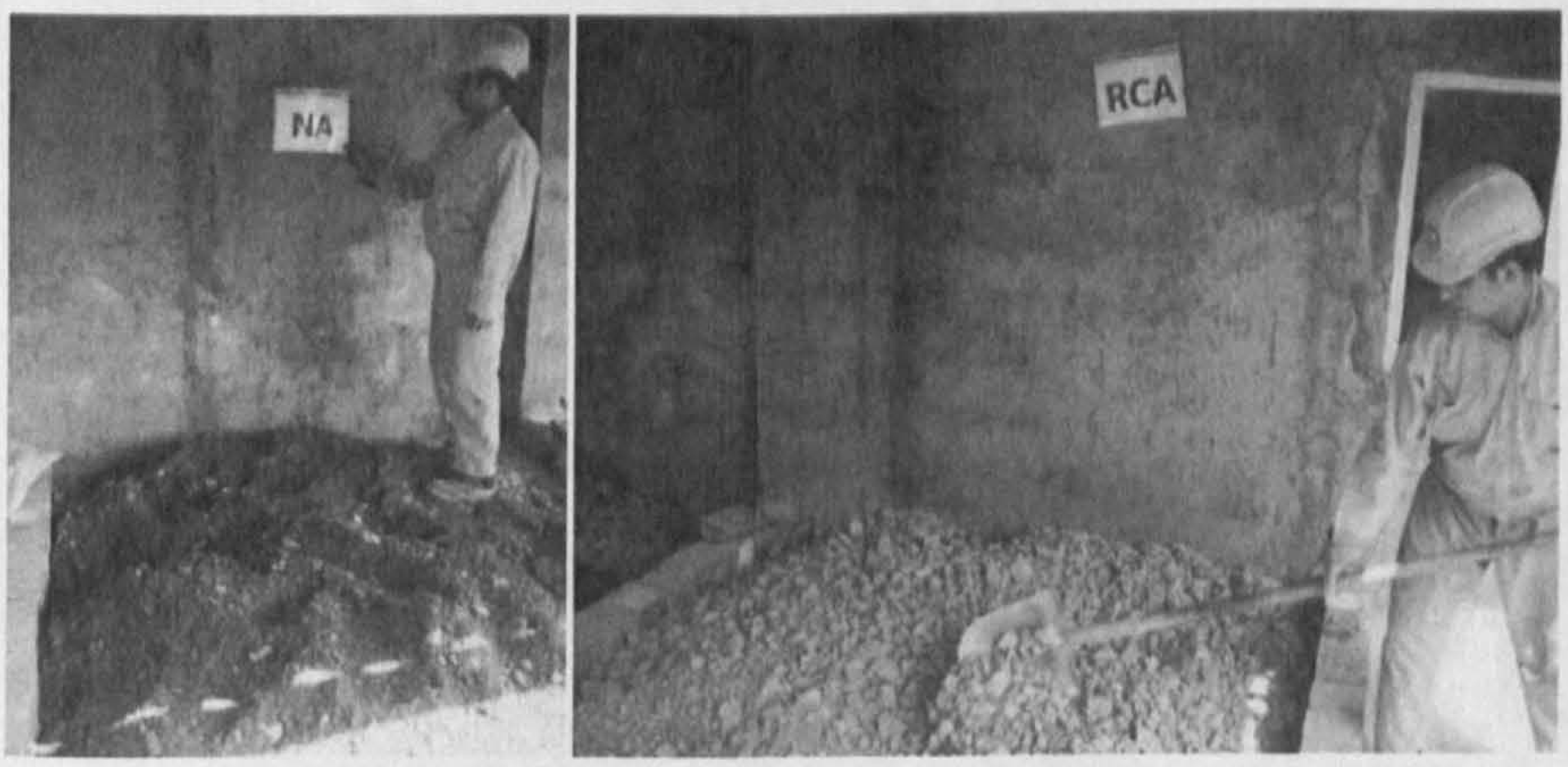


Figure 4.36 Separated aggregates

Proposed mix designs were classified in five batches according to Table 4.10. In batch 2, 3 and 4 the NA, RCA and RAP were tested individually i.e. 100%NA, 100%RCA and 100%RAP, respectively. In batch 1 a portion of NA was replaced by RCA and used as the subbase materials under the testing programme. The replacement levels were 20% and 50% by weight of the RCA, so that the mix designs applied to tests were: 100%NA as control mix, 80%NA+20%RCA, 50%NA+50%RCA, 20%NA+80%RCA and 100%RCA.

Table 4.10 Proposed mixes

BATCH	PROPOSED MIX
BATCH 1 (NA+RCA)	80%NA+20%RCA
	50%NA+50%RCA
	20%NA+80%RCA
BATCH 2 (NA)	100%NA
BATCH 3 (RCA)	100%RCA
BATCH 4 (RAP)	100%RAP
BATCH 5 (RCA+RAP)	80%RAP+20%RCA
	50%RAP+50%RCA
	20%RAP+80%RCA

Figure 4.37 shows the mix of 50%NA+ 50%RCA before and after mixing. In batch 5, a portion of RCA was replaced by RAP aggregate and used as the subbase materials. The replacement levels were 20% and 50% by weight of the RCA, so that the mix designs applied to tests are: 100% RCA, 80%RCA+20%RAP, 50%RCA+50%RAP, 20%RCA+80%RAP and 100%RAP. After preparing the different mixes, they were bagged in 50 Kg bags and delivered to the laboratory (Figure 4.38).

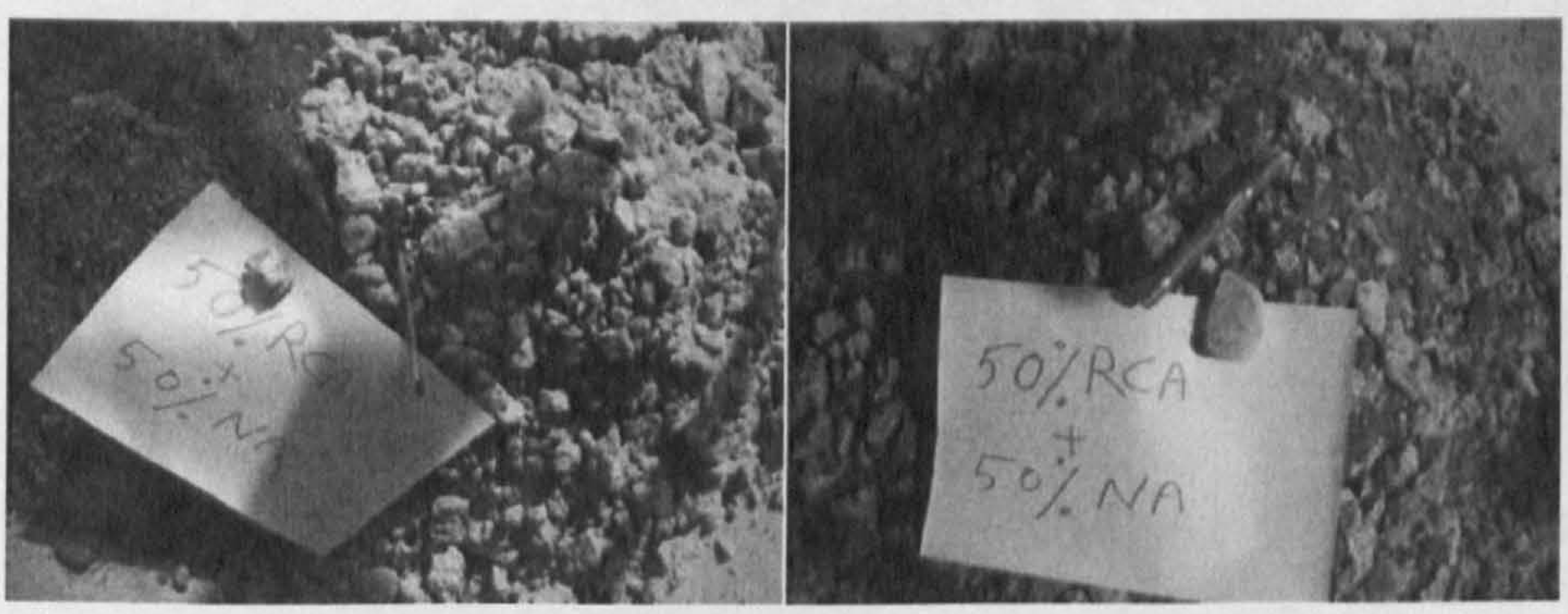


Figure 4.37 50%NA+ 50%RCA before and after blending

Figure 4.38 Bagging and labeling the mixes

CHAPTER 5 EXPERIMENTAL WORKS AND RESULTS

5.0 Preamble

This chapter includes a comprehensive experimental works performed in two phases with different sources: (i) Phase 1 which includes tests carried out on some materials with the sources of the UK based on British standards and cods common in the UK. This phase is discussed in sections 5.1 to 5.14. (ii) Phase 2 which includes test carried out on some materials with the sources of the Iran and according to standards and cods common in Iran. This phase is discussed in sections 5.15 to 5.17.

PHASE 1: TESTS CARRIED OUT IN THE UK

The tests performed in the UK were based upon the British Standards (BS) and MCHW Series 800. Two main standards for unbound subbases are BS EN 13285 and BS EN 13242 which present the essential requirements of material mixtures in highway design and construction. All specifications have been categorized in BS EN 13285 and these categories have been defined in BS EN 13242.

5.1 Mixture Requirements

Type 1 unbound subbase shall be made from natural sands, gravels, crushed rock, crushed slag, crushed concrete, recycled aggregates or well burnt non-plastic shale (MCHW series 800, 2007). Table 5.1 shows the requirements of this type of subbase according to MCHW Series 800.

Table 5.1 Requirement for aggregates used in subbase-Type 1, unbound mixtures (After MCHW series 800, 2007)

STANDARD	PROPERTY	REQUIREMENT
BS EN 13285	Mixture requirement category -Designation -Maximum fines -Oversize	 0/31,5 UF_9 OC_{75}
	Grading requirement category - Overall grading	G_P
BS EN 13242	Crushed, or broken and totally rounded particles - crushed rock, crushed artificial and crushed recycled aggregates	$C_{90/3}$
	Resistance to fragmentation - Los Angeles test	LA_{50}
	Resistance to wear - micro-Deval test	$M_{DE}NR$ (no requirement).
	Resistance to freezing and thawing magnesium sulphate soundness	MS_{35}
	Water absorption	$WA_{24}NR$ (no requirement)
	All other BS EN 13242 aggregate requirements	Category $_{NR}$ (no requirement).
Atterberg Limits	Plasticity Index (Ip)	Zero(Non-Plastic)

The particle sizes for Type 1 unbound subbase materials shall lie within the ranges given in Table 5.2.

Table 5.2 Summary grading requirements for unbound subbase-Type 1 (After MCHW series 800, 2007)

Sieve size, mm	Overall grading range
63	100
31.5	75-99
16	43-81
8	23-66
4	12-53
2	6-42
1	3-32
0.063	0-9

When RCAs are used in unbound mixtures in accordance with MCHW Clause 710, they shall also comply with the additional requirements of Table 5.3.

Table 5.3 Additional requirements for RCA used in unbound subbase-Type 1 (After MCHW series 800, 2007)

Component Identified by Clause 710	Maximum Permitted Content (% by mass)
Asphalt (Class A)	50
Glass (Class G)	25
Other materials (Class X), including wood, plastic and metal	1

Flakiness Index is not a requirement for subbase material according to MCHW series 800. But because of its importance based on reviewed literature, flakiness index was found for different kinds of RCA, NA and RAP according to BS EN 933-3:1997 which were as follow:

Material	Flakiness Index
RCA (three different kind)	5.8%, 8.25%, 9%
NA	16%
RAP	13.4%

5.2 Determination of Particle Size Distribution-Sieving Method

The samples of each kind of aggregates are divided up into smaller samples using a riffle box in accordance with EN 932-2:1999 and then sieved as required by BS 933-1:1997. The grading 'curves for the three kinds of aggregates, the upper and the lower range of Type 1 unbound mixtures given according to MCHW Series800 are illustrated in Figure 5.1.

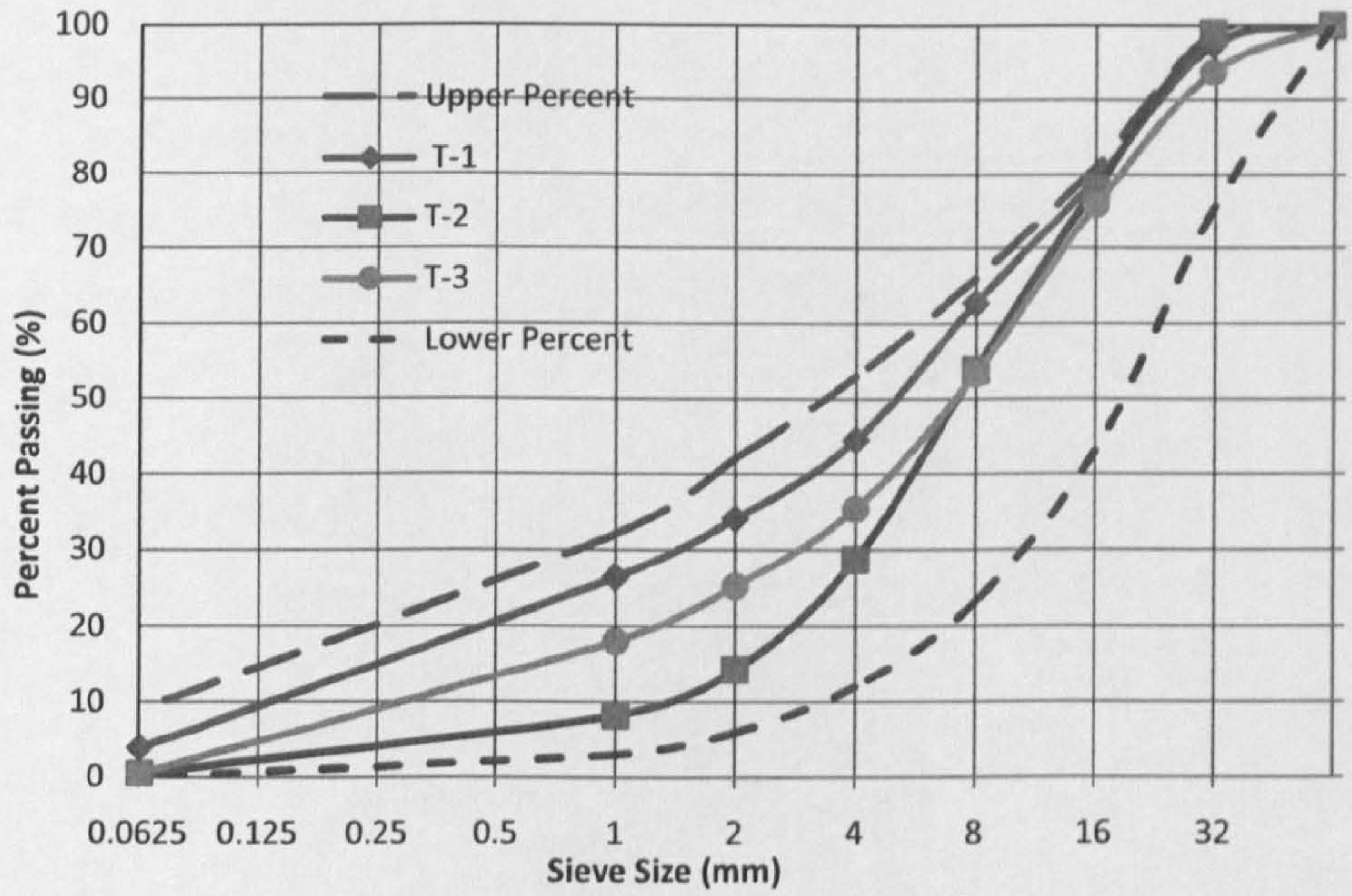


Figure 5.1 Grading curves for the tested aggregates

For all three kinds of aggregates; the mixture designation is 0/31,5, the Maximum fines content is UF_9 and Oversize is OC_{75} , the overall grading is in category G_P which meet the requirements of MCHW Series 800 requirements (MCHW Series 800, BS EN 13285:2003).

5.3 Atterberg Limits

5.3.1 Determination of the liquid limit

The liquid limit is the empirically established moisture content at which the material changes in state from plastic to liquid. The Liquid limits were determined according to BS 1377-2: 1990- Part 2 using the cone penetrometer method. This method is fundamentally more satisfactory than the Casagrande type, because it is essentially a static test depending on aggregates shear strength. The specimens were taken from aggregates passing the 425μm test sieve. After adding water and putting in cone penetrometer, penetration ranges were read from 15mm to 25mm. The moisture content corresponding to a penetration of 20 mm was read off the graph to the nearest 0.1%.The Liquid limits of T-1, T-2 and T-3 were obtained

33.6%, 22.4% and 33.3%, respectively. These magnitudes are illustrated in Figure 5.2.

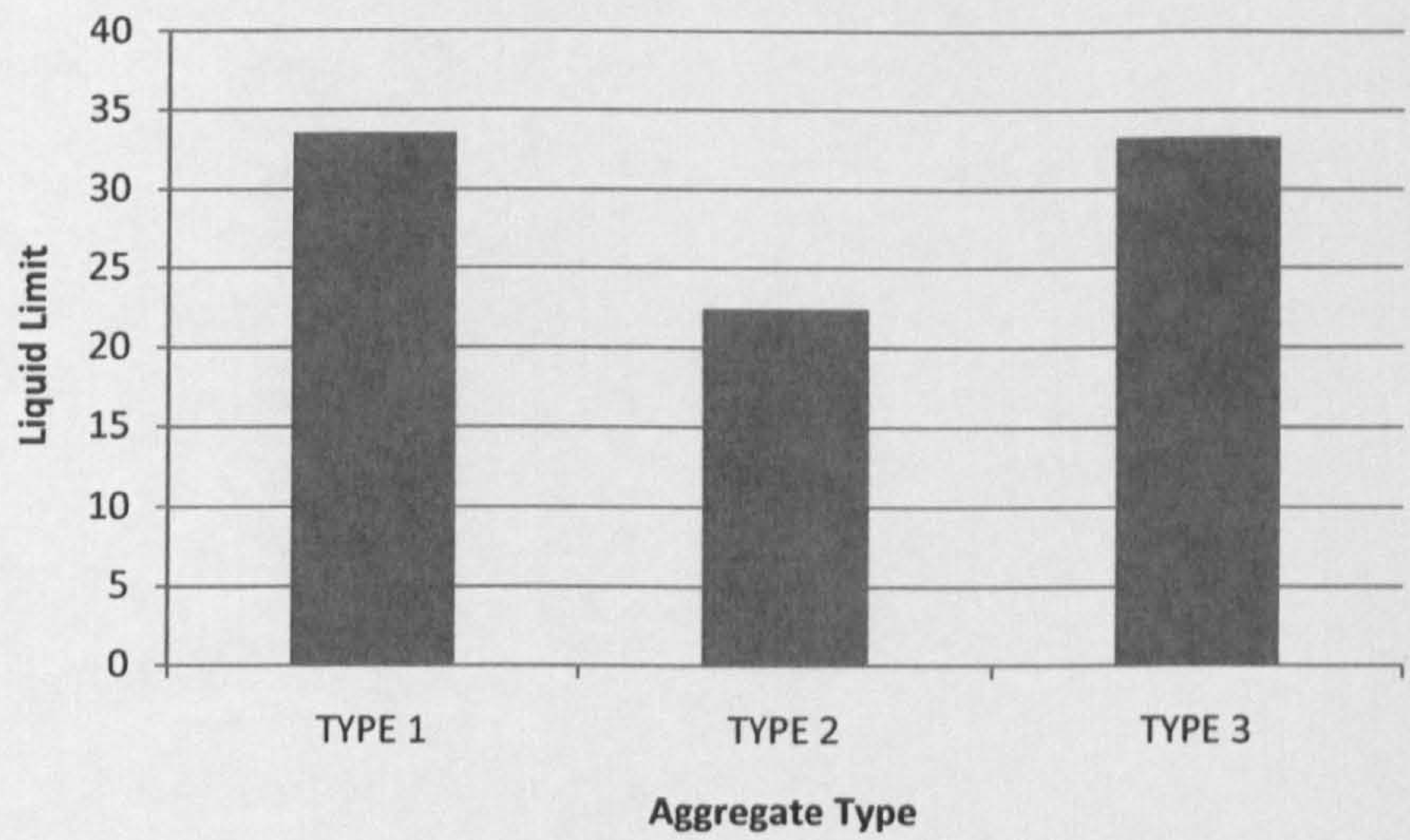


Figure 5.2 Liquid limits of aggregates

As can be seen from Fig. 5.2, natural aggregates have lower liquid limits than recycled aggregates.

5.3.2 Determination of the Plastic Limit and Plasticity Index

The plastic limit is the empirically established moisture content at which the material changes from the solid to plastic state. At moisture contents below the plastic limit the material is too dry to be exhibit plastic deformation. Both of these parameters were determined according to BS 1377-2: 1990- Part 2. As done for liquid limit tests, the specimens for plastic limit tests were taken from aggregates passing the 425μm test sieve. In the first trial for plastic limit condition, T-1 material was not cohesive enough to reach the plastic limit. With the addition of more water, the material became smoother and more cohesive. Rolling of a thin thread of the material on a flat glass surface was impossible in the second and third trials due to crumbling. Therefore the samples could not be rolled down to 3 mm diameter threads. This therefore led to the conclusion that the material was non-plastic (Figure 5.3).

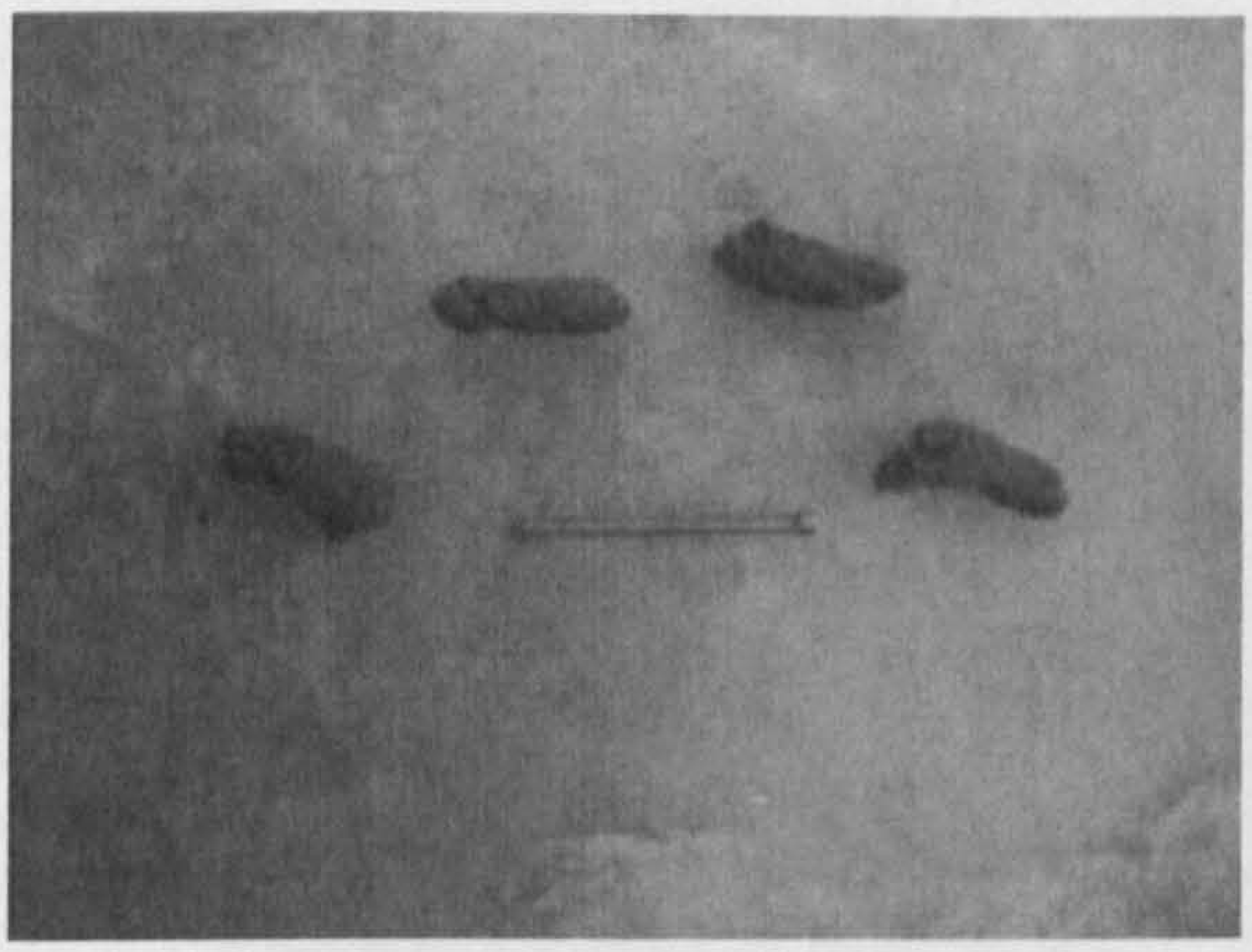


Figure 5.3 Crumbled samples of T-1 after rolling

Rolling out of T-3 to 3 mm diameter thread was also impossible and so it was also classed as non-plastic. In contrast, the T-2 material was rolled successfully to 3 mm and gave a plastic limit of 15.97%. In summarizing, the recycled aggregates were non-plastic (NP) but the natural aggregates had a plasticity index (PI) of 6.43%.

5.4 Classification Test for the Constituents of Coarse Recycled Aggregates

In order to specify and estimate the relative proportions of constituent materials for test mixes, classification tests were carried out on the coarse recycled aggregates according to BS EN 933-11:2009 and clause 710 of MCHW Series 700 (2007). Two washed and dried samples falling in the size range 8 mm - 63 mm were used to measure the total percentages of the following groups of materials:

(i) Class A: asphalt and bituminous materials;

(ii) Class B: clay masonry units such as bricks and tiles, calcium silicate masonry units, aerated non-floating concrete

(iii) Class C: concrete, concrete products, mortar and concrete masonry units

(iv) Class G: glass

(v) Class L: lightweight particles

(vi) Class U: unbound aggregate;

(vii) Class X: other particles such as: cohesive (i.e. clay and soil), miscellaneous: metals (ferrous and non-ferrous), non-floating wood, plastic and rubber and gypsum plaster.

Figures 5.4 and 5.5 describe the test procedure and results respectively.

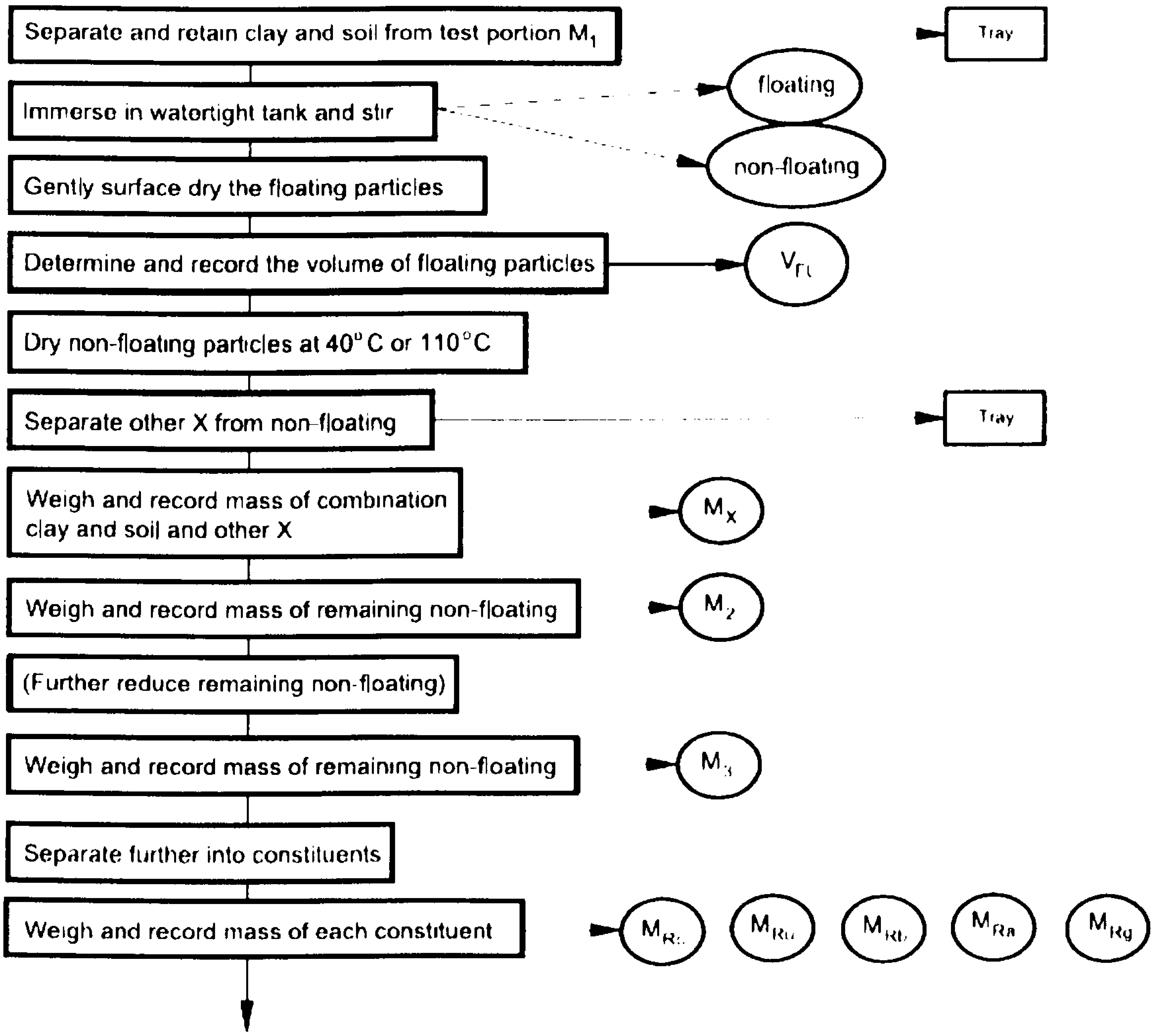


Figure 5.4 Flow chart describing the test procedure (BS EN 933-11:2009 and MCHW Series 700, 2007)

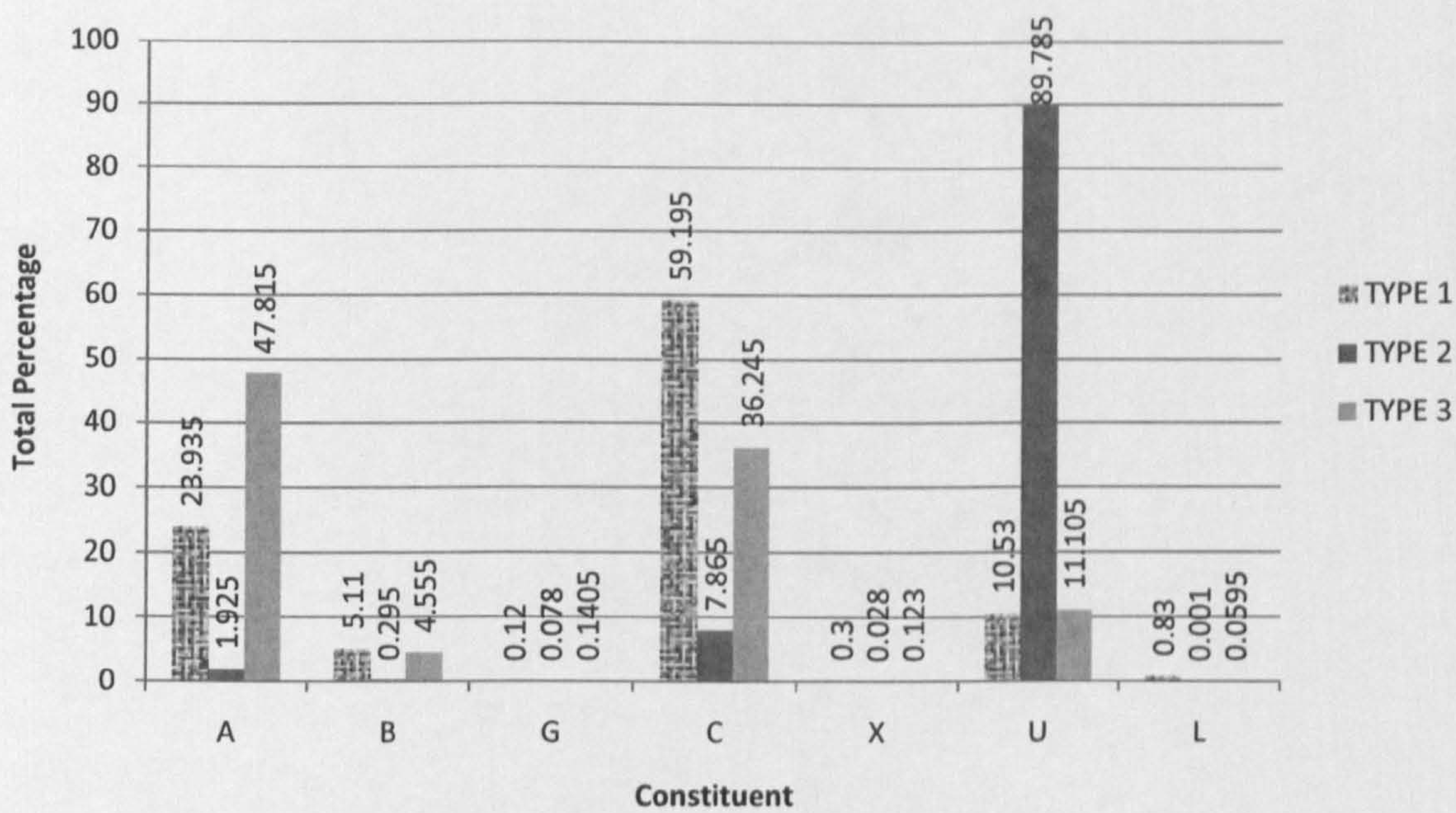


Figure 5.5 Constituents percentage in mixes

5.5 Determination of Water Absorption and Densities

5.5.1 Current specifications

The determination of water absorption was based on the provisions outlined in the three documents: BS EN 13285:2003, MCHW Series 800 and DMRB HD 35/04. Four types of unbound mixtures for subbase application are defined by MCHW Series 800. To determinate of water absorption rate, BS EN 1097-6 requires that the method to be used must be selected based on the particle size range as outlined below (BS EN 1097-6:2000):

Particle size range	Method used
31.5 mm – 63 mm	Wire-basket method
4 mm - 31.5 mm	Pyknometer method
0.063 mm – 4 mm	Pyknometer method

Preparation of test portions in both wire-basket and pyknometer methods included sampling and reduction which were performed in accordance with BS EN 932-1 and BS EN 932-2, respectively.

Density measurements included: (a) oven-dry particle densities (ρ_{rd}), (b) saturated and surface-dry density (ρ_{ssd}), (c) apparent particle density (ρ_a). The parameter

ρ_{rd} is the characteristic used for calculating the volume occupied by the aggregate, for various mixtures of RCA, NA and RAP. In addition to these solid particles, the voids including permeable and impermeable voids (but not including the voids between particles) must be considered to calculate the masses. Interpretation of ρ_{ssd} is based on aggregates soaked in water for 24 hours in water to satisfy the water absorption requirement. The weight of aggregates and water within the voids filled to the extent achieved in soaking time (but not including the voids between particles) were calculated to find the mass in air of each mixture. (ρ_a), pertained to the relative density of aggregates not including the permeable pores.

5.5.2 Method statements

The method statements for wire-basket and pyknometer method are summarized in Figure 5.6 and Figure 5.7, respectively:

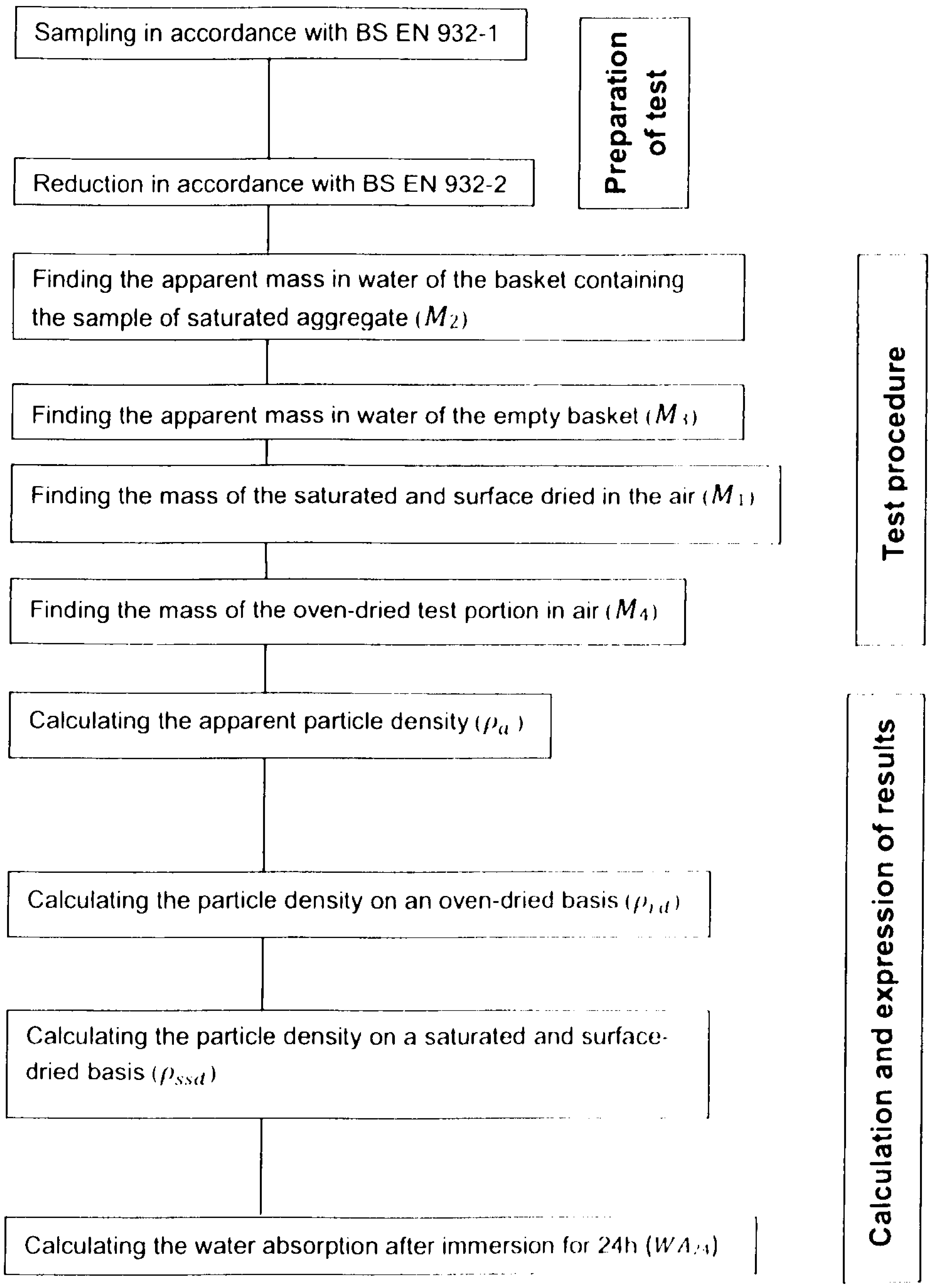


Figure 5.6 The stages involved in wire basket method

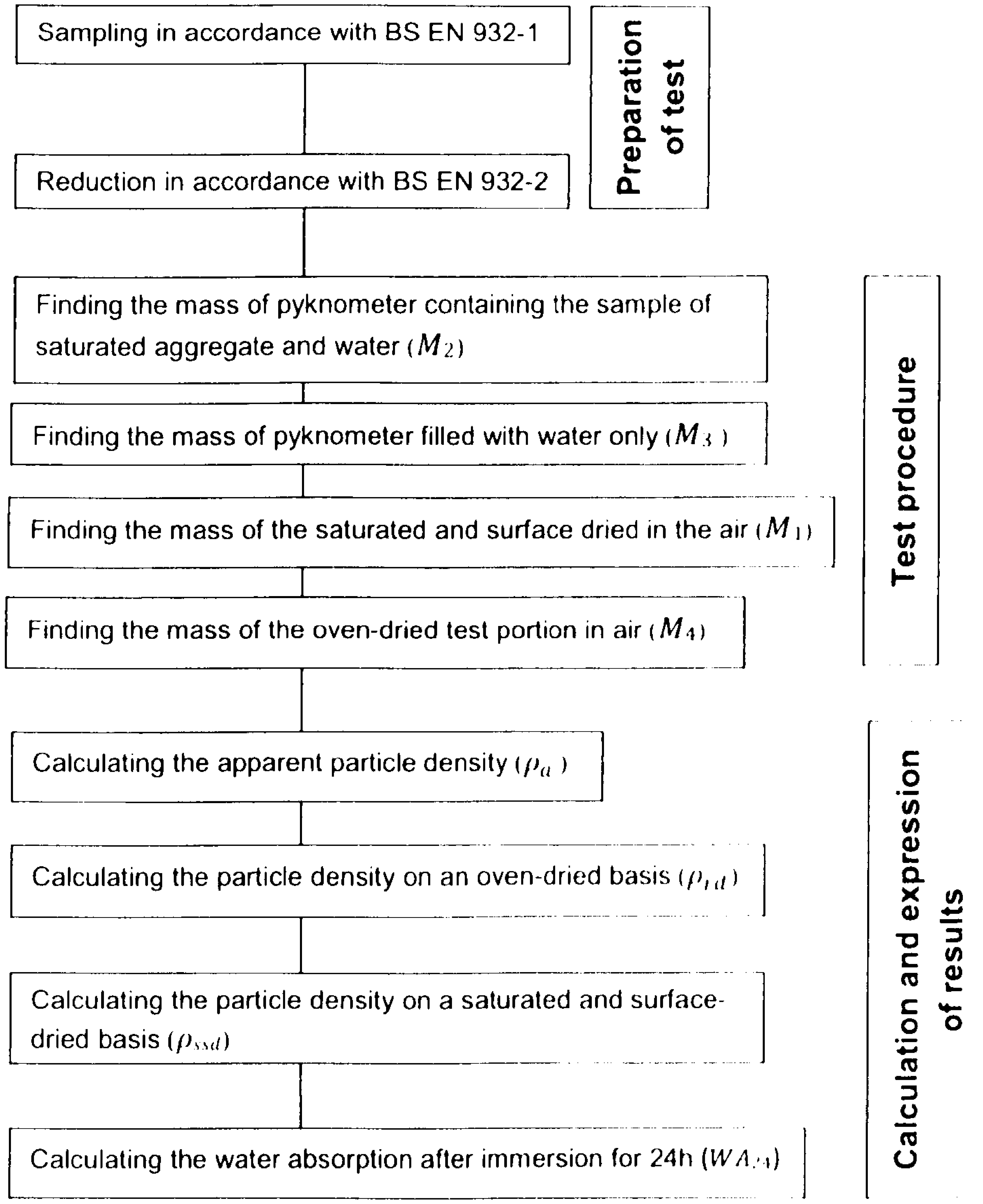


Figure 5.7 The stages involved in the pyknometer method

In the wire-basket method, the prepared test portion was placed in a wire-basket and immersed in water for a period of 24 hours, so all entrapped air was removed. To dry the surface of saturated aggregates, the test portion transferred to a second dry soft absorbent cloth and was spread and turned until all visible films of water were removed, but the aggregate still had a damp appearance. The oven-dry mass was achieved by placing the test portion in an oven set at a temperature of 110 ± 5 ℃ for as long as necessary for the spec imen to reach a constant mass.

In the pyknometer method, the prepared test portion was immersed in water pyknometer for the soaking period of 24 hours to remove all entrapped air.

According to BS EN 1097-6:2000 the pyknometer including aggregates and water must stand in the water bath and keep the test portion at a temperature of (22 ± 3) ℃ for (24 ± 0,5) h. In this research, to get a hig her accuracy, the pyknometer and the refill water was placed in a curing room, which was maintained at a temperature of 20℃ for 24 hours (Figs. 5.8 and 5.9). Achieving saturated and surface-dried aggregates for particles between 4mm and 31.5mm was exactly the same as aggregate particles between 31.5mm and 63mm. But to assess whether the surface dry state has been achieved for aggregate particles between 0.063mm and 4mm, the metal cone mould test was applied.

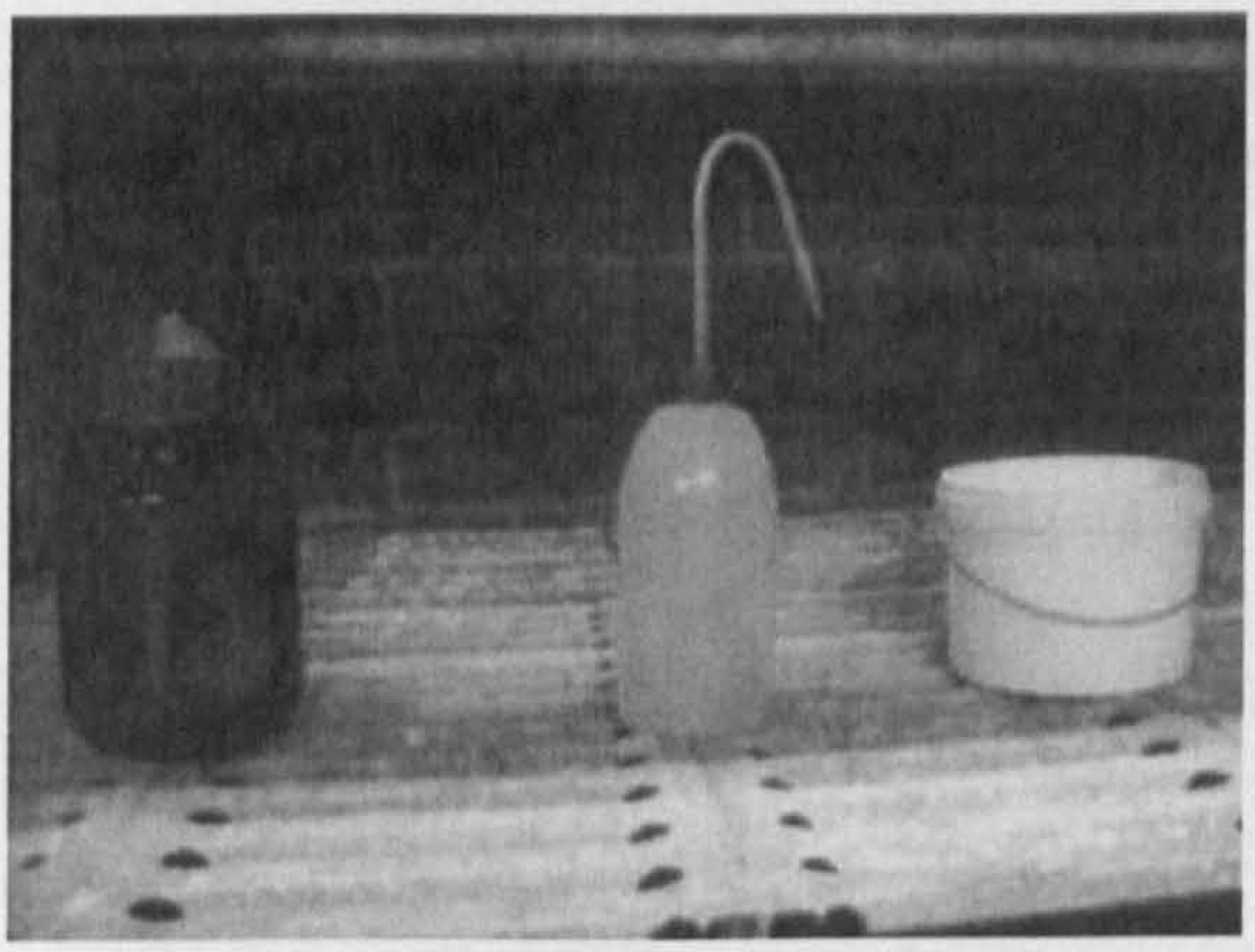


Figure 5.8 The Pyknometer, squeeze bottle and plastic pot in the curing room

Figure 5.9 Keeping the temperature of test portion and water constant at 20℃ in the curing room

5.5.3 Test results and discussions

The test results for materials of different particle sizes are summarized in Figs. 5.10 to 5.13.

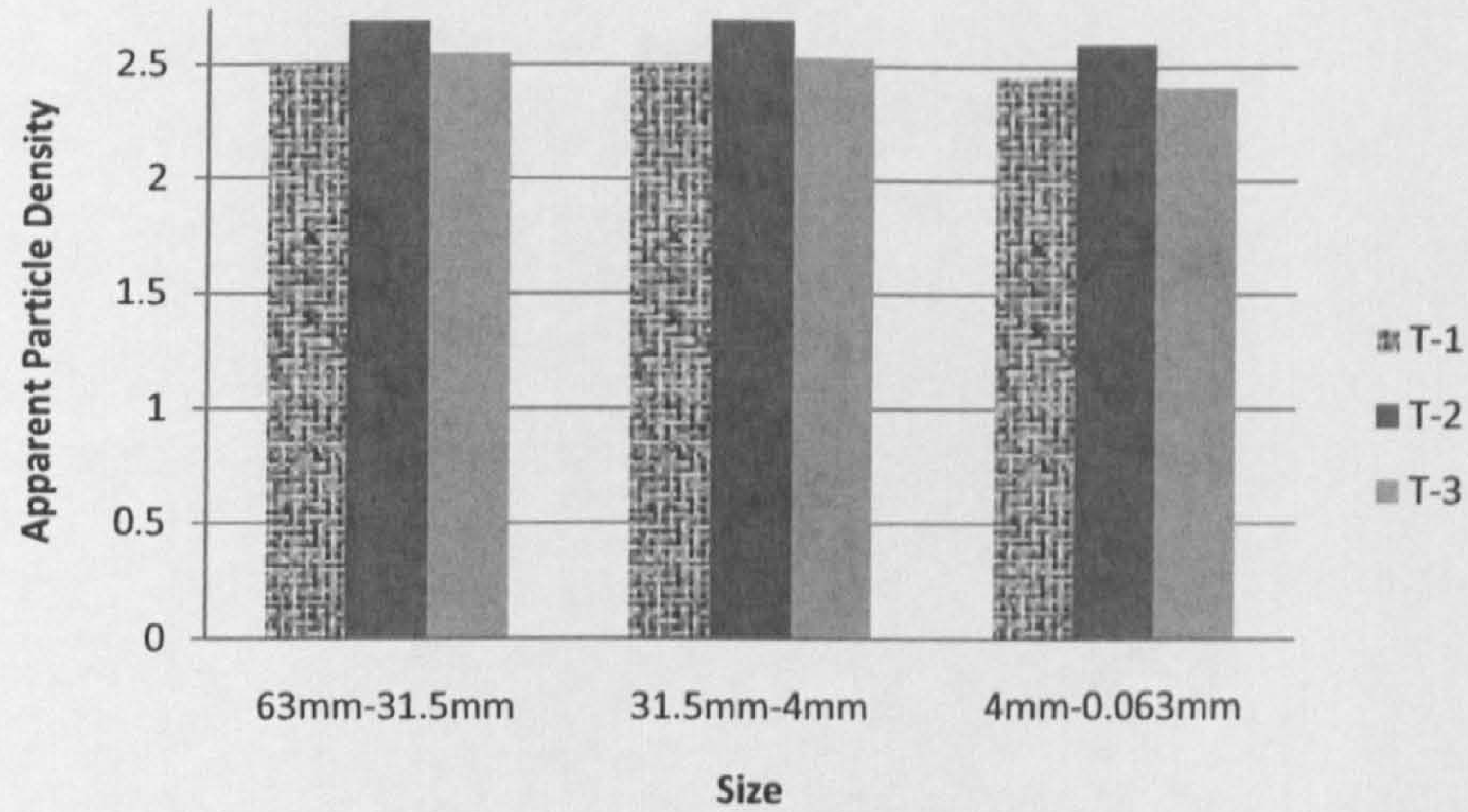


Figure 5.10 ρ_a values for subbase material

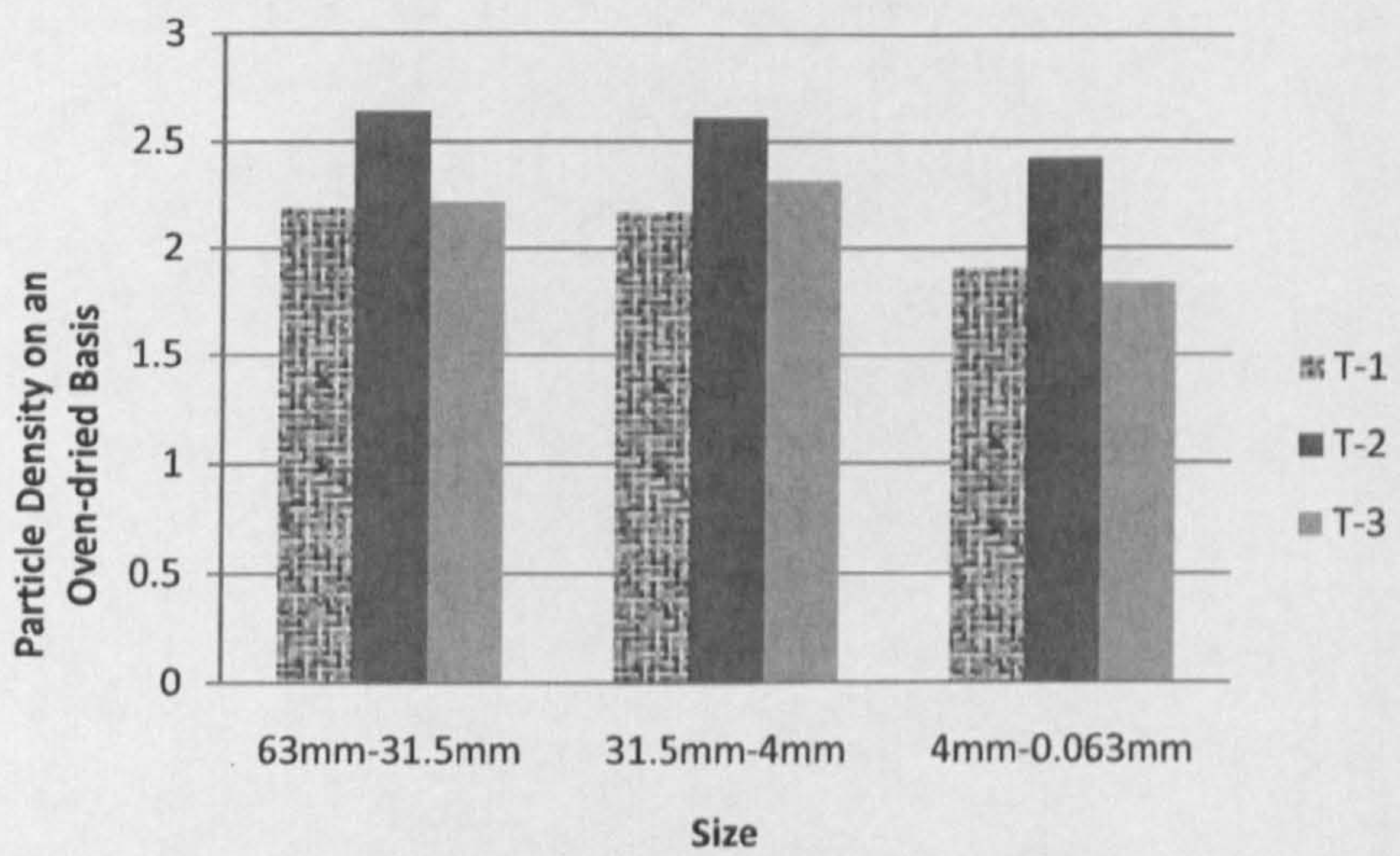


Figure 5.11 ρ_{rd} values for subbase material

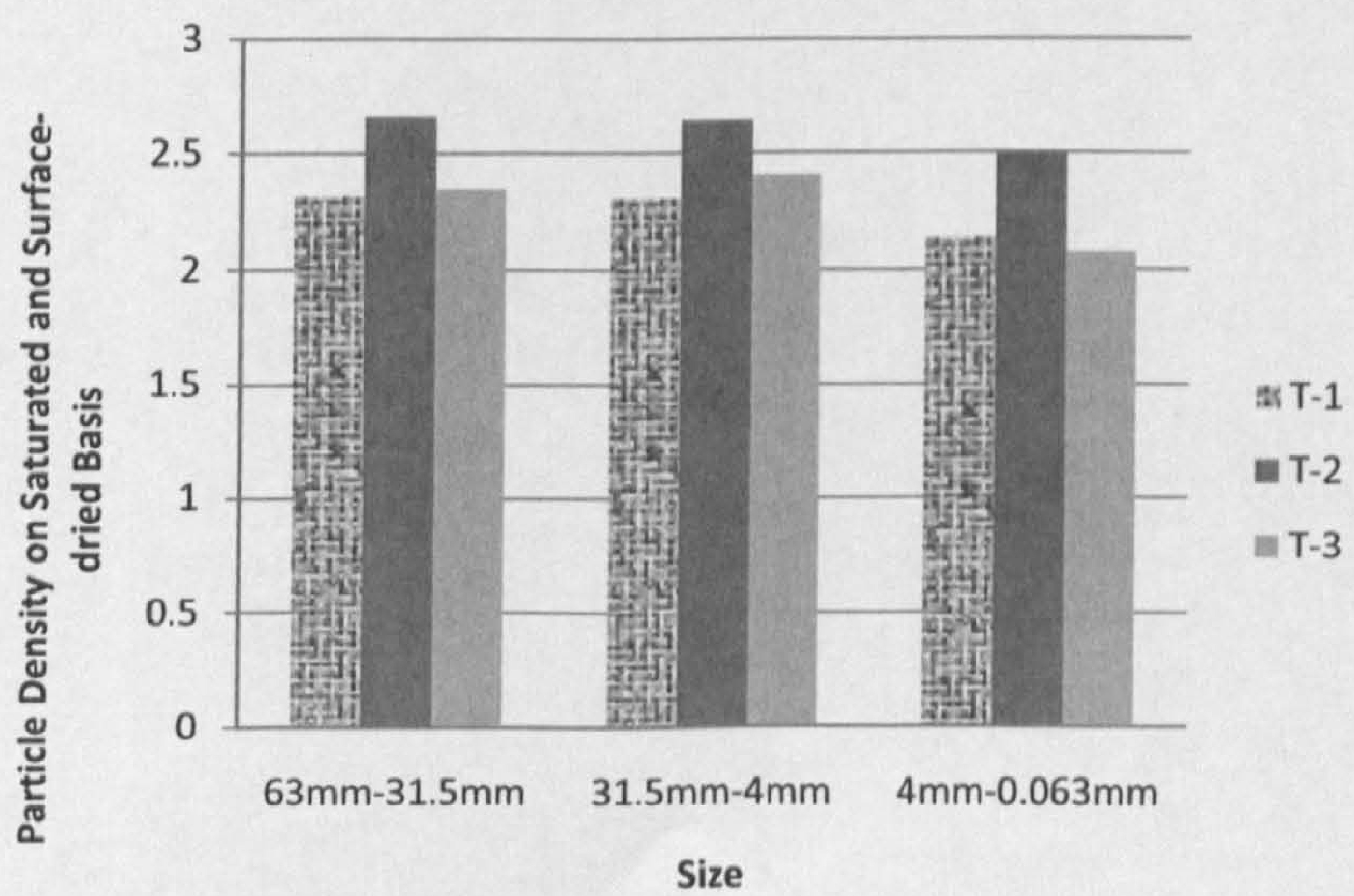


Figure 5.12 ρ_{ssd} values for subbase material

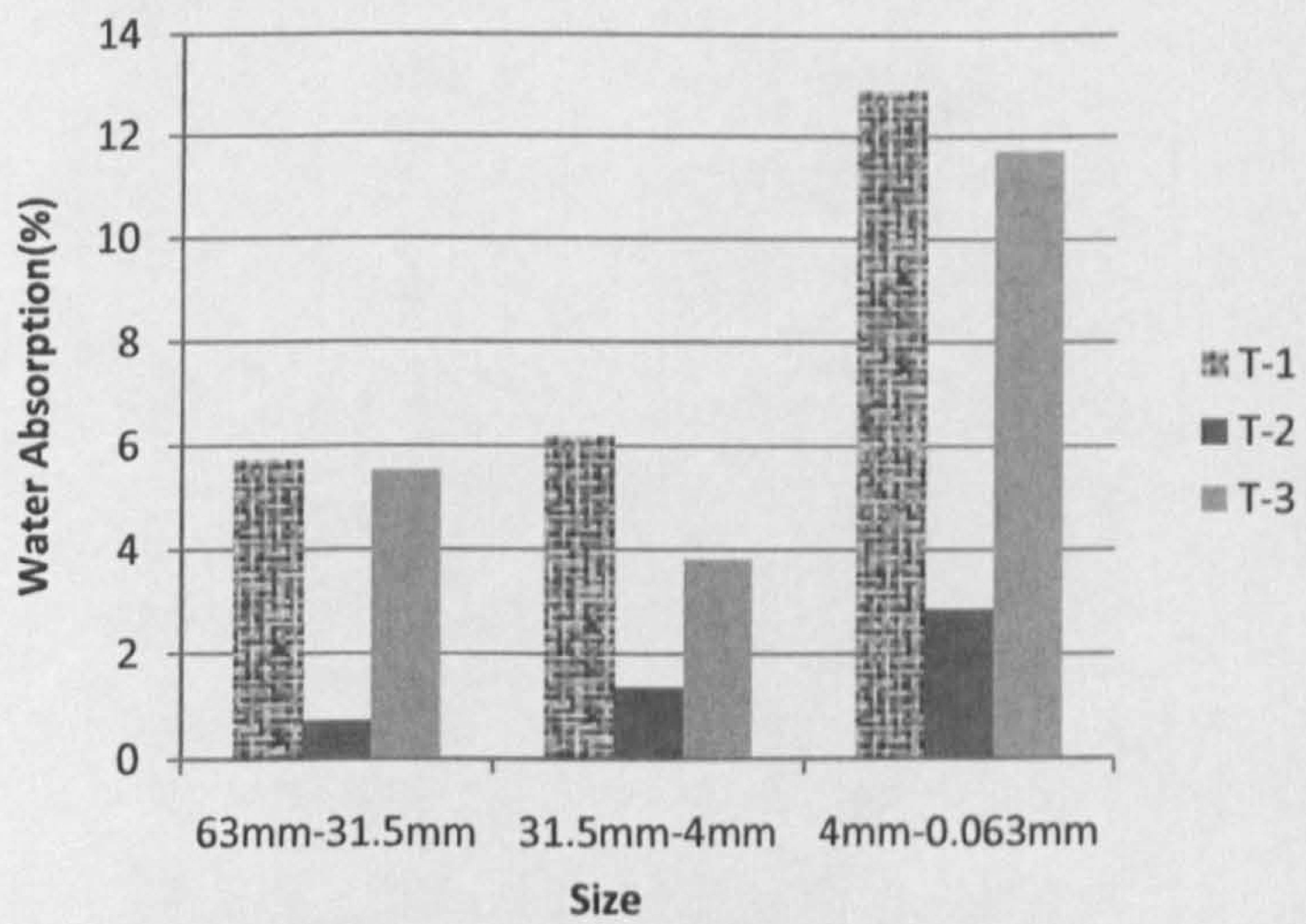


Figure 5.13 WA_{24} values for subbase material

The bar charts in Figs. 5.10 - 5.13 compare the densities (ρ_a, ρ_{rd}, ρ_{ssd}) and water absorption rates (WA_{24}) for 24 h immersion periods. To calculate these densities, the density of water was obtained (from the table mentioned in BS EN 1097-6:2000) at the temperature recorded when the apparent mass in water of the basket containing the sample of saturated aggregate (M_2) was determined.

The tests result showed that natural aggregates had the highest ρ_a, ρ_{rd}, ρ_{ssd} for particles in all ranges. Conversely, they had the lowest water absorption for particles in all fractions.

As seen in Figs. 5.10-5.13, for a given kinD of density (ρ_a, ρ_{rd}, ρ_{ssd}) the trend of variation of the density with particle size is consistent for all three materials types (T-1, T-2 and T-3). The presence of the cement mortar adhering to the stone particles in the RCA creates a relatively weak, porous and cracked layer (Tam et al. 2007b) thereby resulting in a decreased density of the RCA relative to NA. It is likely that the porous layer described above, in addition to residual unhydrated cement, lead to an increase in the water absorption rate of RCA.

As the density of cement mortar (around 1.0–1.6 Mg/m^3) (Tam and Le 2007a) is less than that of stone particles of about 2.60 Mg/m^3, the smaller the particle density, the higher the cement mortar content adhering to the RCA. The reason for the particle densities being lower in RCA as compared to NA is due to the relatively high porosity and low density of the cement paste adhering to the RCA particles.

Furthermore, particle densities of coarse aggregates are larger than those of fine aggregates, implying that a higher amount of cement mortar is attached to the fine aggregates. This behavior affects the water absorption rate of RCA in the sense that fine aggregates of RCA have a higher water absorption rate, a pattern which is consistent with the findings reported by Tam and Le (2007a).

In the mix of RCA and RAP (T-3), bitumen coated particles in reclaimed asphalt pavements are observed mostly in the 31.5mm-4mm particle size range. Thus the particles in the size range of 4mm-0.063mm are less coated with bitumen in T-3 and only a very small part is coated (Figures 5.14 and 5.15). These observations make the water absorption value of T-1 about 40% higher than T-3 in 31.5mm-4mm, but the water absorption of T-1 and T-3 are relatively close in the fine particles.

Figure 5.14 The fine particles of T-3

Figure 5.15 The fine particles of T-1

The particles in the size range 63mm-31.5mm of T-3 are mostly crushed concrete aggregates. This is why the water absorption and apparent particle density of coarse particles of T-3 are close to those of T-1. Figure 5.16 shows the particles after emptying the aggregates from the basket onto dry clothes and Figure 5.17 shows them after reaching constant mass.

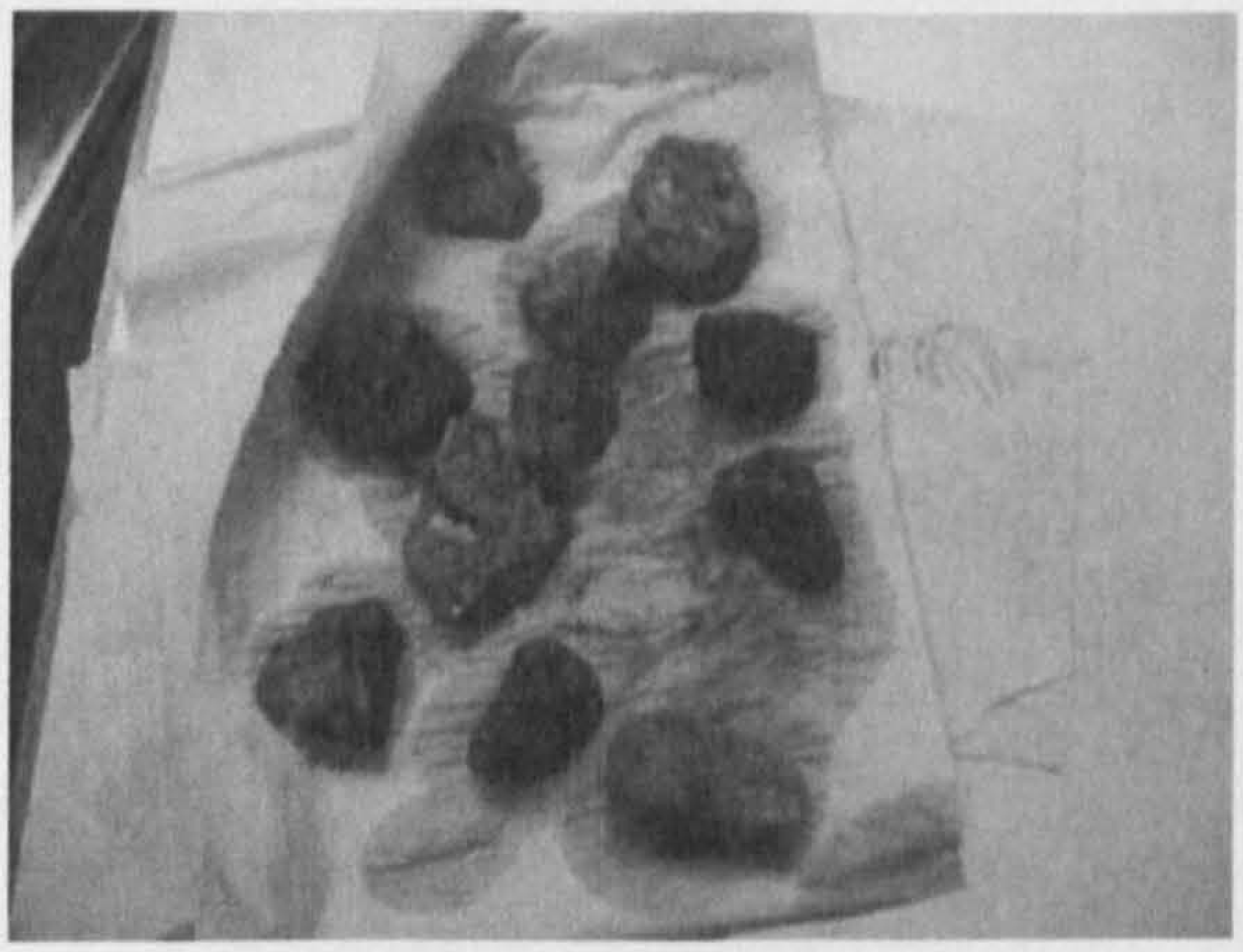


Figure 5.16 The coarse particles of T-3 after removal from water basket

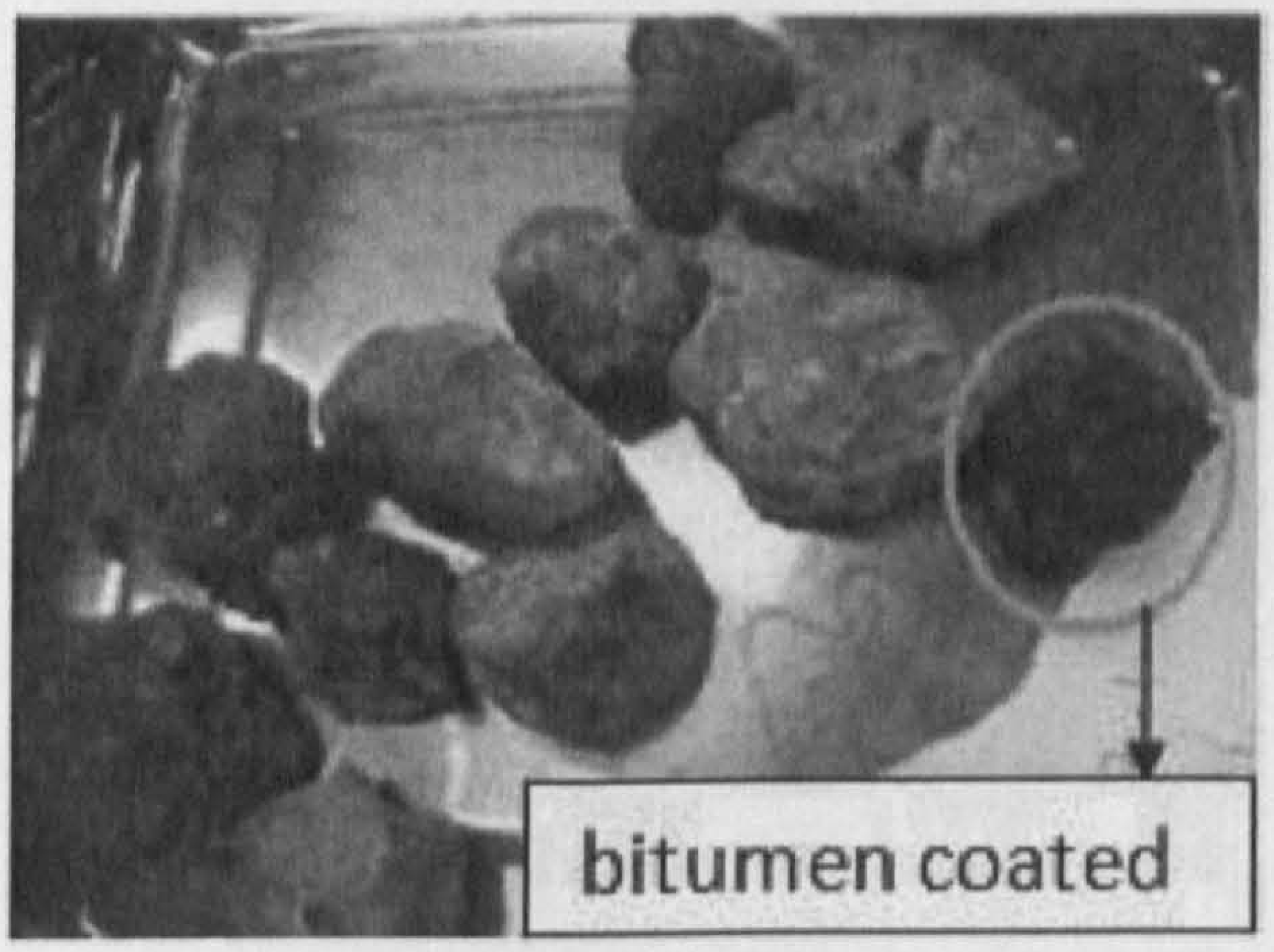


Figure 5.17 The coarse particles of T-3 dried to constant mass

5.6 Determination of Compaction

5.6.1 Current specifications

Test portions for determination of compaction properties were chosen from Type 1 subbase material. Tests were carried out in accordance with BS EN 13286 Part-2 to obtain the laboratory reference density and water content with using the large Proctor compaction mould (designated mould "B"), since all particles were smaller than 100mm and 97.4%, 98.96% and 93.72% of the particles in materials T-1, T-2 and T-3 respectively were finer than the 31.5 mm sieve. The modified Proctor mould B is 150mm in diameter and 120mm in height. Based on the recommendations in BS EN 13286 Part-2 the mass of sample required for the test was determined to be 40 kg. The additional specifications of the modified Proctor test are summarized below (BS EN 13286-2:2004):

Mass of rammer (B) = 4.5kg

Diameter of rammer= 50mm

Height of fall= 457mm

Number of layers= 5

Number of blows per layers= 56

These conditions increase the compaction effort with a specific energy of about 2.7 MJ/ m^3.

5.6.2 Method statement for determination of the moisture-density

Compaction test were carried out on 5 samples for T-1, 7 samples for T-2 and 8 samples for T-3. All these samples were finer than the 31.5 mm sieve (BS EN 13286-2), while AASHTO stipulates materials passing the 4.75 mm (No.4) sieve. (AASHTO Designation: T180-1 (2004)). The behavior of material T-1 was such that gradual variation of water content followed by standard compaction produced a clear pattern of dry density versus moisture content curve from which the maximum dry density could be easily discerned. In the case of materials T-2 and T-3, it was found necessary to test more than 5 compaction samples in order to obtain meaningful results of MDD and OMC. To ensure homogenous mixes for compaction tests, all materials T-1, T-2 and T-3 were completely dried using a warm air dryer before starting to vary moisture contents. Figure 5.18 shows drying the T-1 samples with warm air current before compaction to reach the constant mass.

Figure 5.18 Drying with warm air dryer

The following calculations were carried out in order to produce the complete compaction characteristics of the materials on standard kinds of graphs:

- Dry densities and corresponding water contents were calculated from the equation:

  $\rho_d = (100 * \rho) / (100 + w)$ **(Equation 5.1)**

  Where

  ρ_d is dry density in (Mg/m^3), ρ is bulk density in (Mg/m^3) and w is water content of mixture, in percent (%).

- The curve corresponding to 0% air voids and the particles density was calculated from the equation:

  $\rho_d = (\rho_s{-1} + 0.01 \cdot w \cdot \rho_w{-1})$ **(Equation 5.2)**

  Where

  ρ_d is dry density in (Mg/m^3), ρ_s is the particle density, in (Mg/m^3), ρ_w is the density of water assumed equal to 1 Mg/m^3 and w is water content, in percent (%).

5.6.3 The Results of compaction tests

On completing the compaction tests, for each kind of aggregates two graphs were drawn as shown in Figs 5.19-5.21. Thus the optimum moisture content (OMC) and maximum dry density (MDD) were derived from the curves as shown in Figures 5.19 to 5.21. The following results were therefore extracted:

- 100%RCA had an OMC of 13.71% and an MDD of 1.84g/cm^3.
- 100%NA had an OMC of 6.71% and an MDD of 2.03 g/cm^3.
- 100%RCA+100%RAP had an OMC of 11.67% and an MDD of 1.89 g/cm^3.

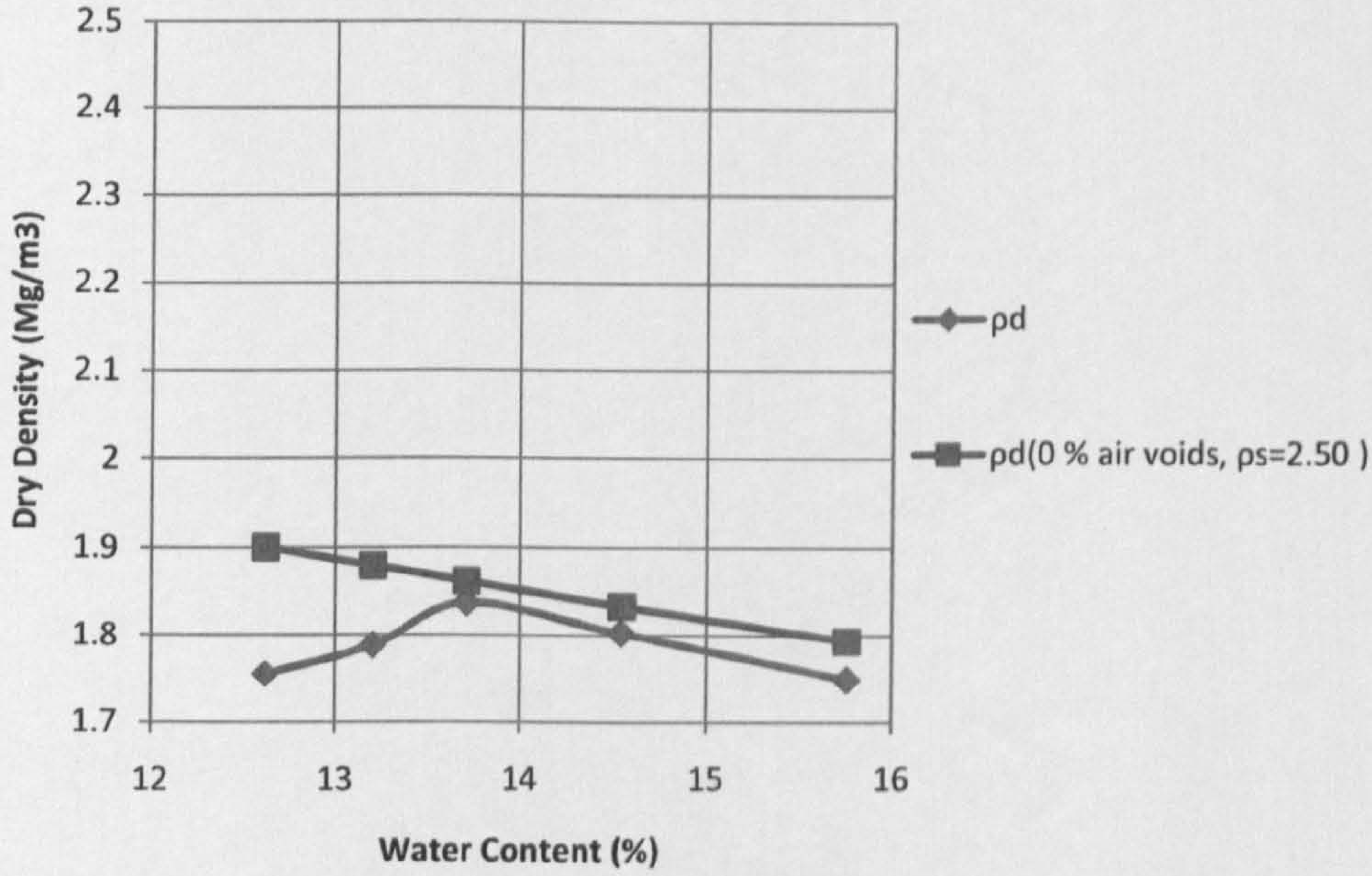


Figure 5.19 Dry densities and corresponding water contents of 100%RCA

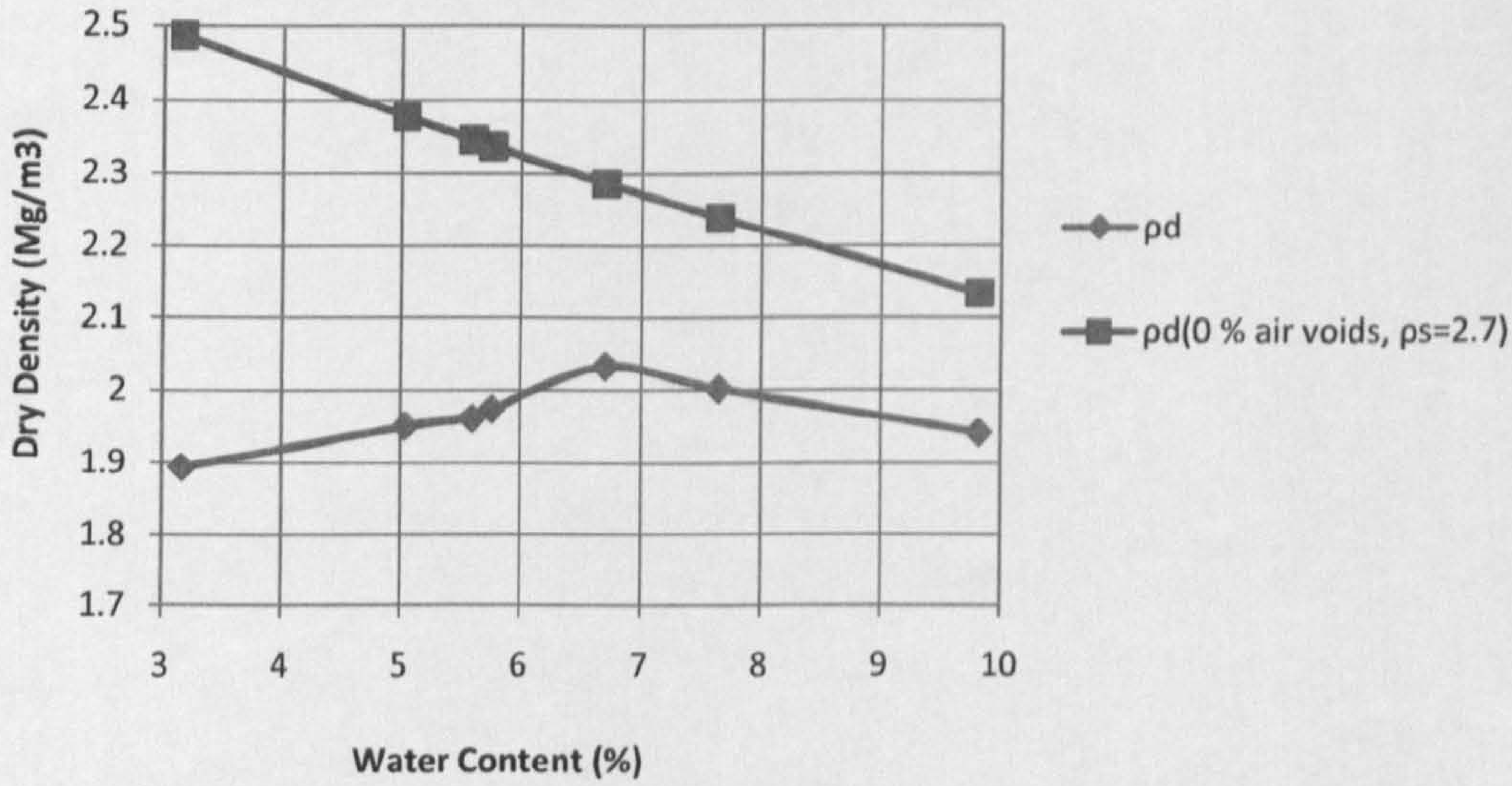


Figure 5.20 Dry densities and corresponding water contents of 100%NA

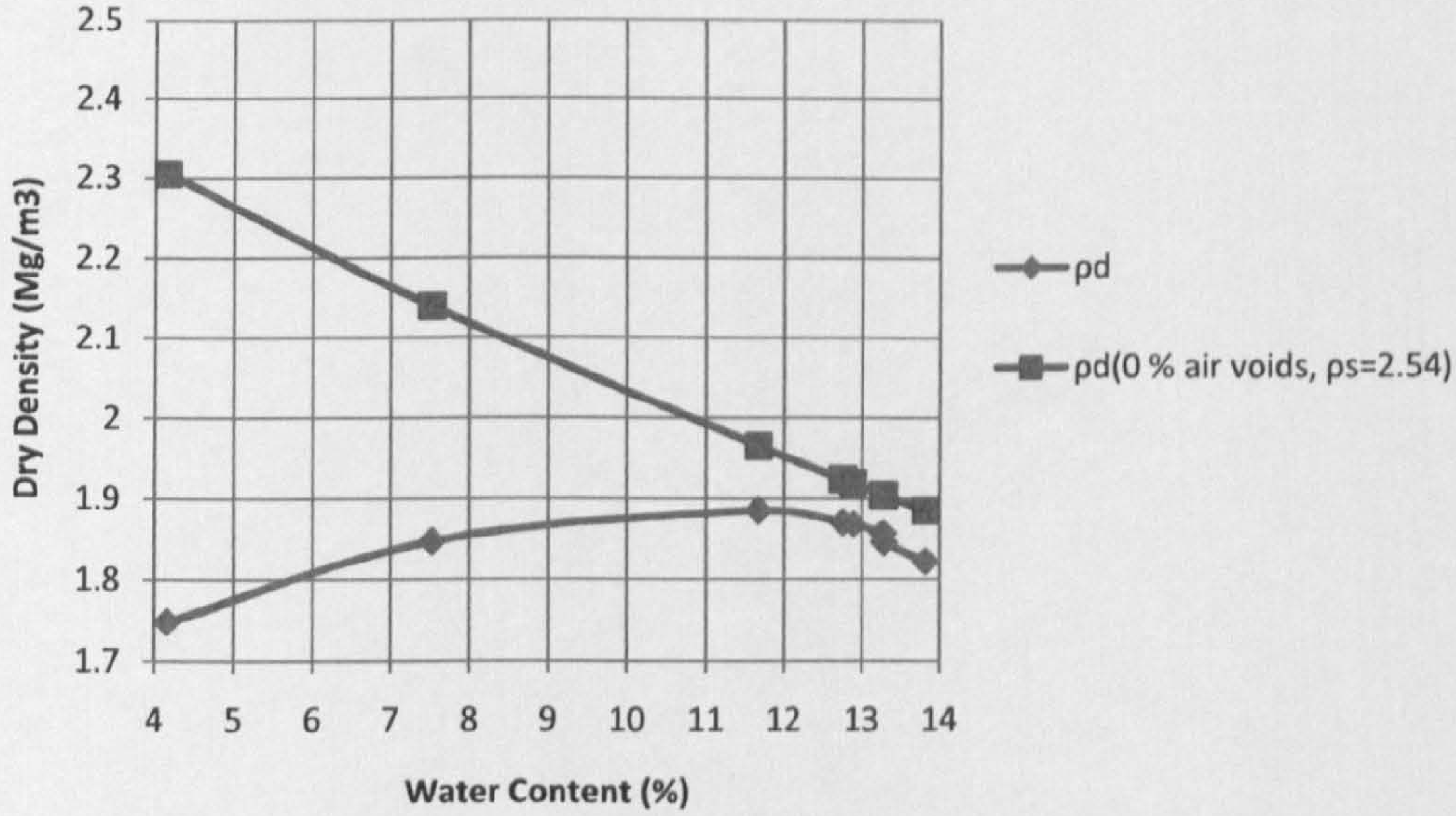


Figure 5.21 Dry densities and corresponding water contents of 50%RCA+50%RAP

During compaction it was observed that RCA particles broke into smaller pieces more than NA and RAP materials. It was also apparent that an increase in the percentage of fine materials in a RCA mix caused a decrease in the density and increase in the water content. Tam *et al.* 2007b reported that the any existing unhydrated cement in RCA adheres to the stone particles in the RCA to form a relatively weak, porous and cracked layer which decreases the density of the RCA compared to NA. Addition of water in RCA to a level slightly beyond the OMC creates a submerged condition, which aids compaction. This is an important point to be noted in actual practice on site. Turning to RAP, the effect of the bitumen layer coating the stone particles increases the flexibility of the bound aggregates so that breaking up of the stone particles under the action of a rammer is inhibited to an extent.

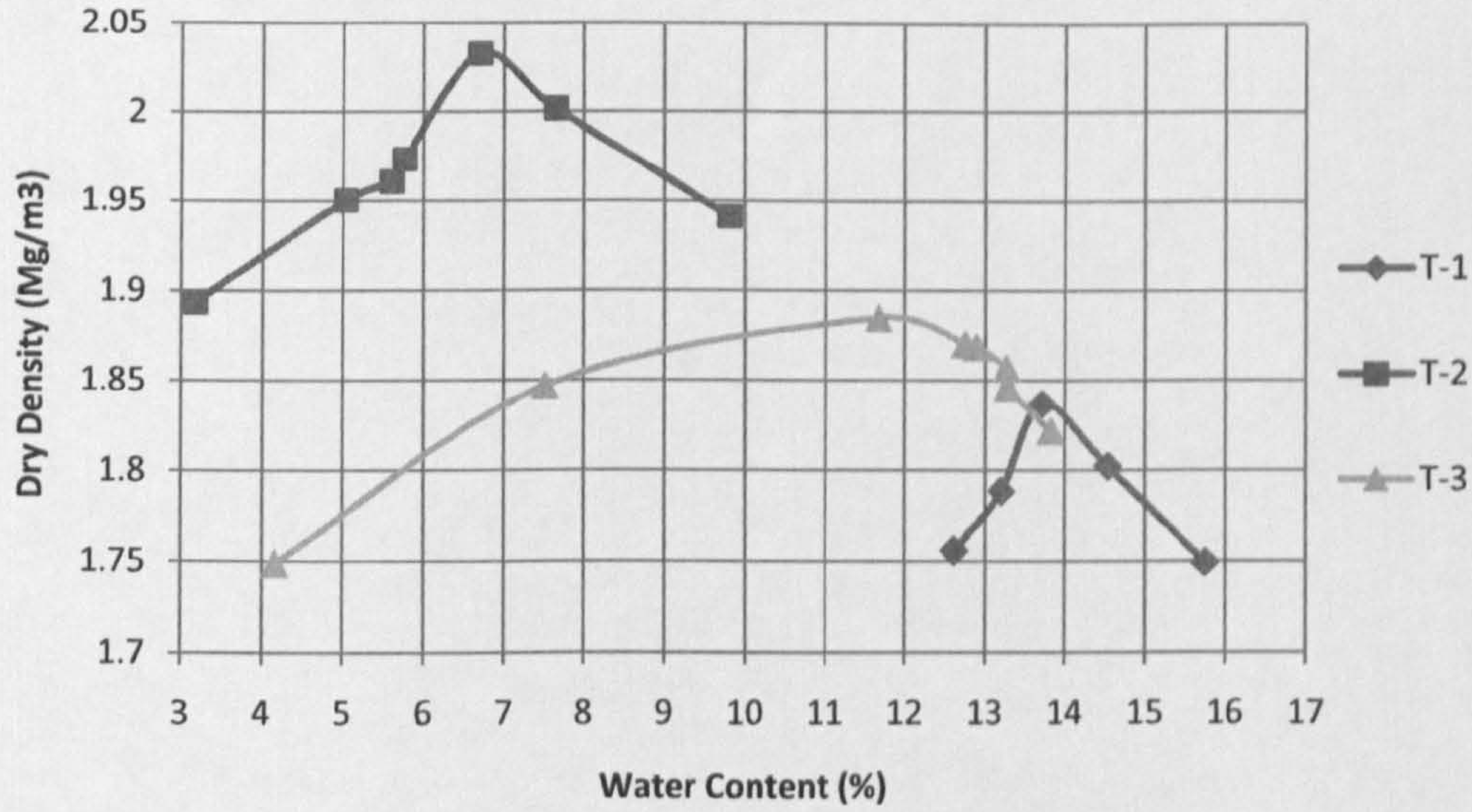


Figure 5.22 Moisture- density relationships for three subbase materials

From the graph in Figure 5.22 it is clear that T-1, which is the mixture containing 100%RCA, displayed the sharpest increase in dry density with increasing moisture content up to the OMC state. Therefore it is essential to compact this materials at close to OMC as possible in order to achieve the desired compaction state. As Fig. 5.21 shows, the incorporation of RAP in T-3 moved the compaction curve closer to the 0% air voids line particularly on the right hand-side of OMC (often called the "wet" side). This might be due to the regular shapes of bitumen coated particles in RAP, which therefore encourage more interlocking RAP with RCA particles in the mix for T-3. Of all the materials (T-1, T-2 and T-3), at OMC and beyond, the compaction curve for T-1 lies closest to the 0% air voids line (Figures 5.19 and 5.20). This means that the T-1 (100%RCA) achieved a greater compaction degree than the NA at OMC. This difference was mainly attributed to physical properties and the shape of RCA particles after breaking under rammer blows. The RCA particles were broken more easily than rounded corner natural aggregates, then the separated cements filled the voids and the coarse particles interlocked with each other very firmly.

5.7 Determination of CBR

5.7.1 Current specifications

BS EN13286 part-47 specifies the test methods for the laboratory determination of California bearing ratio. In this research, CBR moulds and rammers conforming to BS EN 13286-2 were used.

5.7.2 Method statement for determination of CBR

The test sample was taken from material passing the BS 22.4 mm sieve. According to BS EN 13286-2, if the CBR of the material was expected to be greater than 5% then a 40 N seating force was applied. The CBR values corresponding to a range of water contents were measured. To prepare a CBR test specimen, the material was compacted in the standard mould in 5 layers using 56 hammer blows per layer. During testing to develop the force-penetration curve, the specimen was subjected to plunger loads to maintain a constant rate of penetration of approximately 1.27mm/min. Load readings were recorded at penetration increments of 0.25mm up to a total penetration 10mm. After the test was completed, the specimen was removed from the CBR mould and dried wholly in an oven to determine the water content. Taking the whole specimen, rather than a small sample of it, for moisture content measurement helped minimize errors due to variations in moisture content in different parts of the tested specimen. For each material (T-1, T-2 and T-3) unsoaked CBR values corresponding to various moisture contents were measured and a plot of CBR versus moisture content plotted. In parallel, soaked CBR values for all three materials above were measured from specimens immersed in water for 96 hours.

5.7.3 The Results of CBR Tests

According to the recommendations of BS EN13286 parts 2 and 47 the CBR test is performed on materials having smaller particle sizes in comparison to the compaction test. This difference in particle sizes resulted in material T-2 showing an apparent OMC value of 8.24% for the CBR specimens in contrast to 6.71% (a difference of 23%) exhibited by the compaction test specimens, as typified by Figs. 5.22 and 5.33. The same difference in particle sizes resulted in only a marginal difference (<1%) in MDD for the CBR and compaction test samples of material T-2 (natural aggregate), as shown in Figs 5.22 and 5.33. There is some consistency between the present findings and those published by Robinson and Thagesen (2004, p. 152) and Scroochi and Sedighi Manesh (2009, pp. 80-83) who found that for natural aggregates, reducing the percentage of coarse particles from the mixture decreases the MDD and increases the OMC.

As for the recycled materials (T-1 and T-3), the trends of disparities in OMC between CBR and compaction test samples were opposite to the ones observed in T-2 (Figs. 5.22 and 5.33). For example, in T-1 the CBR test samples gave an apparent OMC of 11.90% in contrast to 13.71% shown by the compaction test samples (Figs. 5.22 and 5.33). During the tests, it was clear that the coarse particles of RCA absorbed water very quickly, probably due to exposed surfaces of cement paste and also the presence of dust particles. But the available unhydrated cement in the fine aggregates of RCA did not have enough opportunity to absorb moisture in the first few hours of wetting. This caused OMC to decrease with decreasing particle sizes.

For T-3 Figures 5.22 and 5.33 shows that MDD increases and OMC decreases when particles become smaller. In material T-3, owing to differences in the shapes of particles in RAP and RCA, there was reduced particle interlocking (which increase the porosity), resulting in a decrease in MDD. It was apparent that particle interlocking was better in the CBR test samples than in the compaction samples, as judged by the MDD values from these tests. As Figs. 5.22 and 5.36 illustrate, the CBR test samples had a lower OMC than the compaction test samples.

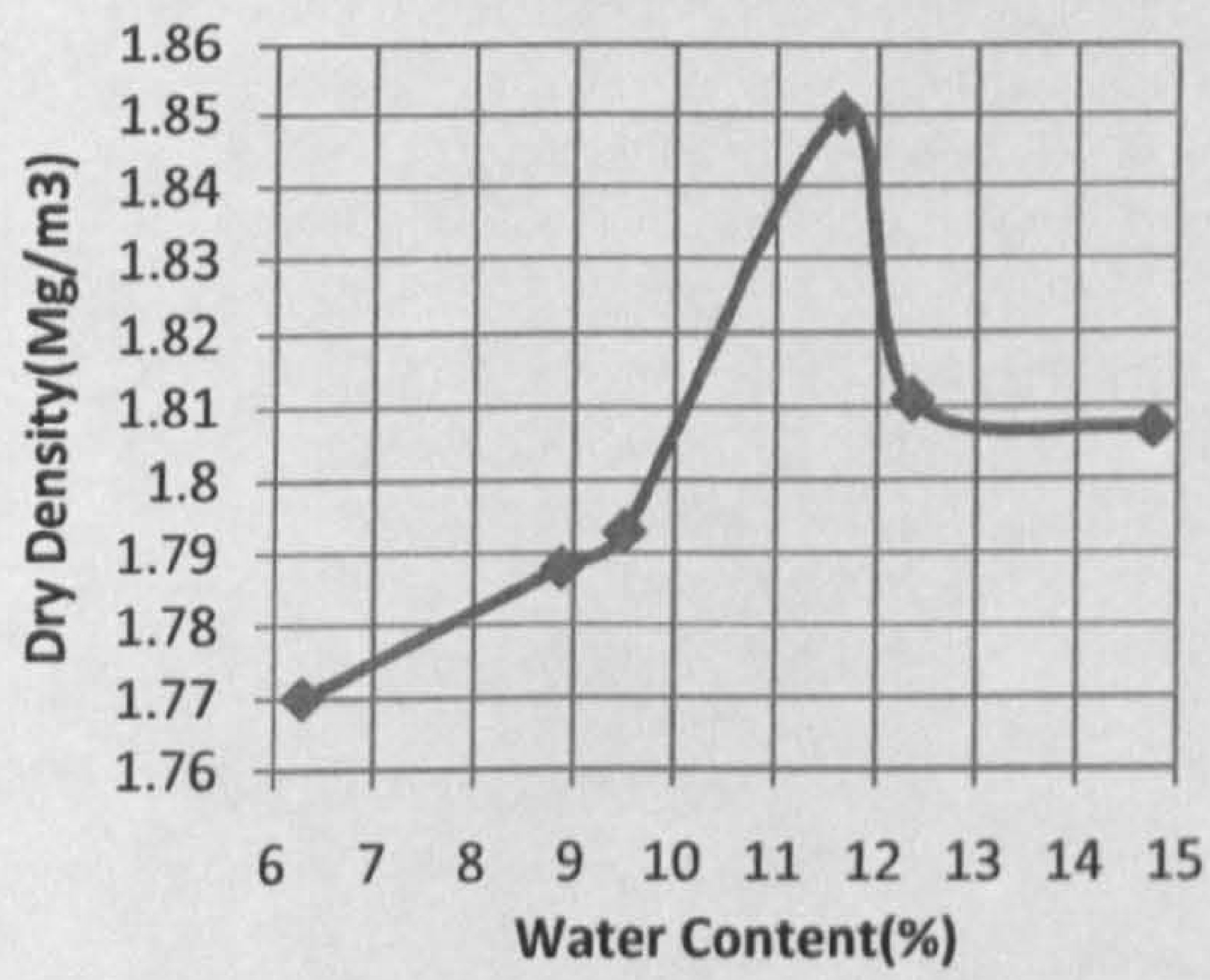


Figure 5.23 Dry densities in water content range of T-1 for determining CBR

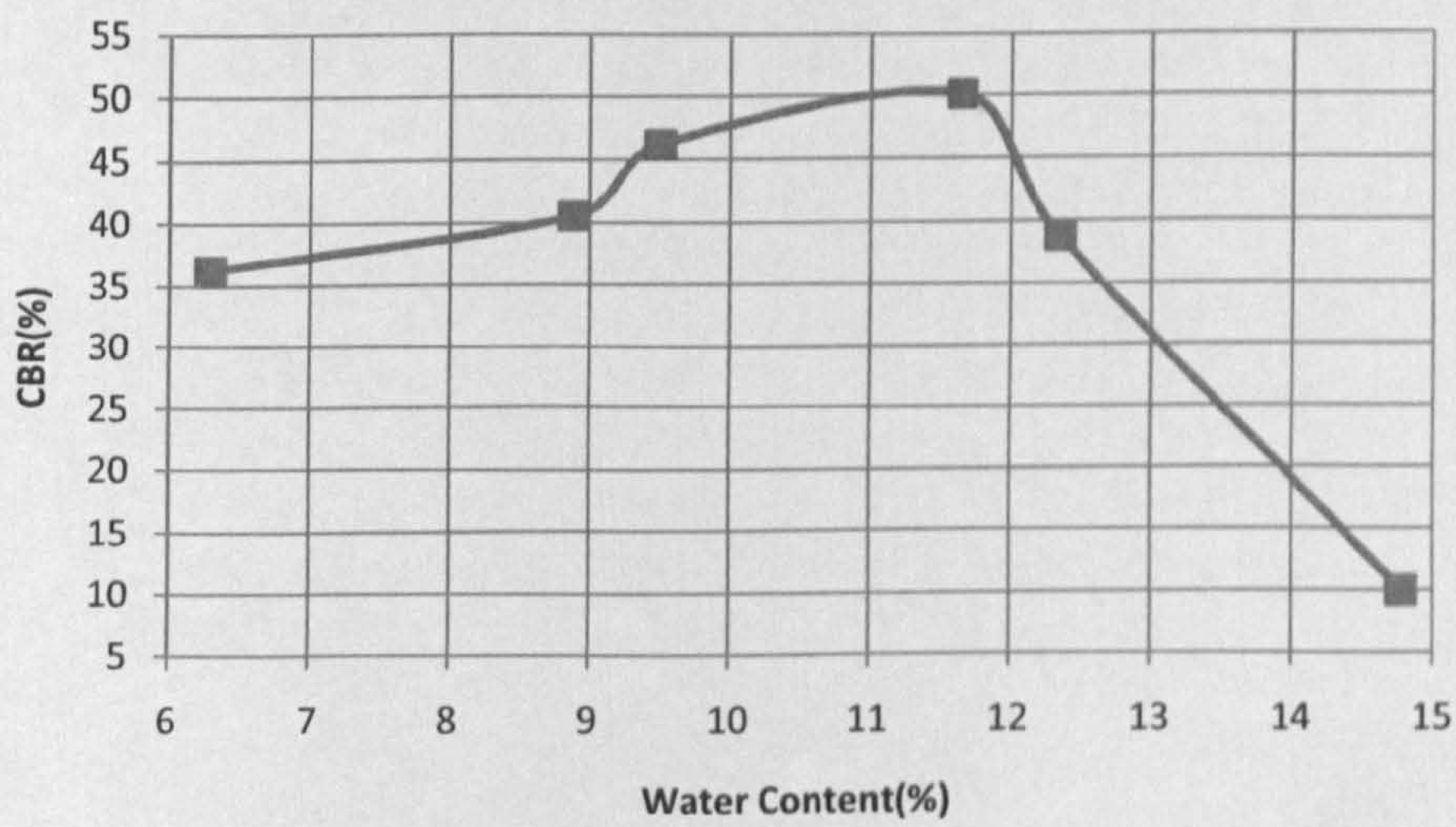


Figure 5.24 CBR in water content range of T-1

Figure 5.24 shows that the CBR of T-1 was more sensitive to moisture content changes in the wet side than in the dry side. The maximum CBR of 50% was reached at about 11.65% moisture content.

For material T-1 mix used for CBR tests, Figure 5.23 shows that the first trial moisture content of approximately 6.3% corresponded to a dry density was 1.77 Mg/m^3, which equates to 96% of the MDD of 1.84 Mg/m^3 observed from the compaction test sample of the same T-1 material (Fig. 5.19). Furthermore, the CBR corresponding to the 6.3% moisture content was found to be 36% (Fig. 5.24). This CBR result is already more than the minimum value of 30% recommended by HD 25/94 for granular subbase. Therefore, it is possible for material T-1 to meet the requirements of HD 25/94 even at moisture contents much lower than the OMC. The CBR increases to 50% at 100% compaction degree (Fig. 5.24).

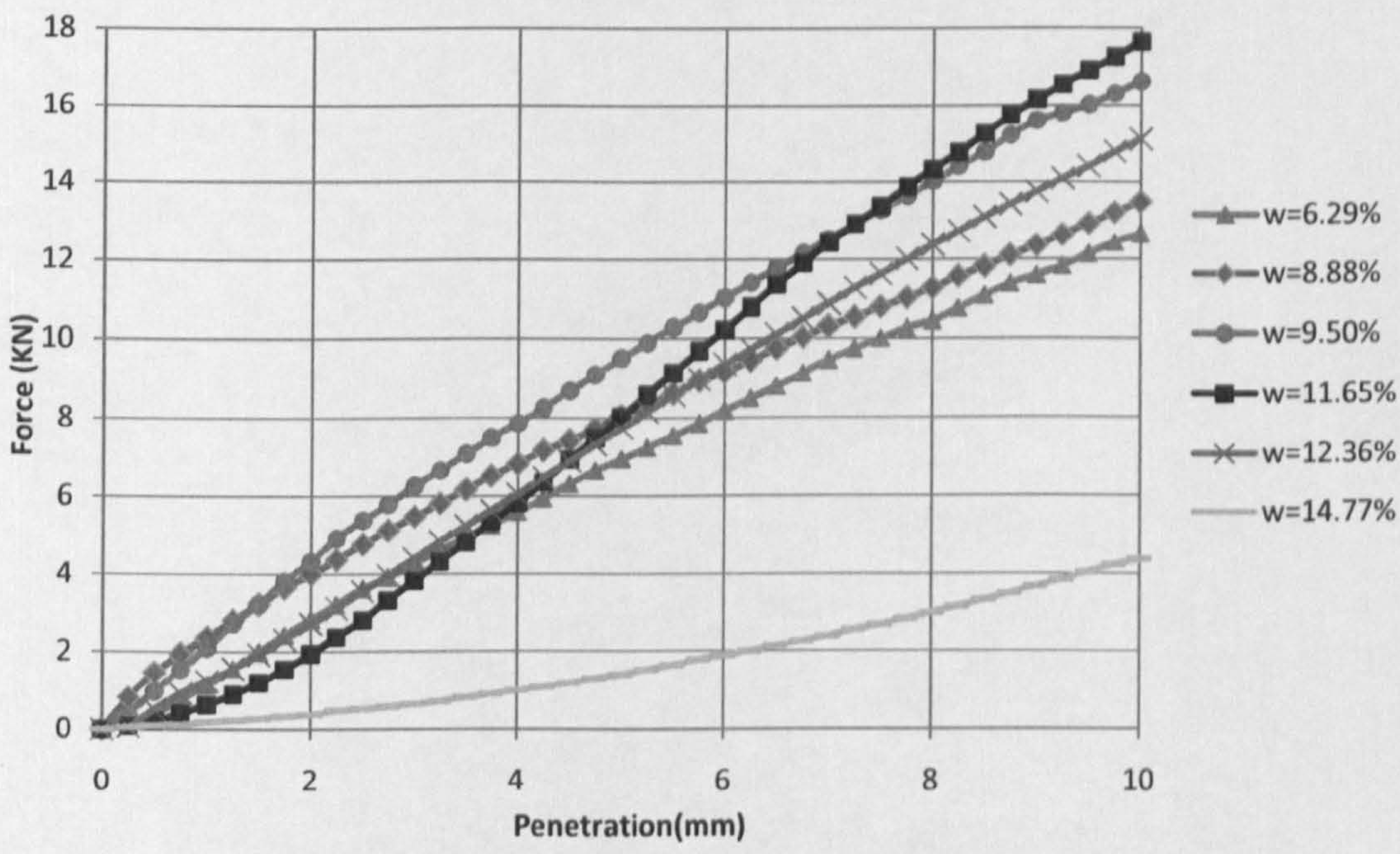


Figure 5.25 Force/penetration curves for T-1 (100% RCA)

Figure 5.25 shows the force-penetration curve in CBR tests for different moisture contents. Following usual correction procedures for CBR test graphs, when the early part of the graph curves upwards (concave) followed by a transition at a certain point to the normal convex shape, the curve is corrected by drawing a straight line backwards from the transition point to the horizontal axis. Applying this correction to the test curve for 11.65% moisture content gave the CBR of 50% mentioned earlier.

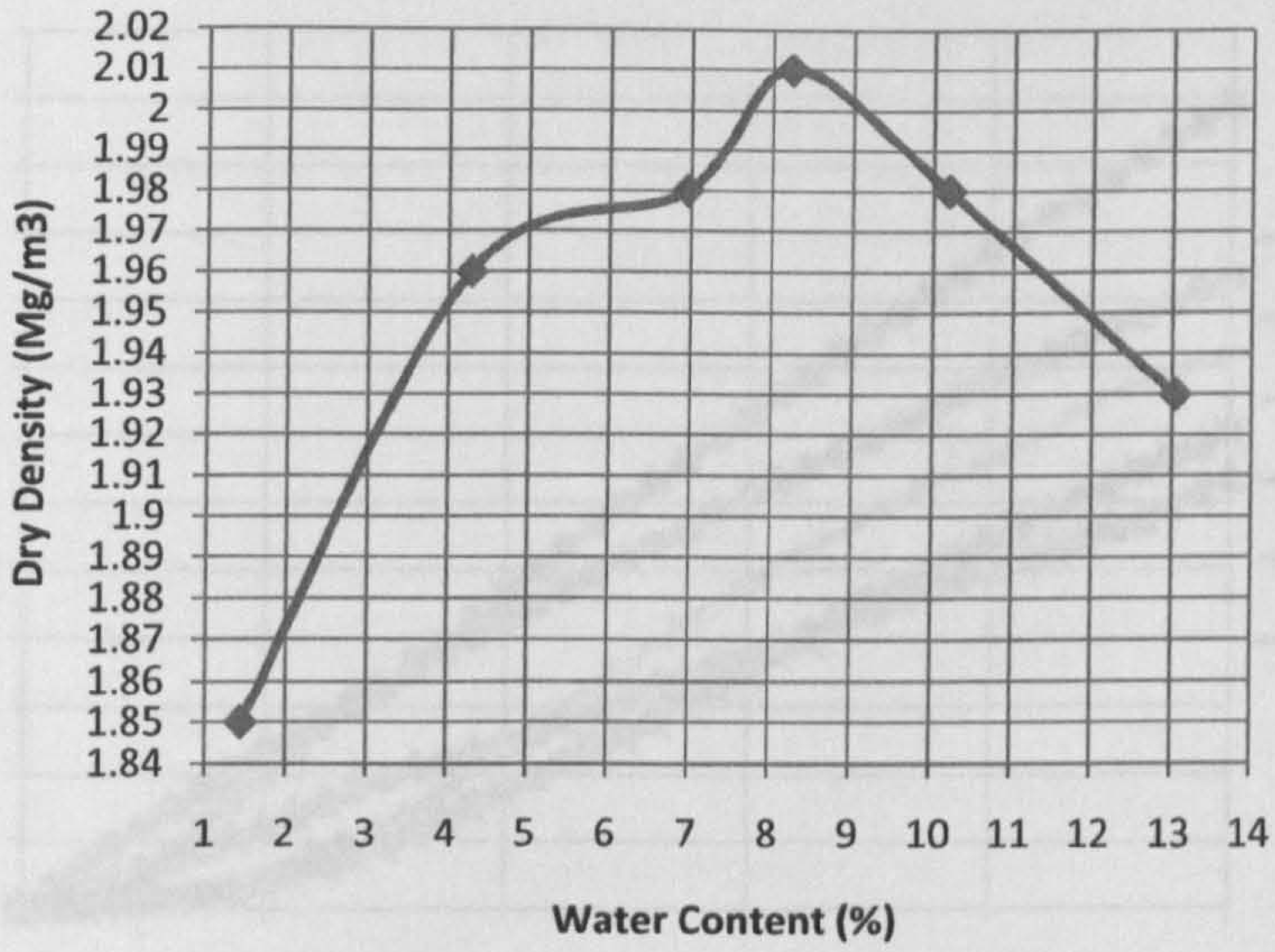


Figure 5.26 Dry densities in water content range of T-2 for determining CBR

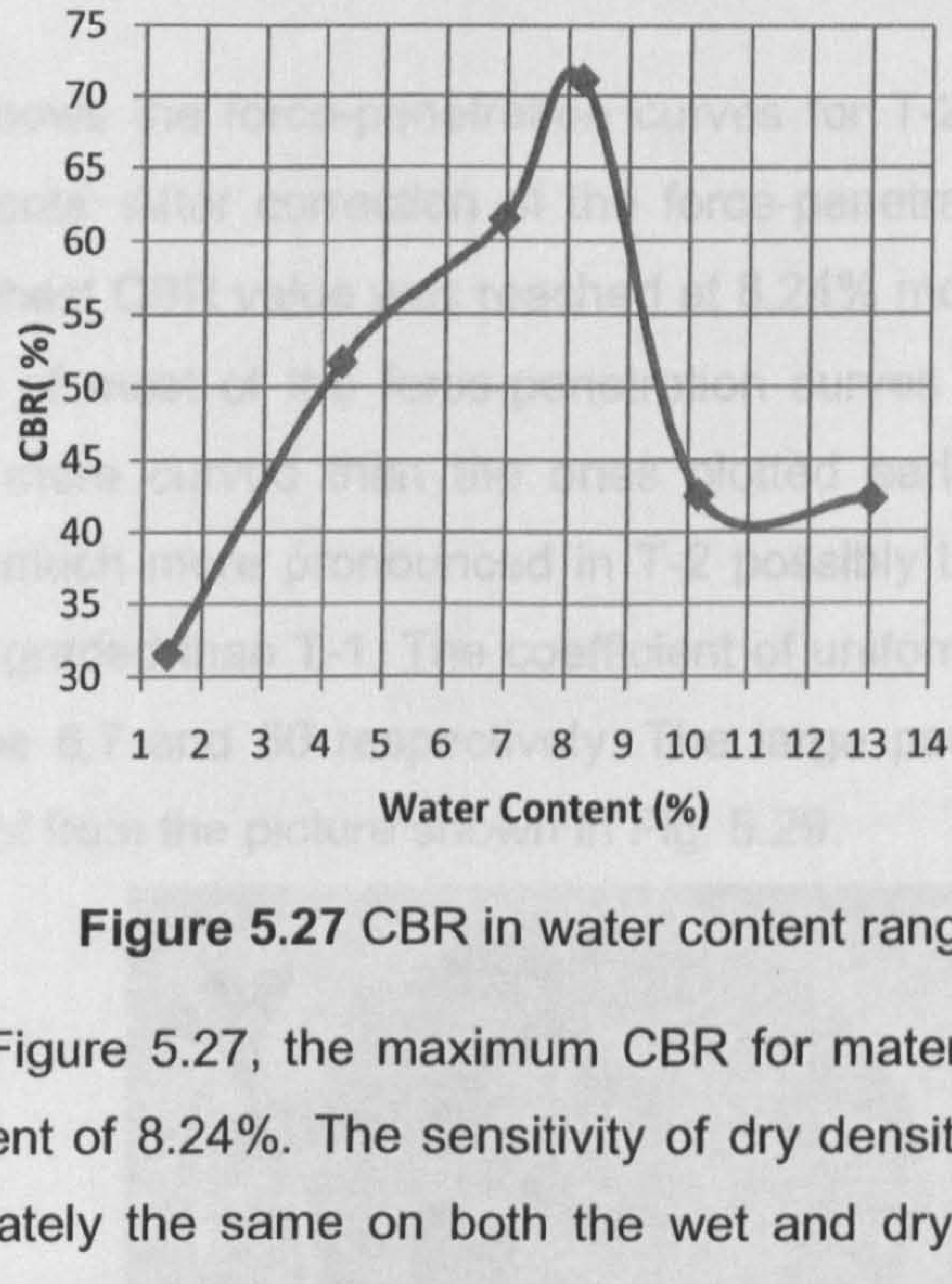


Figure 5.27 CBR in water content range of T-2

According to Figure 5.27, the maximum CBR for material T-2 was reached at a moisture content of 8.24%. The sensitivity of dry density of T-2 to moisture level was approximately the same on both the wet and dry sides of the compaction curve. For material T-2 mix used for CBR tests, Figure 5.25 shows that the first trial moisture content of approximately 1.5% corresponded to a dry density was 1.85 Mg/m^3, which equates to 91% of the MDD of 2.03 Mg/m^3 observed from the compaction test sample of the same T-2 material (Fig. 5.20). Furthermore, the CBR corresponding to the 1.5% moisture content was found to be 32% (Fig. 5.27). Thus, as was found for T-1, the CBR value for T-2 even at the low moisture level of 1.5% exceeds the minimum requirement (HD 25/94) of 30% for subbase layers.

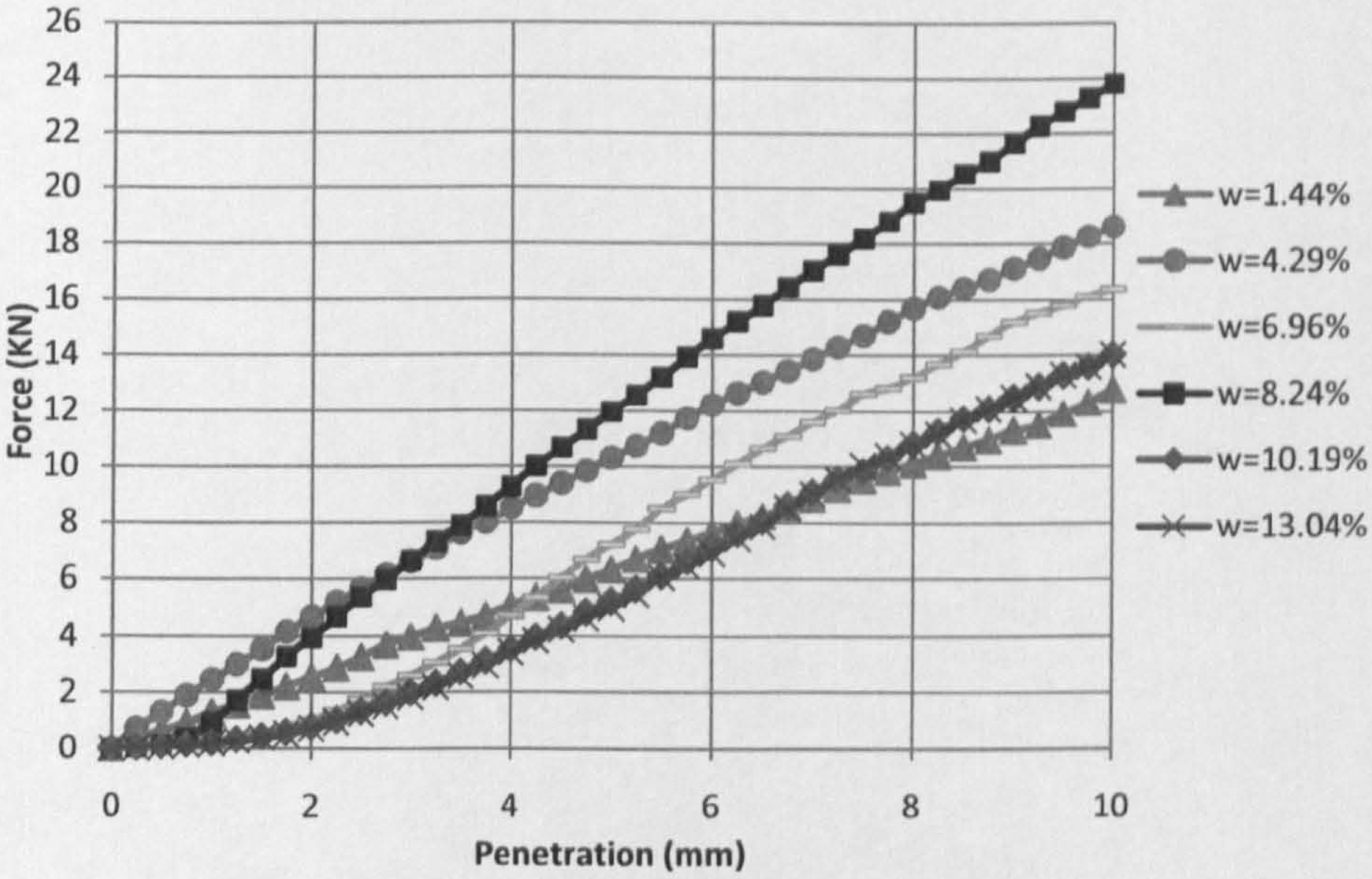


Figure 5.28 Force/penetration curves of 100% NA

Figure 5.28 shows the force-penetration curves for T-2 in CBR test for different moisture contents. After correction of the force-penetration curves as discussed earlier, the highest CBR value was reached at 8.24% moisture content. Clearly the initial portions of most of the force-penetration curves in Fig. 5.28 are concave upwards and more curved than the ones plotted earlier for T-1. This trend of behavior was much more pronounced in T-2 possibly because this material was less uniformly graded than T-1. The coefficient of uniformities of T-2 and T-1 were assessed to be 6.7 and 50 respectively. The large particle size range in T-2 is strongly evident from the picture shown in Fig. 5.29.

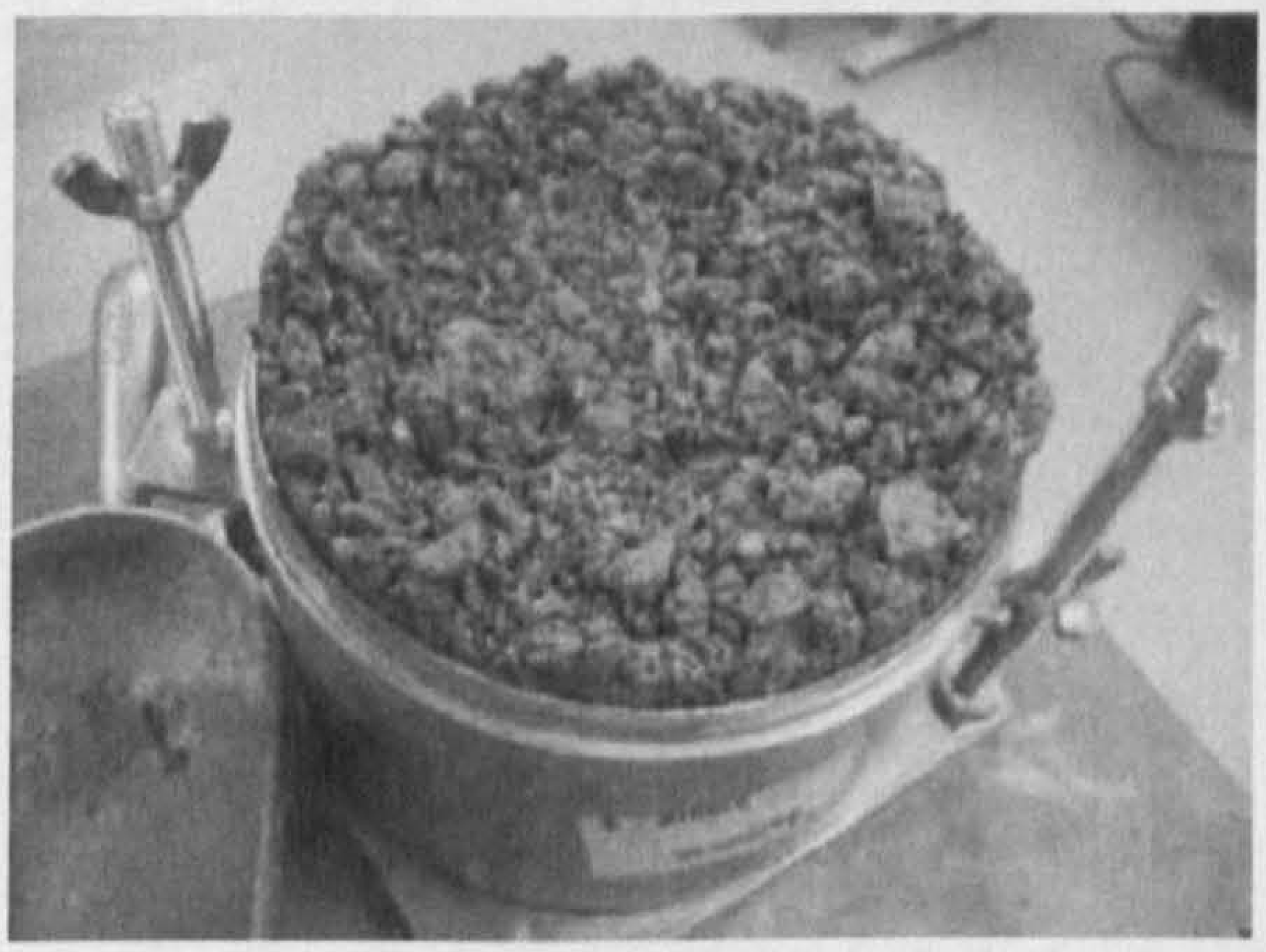


Figure 5.29 Apparent segregation irregularities in T-2 (100% NA) before CBR test

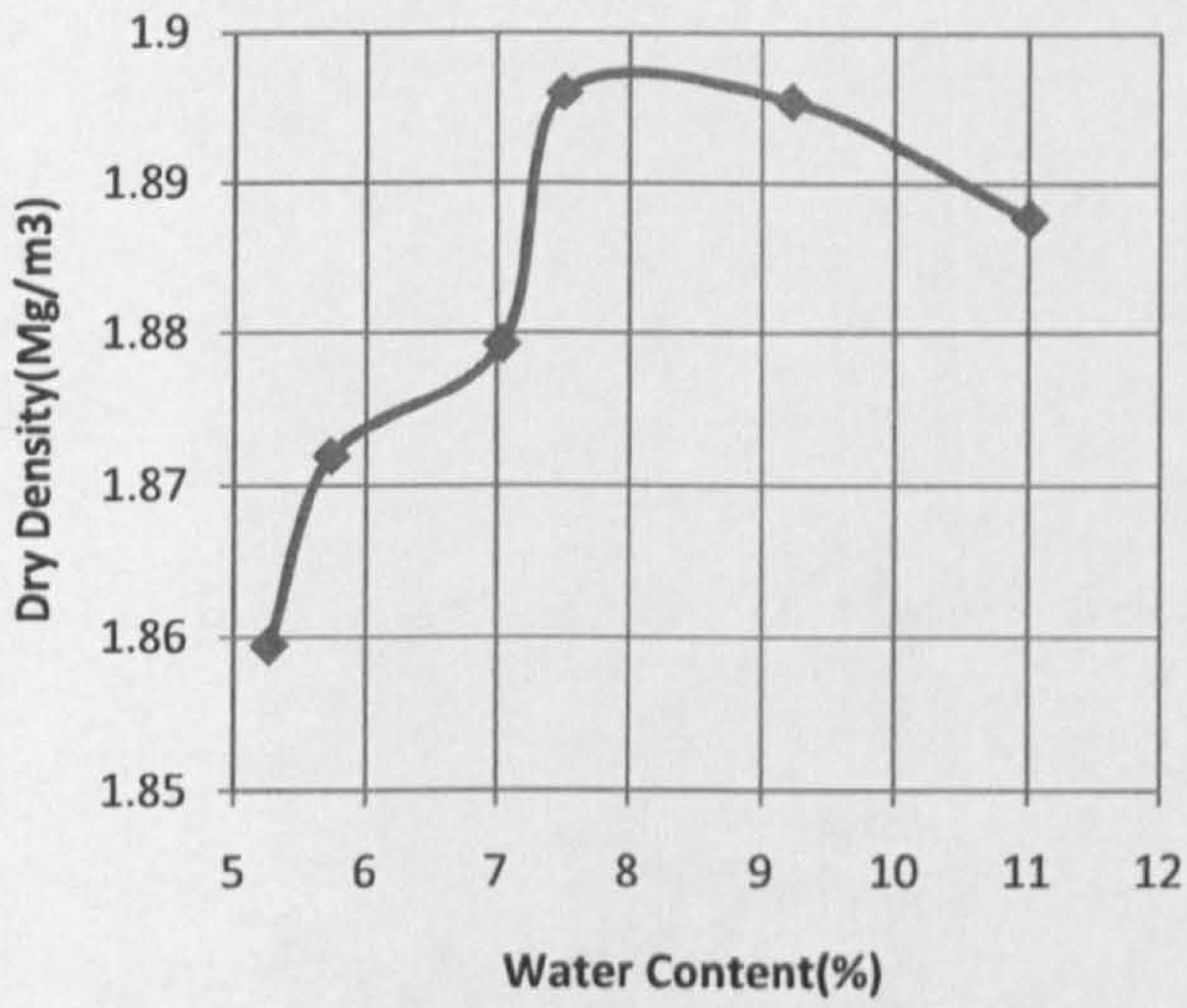


Figure 5.30 Dry densities in water content range of T-3 for determining CBR

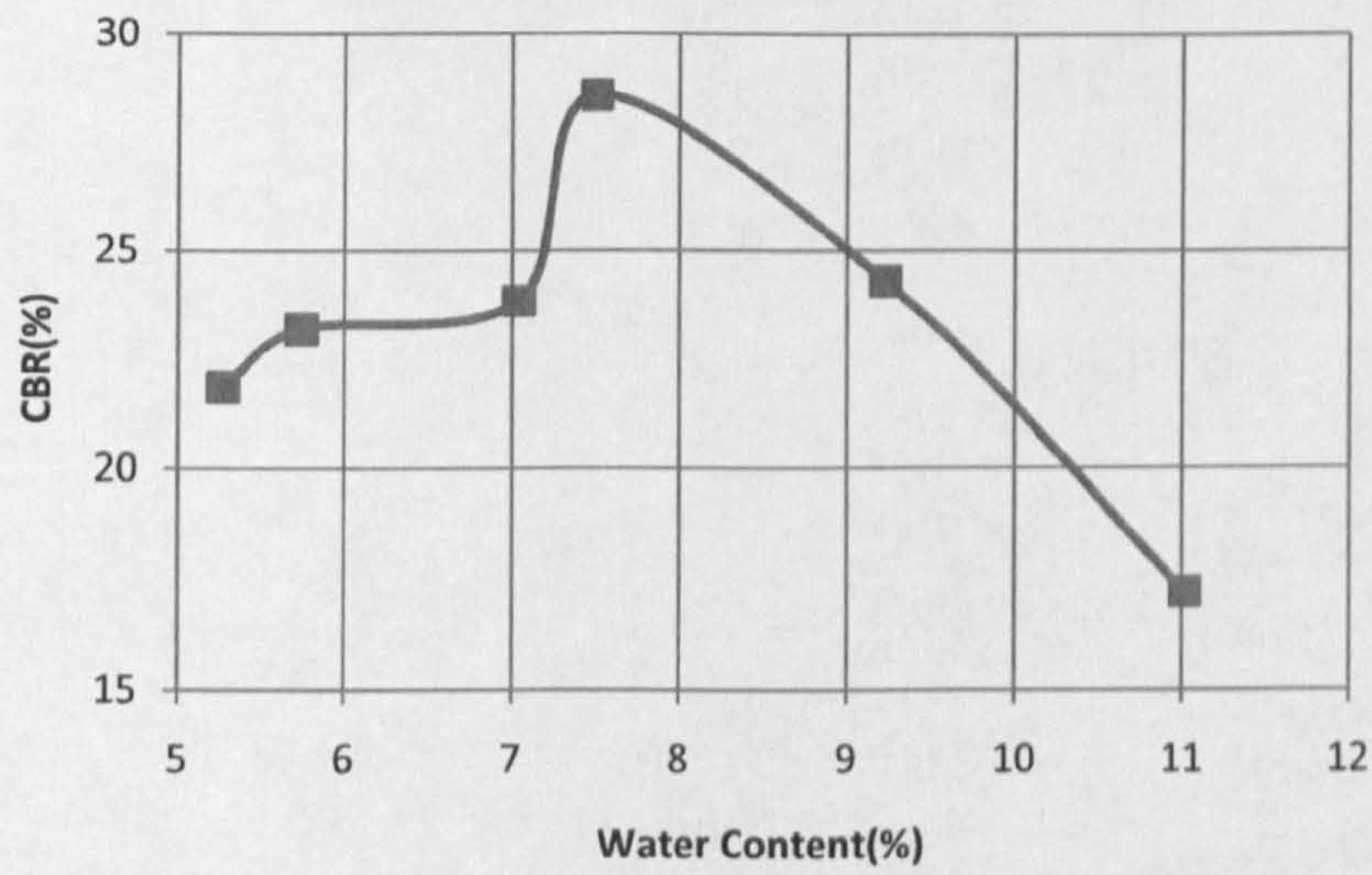


Figure 5.31 CBR in water content range of T-3

Turning now to material T-3, Fig. 5.31 shows that, like in T-1, the CBR of T-3 was much more responsive to moisture content variation on the wet side than on the dry side of the compaction curve. The maximum CBR of 28.59% was reached at about 7.51% water content.

The MDD for T-3 from compaction test samples was found to be approximately 1.89 Mg/m3 (Fig. 5.21). As for the T-3 samples taken for CBR tests, the apparent MDD as judged from Fig. 5.30 was slightly higher than the figure 1.89 Mg/m^3 reported above. In Fig. 5.31, it is seen that the CBR values for all moisture contents examined were less than 30%. Therefore material T-3 clearly did not meet the requirements of HD25/94 regarding a minimum CBR of 30% for subbase application. It is possible that with some modification in grading or mix proportions containing RCA and RAP, a material having CBR greater than 30% may be achieved. The CBR properties of mixes having different proportions of RAP and RCA will be examined later under the section entitled "Phase 2: Tests carried out in Iran".

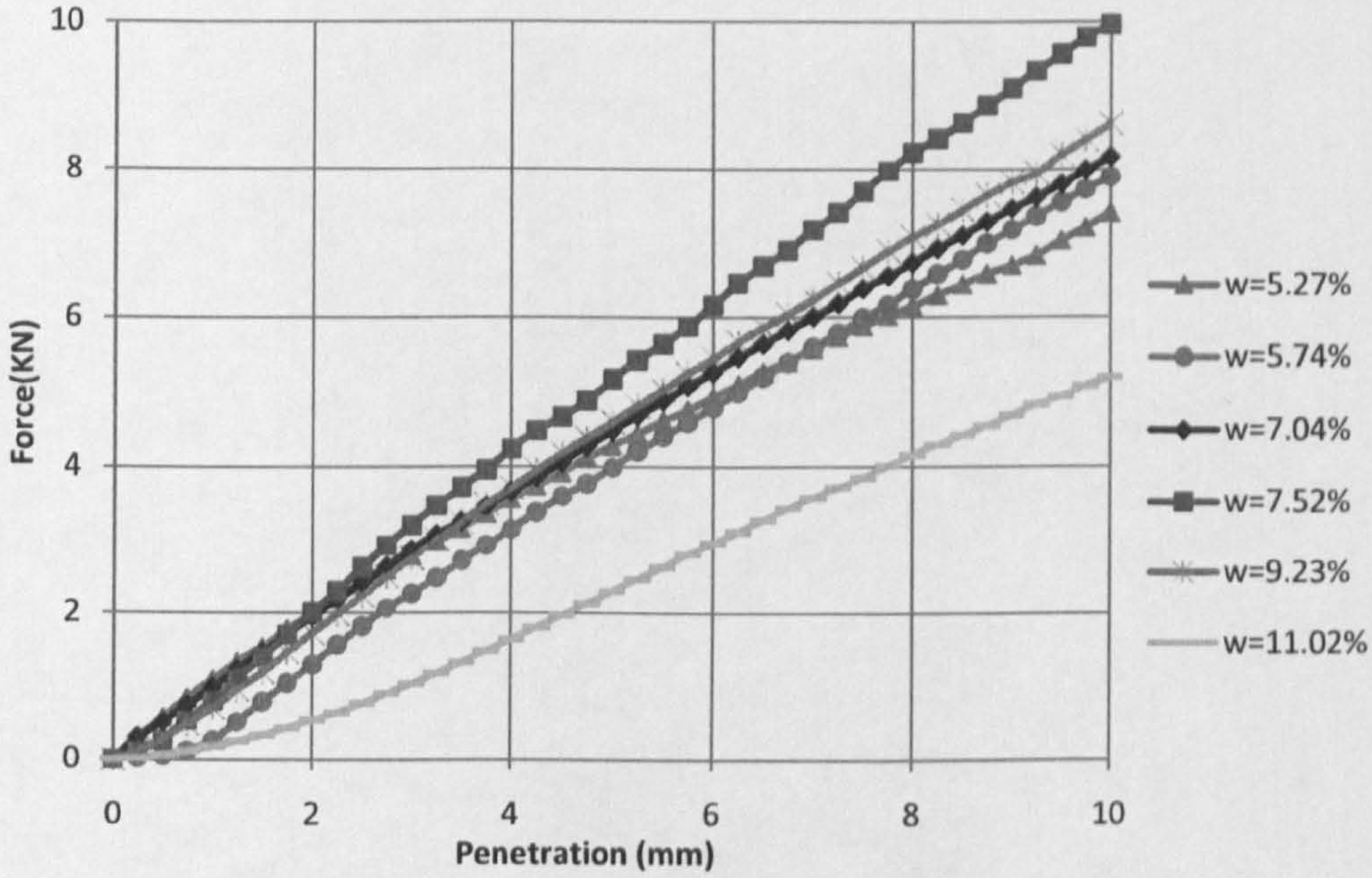


Figure 5.32 Force/penetration curves of 50%RCA+50%RAP

Figure 5.32 shows the force-penetration curves from CBR tests for different moisture contents in T-3. After applying corrections to the curves as described earlier, it was found that a moisture content of 7.52% gave the highest CBR of 29%.

In order to compare the behavior of T-1, T-2 and T-3, the individual curves in Figs. 5.23, 5.26 and 5.30 are plotted together as shown in Figure 5.33. The effect of the fine hydrated cement and unhydrated cement (which has a low density) in RCA is to decrease the density and increase the water absorption potential. It should be noted that the source of RCA has a great influence on the content of fine hydrated cement and the unhydrated cement, so that different behavior should be expected from different types of RCA. Furthermore, the process by which concrete is crushed to produce RCA may lead to particles of different sizes, shapes and surface texture thereby controlling the amount of voids within the material during compaction to determine the MDD. As was shown in Figs. 5.13 and 5.11 the water absorption of RCA was more than four times that of NA. The bulk density of RCA was found significantly lower than that of NA. The high porosity of RCA was responsible for the OMC and MDD in T-1 being greater and lower respectively than in NA.

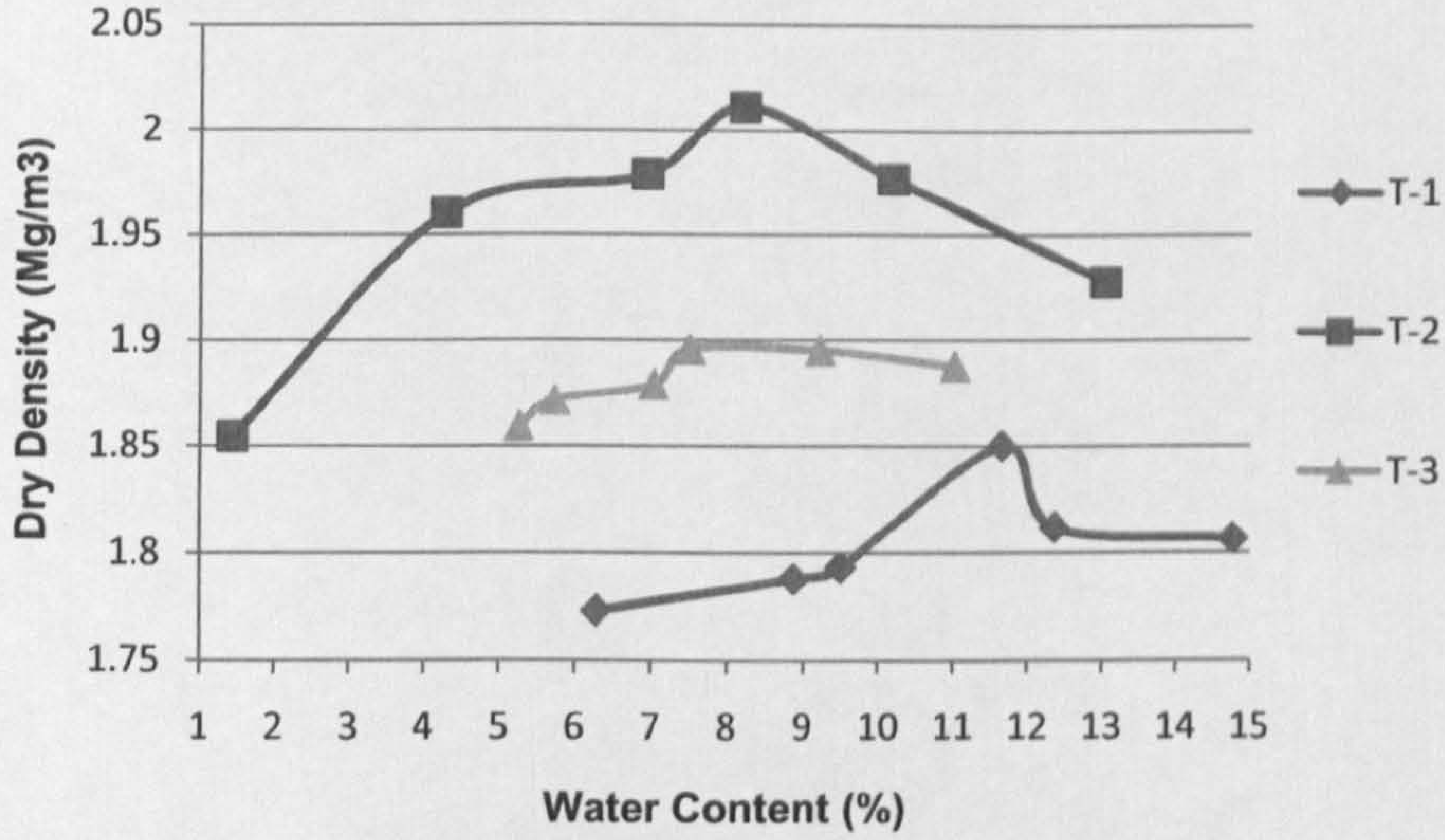


Figure 5.33 Moisture-density curves

In order to compare the behavior of T-1, T-2 and T-3, the individual curves in Figs. 5.24, 5.27 and 5.31 are plotted together as shown in Figure 5.34.

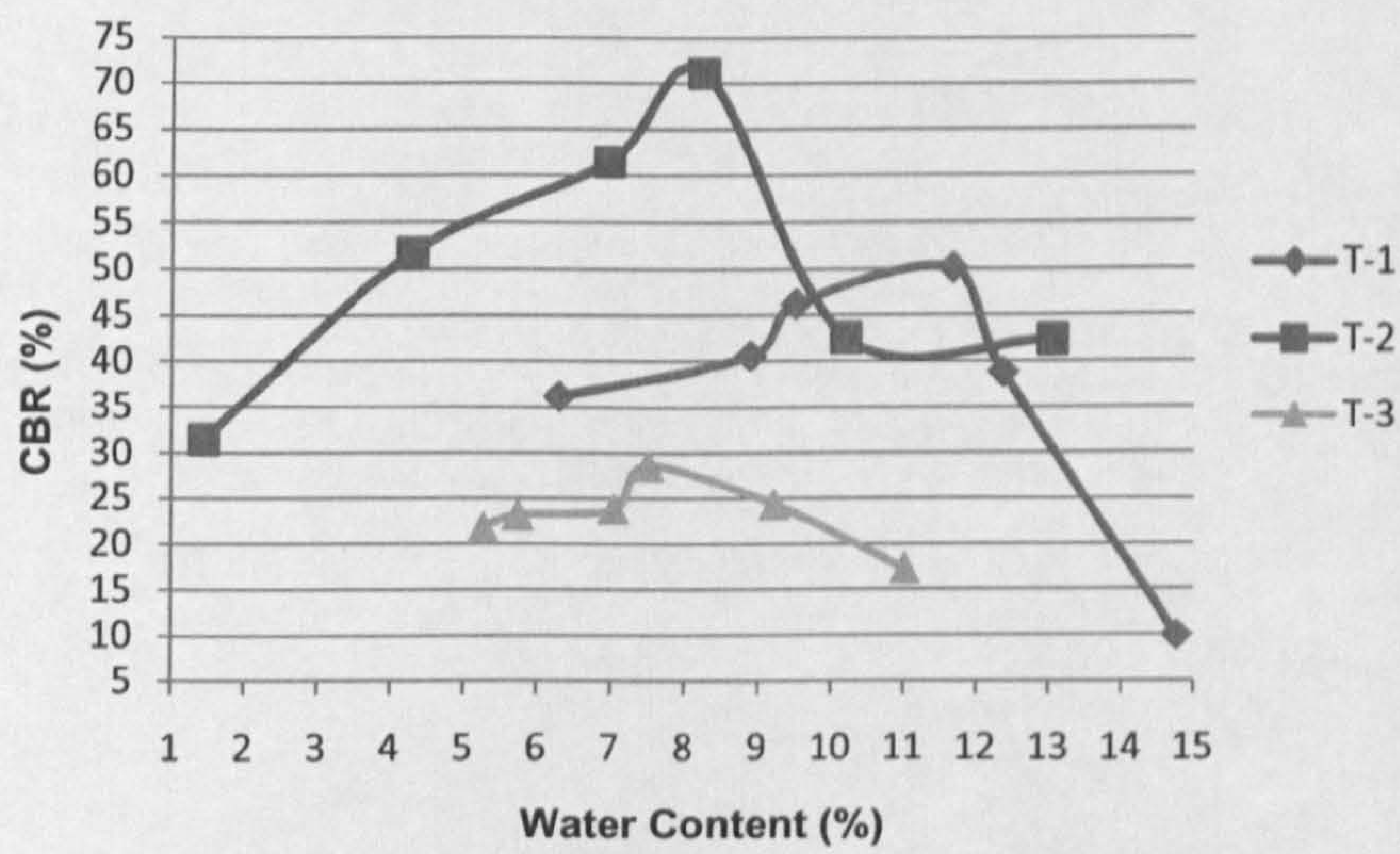


Figure 5.34 Moisture-CBR curves

Although Fig. 5.33 showed that material T-3 had higher dry densities than T-1 for the entire moisture content range examined, the same material displayed lower CBR values than T-1 (Fig. 5.34). Although the grading curves in Fig. 5.1 showed that T-3 had a greater uniformity coefficient than T-1, the material (T-3) was better graded than T-1 as evident from the curvature of the graphs and also from Fig. 5.35. The better grading of T-3 enhanced interlocking of particles thereby increasing the dry density. However, the hardness of RCA stone particles meant increased resistance to plunger penetration in the CBR test consequently producing higher CBR values for T-1.

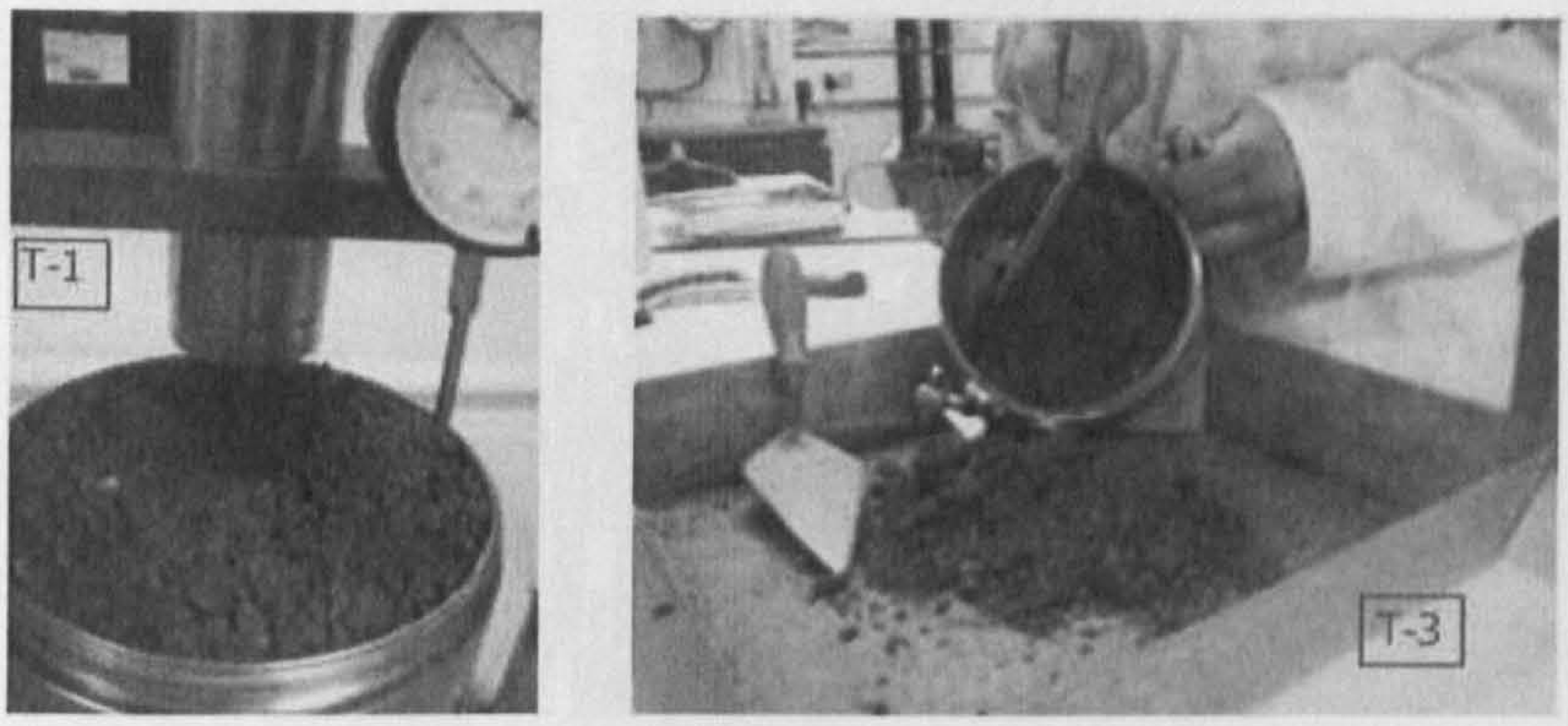


Figure 5.35 CBR samples of T-1 and T-3

Soaked CBR values for T-1, T-2 and T-3 for 96 hours soaking period were found to be 31%, 42% and 25%, respectively.

5.8 Determination of Aggregates Crushing Value (ACV)

5.8.1 Method Statement and specifications

Part 110 of BS 812 (1990) was used for the determination of the aggregate crushing value (ACV) which gave a relative measure of the resistance of an aggregate to crushing under a gradually applied compressive load. This method was applied to aggregates passing a 14.0mm test sieve and retained on a 10.0mm test sieve. Two test specimens from each of T-1, T-2 and T-3 materials were compacted in a standardized manner in a steel cylinder fitted with a freely moving plunger. Test specimens were added in three layers of approximately equal thicknesses. Using a standard tamping rod dropped freely over a height of 50 mm, each layer was compacted with 25 evenly distributed blows over the surface of the layer. According to the standard procedure in the ACV test, each compacted specimen was subjected to a 400 kN force applied at a uniform rate for a period of 10min. After sieving the tested material through a 2.36 mm sieve, the ACV was then calculated as the ratio of the mass passing to the total mass of the material sieved. The mean of two ACV for each material was recorded as ACV of that type, because individual results will less than 0.07 times the mean value (BS 812-110).

5.8.2 Test results

The ACV results were obtained to be 24%, 21% and 16% for T-1, T-2 and T-3, respectively. Therefore, as anticipated, T-1 (100%RCA) crushed the most under the load while T-3 (50%RCA+50%RAP) crushed the least. On transferring the tested materials T-1 (cement-bound) and T-3 (cement and bitumen-bound), into a tray for inspection, the particles of both materials were cemented together but this was more evident in T-3 than in T-1 (Figures 5.36 to 5.38). Furthermore, since the RCA particles in T-1 were initially not very strongly interlocked, they were weakened more in the ACV test (Figure 5.36).

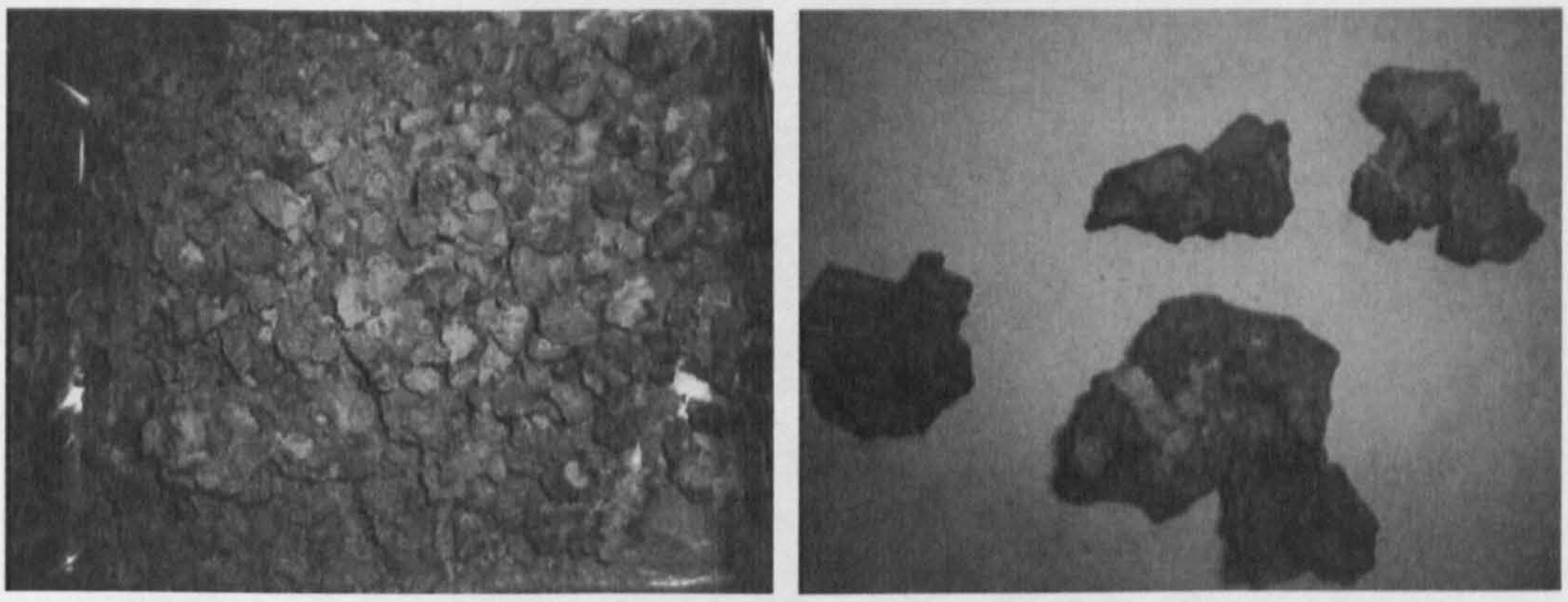


Figure 5.36 T-1 after crushing

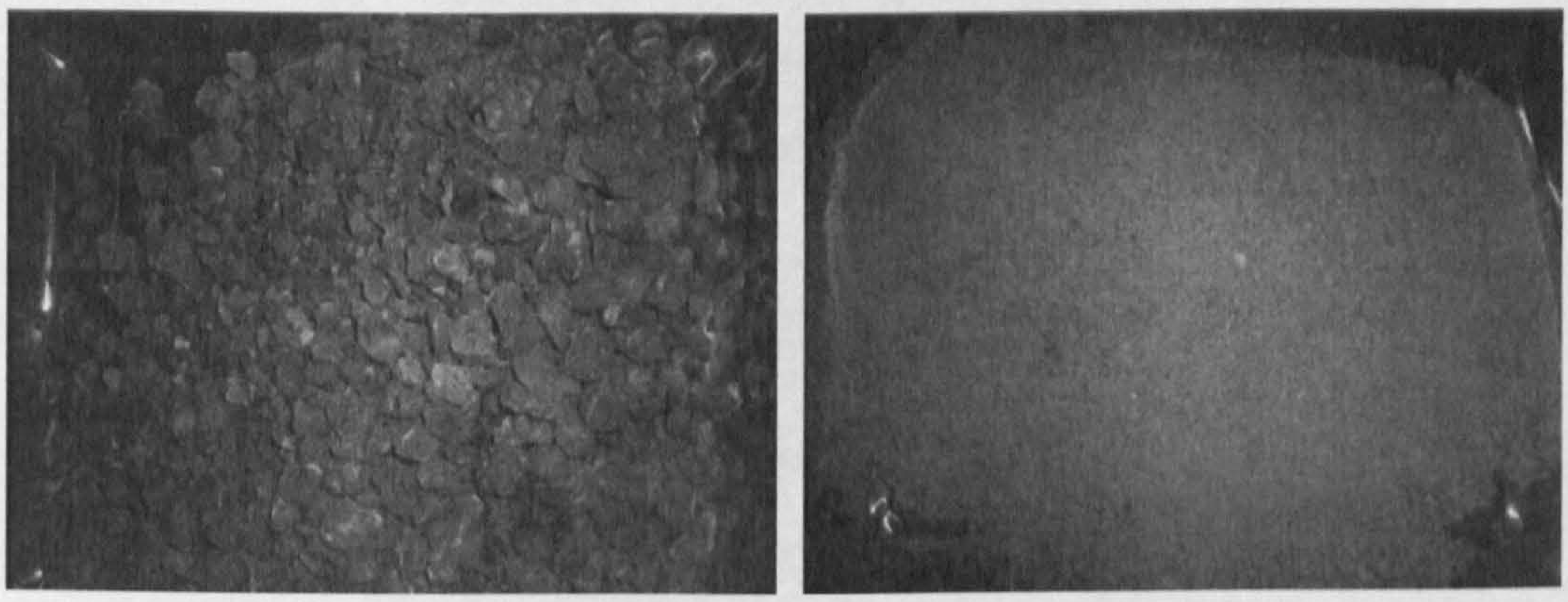


Figure 5.37 Coarses and fines fraction of T-2 after crushing and sieving

Figure 5.38 Bitumen attached fractions of T-3 on the 2.36 mm test sieve

5.9 Determination of Aggregates Impact Value (AIV)

5.9.1 Method Statement and specifications

The aggregate impact value (AIV) of aggregates is a measure of the resistance of aggregate to impact or sudden shock. This test was performed based upon Part 112 of BS 812 and applied to aggregates passing the 14.0mm test sieve but retained on the 10.0mm test sieve. To measure the AIV of materials T-1, T-2 and T-3, they were subjected to 15 blows of a standard hammer falling 380 mm height. The resulting disintegration of the materials was measured in terms of the quantity of material passing the 2.36 mm sieve, which was then expressed as a percentage of the mass of the tested sample to give the AIV. For each material, an average AIV was found from two tests as AIV of that type, because individual results were less than 0.15 times the mean value (BS 812-112).

5.9.2 Test results

The AIV results were found to be 28%, 19% and 16% for T-1, T-2 and T-3, respectively. It is seen that the AIV results for these materials mirror the previously discussed ACV results in that T-1 is the weakest material and T-3 is the strongest one.

5.10 Determination of Shear Strength, Effective Angle of Internal Friction and Effective Cohesion of Aggregates

5.10.1 Method Statement and specifications

The effective angle of internal friction ϕ' and the effective cohesion c' of the materials T-1, T-2 and T-3 were measured using large shear box equipment as recommended by MCHW Series 600 (2009) and BS 1377-7 (1990). The shear box was 300mm square in plan dimensions by 150mm in height (Figure 5.39). Test specimens were taken from materials passing the 20 mm sieve and three samples from each of T-1, T-2 and T-3 were tested. Each sample was remoulded and compacted at the OMC to a dry density of 92%±2% of the MDD determined earlier in Section 5.6. Each of the samples was subjected to three normal stresses 50kPa, 100kPa and 200kPa simulating the approximate in-service load at the mid-height, quarter height and bottom of the subbase layer, respectively. The constant rates of shearing in controlled strain were chosen as 2.7 mm/min for T-2 and 1mm/min for both T-1 and T-3. At these rapid loading rates, and given the particulate nature of the materials pore water pressure accumulation was negligible. Appendix A gives shear stress, horizontal displacement and height change under any normal stress for any kind of mixes.

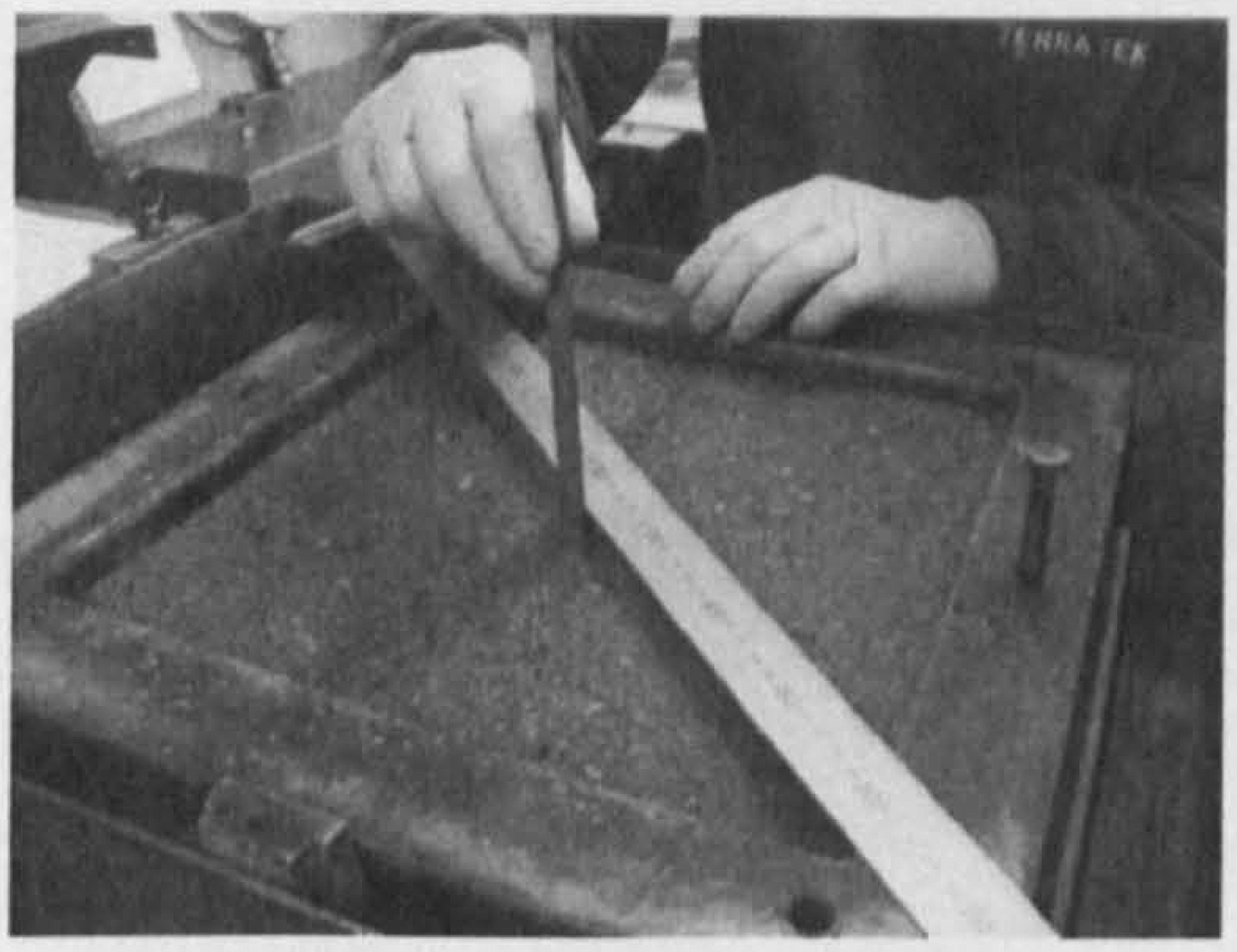


Figure 5.39 Controlling the level of compacted sample in shear box test

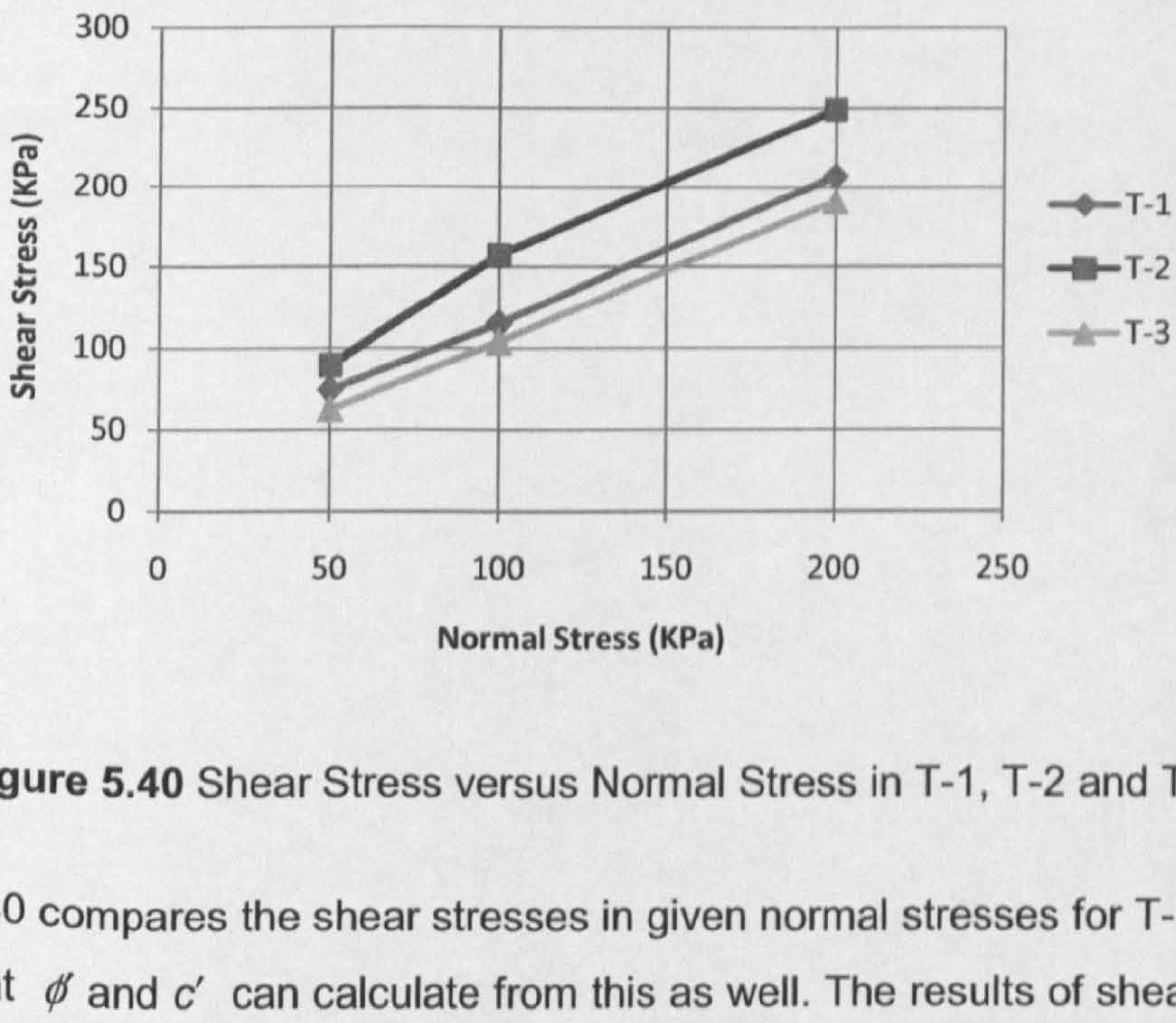


Figure 5.40 Shear Stress versus Normal Stress in T-1, T-2 and T-3

Figure 5.40 compares the shear stresses in given normal stresses for T-1, T-2 and T-3 so that ϕ' and c' can calculate from this as well. The results of shear strength test showed that T-1 has the most ϕ' and c' and T-3 has the least ones (Figure 5.40 and Table 5.4). As is seen the presence of RCA in T-3 has increased flakiness index (Section 5.1) and as a following result has effected on shear characteristics of T-3.

Table 5.4 Shear characteristics of aggregates

Aggregate	Effective Angle of Internal Fraction (degree)	Effective Cohesion (Kpa)
T-1	41.5	28
T-2	46	44
T-3	40.5	20

The state of packing of the grains in T-1 is represented diagrammatically in Figure 5.41. If the limestone particles were sheared along a plane such as X-X, and if it is assumed that fragmentation and crushing of individual particles does not happen, grains lying just above the surface X-X will be forced to ride up and over those lying just below when relative movement occurs. This behavior can be used to explain the resulting increase in volume for T-1 and T-2 materials during the initial stages of shearing. The small initial contraction in first steps of horizontal displacements is due to some bedding down of particles when shearing begins (Appendix A). It is observed (Appendix A) that the shear stress curves rise quite sharply to a peak and then slightly start to fall off to a somewhat lower value (Appendix A).

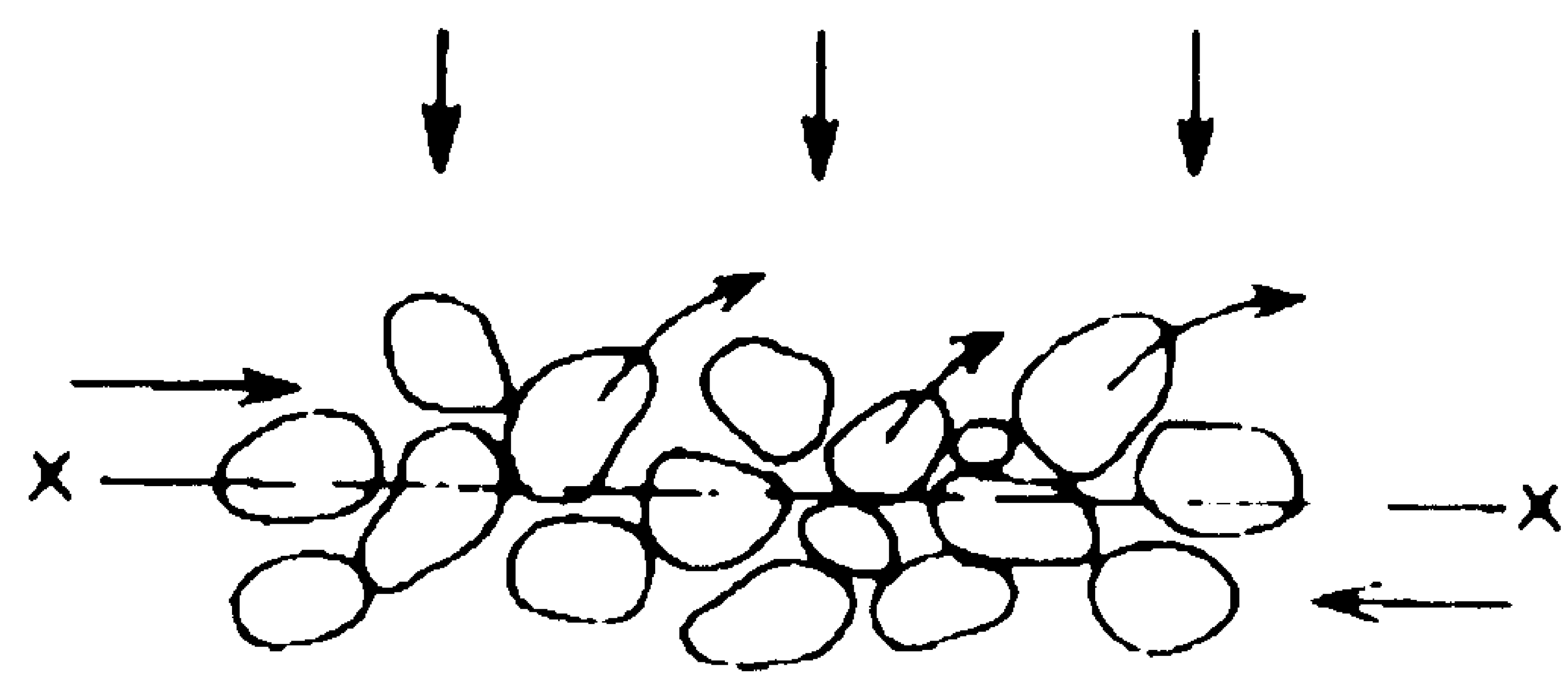


Figure 5.41 Effect of shear on grain structure in T-1 and T-2 (From Head, K.H. Vol.2, p.205)

The state of packing of the particles in T-3 is indicated diagrammatically in Figure 5.42. Shearing along a plane such as YY will result in a collapse of the relatively poor interlocked structure between RCA and RAP particles, so that the grains will move downwards into void spaces. This causes a volume or height changes decrease which have been shown in (Appendix A). The resulting shear stress and horizontal displacement curves for T-3 are less steep than those of T-1 and T-2.

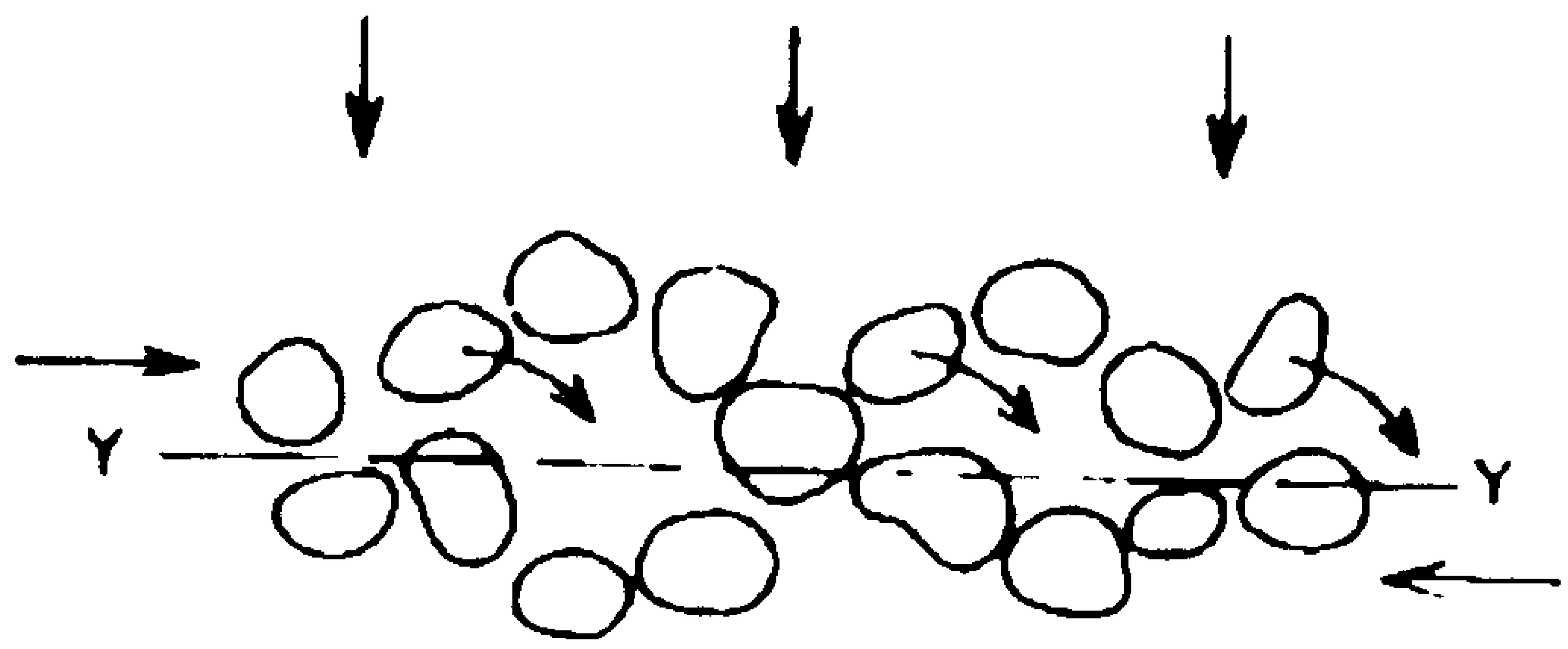


Figure 5.42 Effect of shear on grain structure in T-3 (From Head, K.H. Vol.2, p.205)

5.11 Determination of the Resistance to Wear (Micro-Deval)

This test was carried out to measure the resistance of aggregates to wear in accordance with BS EN 1097-1:1996. Micro-Deval test was carried out on two fractions of aggregates in the wet condition using a rotating drum with steel spheres. This type of apparatus is essentially the same as those used in the Los Angeles abrasion test, only that there are variations in the sizes of the drums and steel spheres. The final test portion had a mass of 500g including two fractions below:

- Size fraction 1: passing the 14mm test sieve and retained the 12.5mm test sieve.
- Size fraction 2: passing the 12.5mm test sieve and retained the 10mm test sieve.

On each aggregates two specimens were tested and the mean value of micro-Deval coefficients of these specimens were calculated (BS EN 1097-1). This coefficient was calculated from the equation which included the quantity of aggregates retained on 1.6mm sieve after rolling (Equation 5.3).

$$M_{DE} = \frac{500-m}{5} \qquad \textbf{(Equation 5.3)}$$

Where:

M_{DE} is the micro-Deval coefficient

m is the mass of the oversize fraction retained on a 1.6 mm sieve, in grams.

A lower value of this coefficient presents a better resistance. The results of micro-Deval test tend to polish aggregate particles while the L.A. abrasion tests tend to break them. Clearly, the micro-Deval coefficients show the resistance to wear but

Los Angeles coefficients show the resistance to fragmentation. Micro-Deval (M_{DE}) coefficient for T-1, T-2 and T-3 were obtained as 23, 18 and 18.6, respectively.

5.12 Determination of the Resistance to Fragmentation (Los Angeles Test)

The Los Angeles abrasion test was carried out to measure the resistance of aggregates to fragmentation in accordance to BS EN 1097-2. The test was carried out on two fractions of aggregates through rolling them with steel balls in a rotating drum. The final test portion had a mass of 5kg including two fractions below:

- Size fraction 1: passing the 14mm test sieve and retaining the 12.5mm test sieve.
- Size fraction 2: passing the 12.5mm test sieve and retaining the 10mm test sieve.

The Los Angeles coefficient was calculated from the equation which included the quantity of aggregates retained on 1.6mm sieve after rolling. A lower value of this coefficient presents a better resistance (Equation 5.4).

$$LA = \frac{5000-m}{5} \qquad \textbf{(Equation 5.4)}$$

LA is the Los Angeles coefficient

m is the mass of the oversize fraction retained on a 1.6 mm sieve, in grams.

Los Angeles (LA) coefficient for T-1, T-2 and T-3 were obtained 33, 21, and 22.4, respectively. Therefore, as found here, all the three types of aggregates met the requirements of MCHW Series 800 for Los Angeles test with the specification of LA_{50}.

5.13 Determination of the Soundness with Magnesium Sulphate

The soundness of aggregates with magnesium sulphate obtained in accordance with BS EN 1367-2: 1998. Two laboratory samples from each type of aggregates in the size range 10mm to 14mm were subjected to five cycles of immersion in a saturated solution of magnesium sulphate. After immersion for (17 ±0.5) h, aggregates were drained for (2±0.25) h and were dried at (110± 5) °C for (24 ± 1) h. After completing the five cycles the means of the magnesium sulphate value (MS) in percentage by mass were calculated for two samples of each type of aggregates from Equation 5.4a.

$$MS = \frac{100(M_1 - M_2)}{M_1}$$ **(Equation 5.4a)**

M_1 is mass of the test specimen and M_2 is the final mass of aggregate retained on the 10 mm sieve, in grams.

The results of MS for T-1, T-2 and T-3 were 7%, 3% and 6% respectively. It should be noted that the Magnesium sulphate soundness test based on BS EN 1367-2: 1998 utilises representative grading while under the AASHTO method, the same test is done for the whole grading range. As such, the author is of the opinion that the results from the AASHTO procedure has more merit than those from the BS EN 1367-2:1998 procedure. It was surprising to find that, despite the presence of RAP in material T-3, the MS soundness test result was only 1% less than that of T-1. It is probable that the selective manner in which the MS soundness test is done according to BS EN 1367-2:1998 may mean that the results show significant variability owing to effects such as particle sizes, shapes and nature of the material. Indeed, testing experience by Surrey County Council (UK) with materials from different sources supported the explanations suggested by the author. In general, all the three kinds of aggregates were found to fulfill the requirements of MCHW Clause 801 which limits the MS soundness of unbound aggregates to less than or equal to 35%.

5.14 Determination of the Frost Heave

Frost-heave can happen during extended periods of freezing when the temperature of the road pavement at lower layers falls below 0°C. If other conditions allow, water will be drawn from the water-table into the freezing area and may lead to the formation of ice-lens. During formation of ice-lens, further water may be drawn into the freezing area which can result in the formation more ice and considerable expansion. This expansion will manifest itself as heaving at the surface. After thawing, the water produced will act as the excess moisture which weakens the foundation of the road. Passing the traffic during the period of thaw can cause the whole pavement structure to fail (Smith and Collis, 2001). Frost heave test was performed according to BS 812-124: 2009 and simulated in self-refrigerated unit (SRU). Cylindrical specimens of each aggregate compacted at a predetermined OMC and MDD in Section 5.6 were placed in SRU. The upper surface of the specimens in SRU was subjected to freezing air at -17 °C whilst their lower ends were allowed access to water maintained at +4 °C. This temperature gradient created through the specimens cause water to be drawn into the freezing area and may result the formation of ice-lens. These cause an increase in height of the specimens which was measured at intervals over a period of 96 hours. The frost heave values of T-1, T-2 and T-3 were found to be 10.7mm, 9.0mm and 8.7mm, respectively. It is seen that these aggregates can be classified as non-frost susceptible according to MCHW Series 800, because the mean heave of them is less than 15mm. For RAP aggregate particles, the remaining adhered binder may make them more resistant to freezing and thawing because this binder prevents water penetration.

PHASE 2: TESTS CARRIED OUT IN IRAN

The second stage of the test programme was carried out in Geotechnical laboratories of the Municipality of Tehran in Iran. According to section 4.10.2.4 the tested materials included blends of Natural Aggregates (NA) with Recycled Concrete Aggregates (RCA) and Reclaimed Asphalt Pavement (RAP) with Recycled Concrete Aggregates (RCA). Individual aggregates such as 100% NA, 100% RCA and 100% RAP were studied as comparative measure, as well. This phase of research was divided into two sections:

- Mixes of Natural Aggregates (NA) with Recycled Concrete Aggregates (RCA)
- Mixes of Reclaimed Asphalt Pavement (RAP) with Recycled Concrete Aggregates (RCA)

5.15 Current Specifications and Mixture Requirements

There are two main documents governing highway works in Iran: (i) Iran Highway Asphalt Paving Code (2003) and (ii) Road General Technical Specifications (2003). These documents provided the basis of testing in phase 2 of this research. The framework of these codes is based on AASHTO and ASTM. The chapter dealing with subbases recommends five grading types which all must meet the criteria set out in Table 5.5. There is much experience in the use of RAP as base and subbase material in Iran and extensive studies in these materials have been performed. However, the use of RCA in Iran is only in its early stages so far. For this reason, there is as yet no special code relevant to the application of RCA. Therefore, for this research, AASHTO specifications which cover the use of RCA as an unbound granular subbase material were used.

Table 5.5 Requirements for subbases

Properties	Requirements	Test method	
		AASHTO	ASTM
Plasticity index	Max 6%	T90	D4318
Liquid limit	Max 25%	T89	D4318
Sand equivalent value	Min 30%	T176	D2419
Los Angeles abrasion	Max 50%	T96	C131 & C535
CBR	Min 25%	T193-99	D1883
Soundness	Max 12%	T104	C88

5.16 Mixes of Natural Aggregates (NA) with Recycled Concrete Aggregates (RCA)

In this study, a portion of natural aggregate was replaced by recycled concrete aggregate and evaluated as a subbase material. The replacement levels were 20% and 50% by weight of the recycled concrete aggregate, so that the mix designs applied to tests, were:100% NA as control mix, 80%NA+20%RCA, 50%NA+50%RCA, 20%NA+80%RCA and 100%RCA. The particle size distributions of these mixtures were within Type 2 subbase grading curves which were illustrated in Figures 5.43 to 5.47:

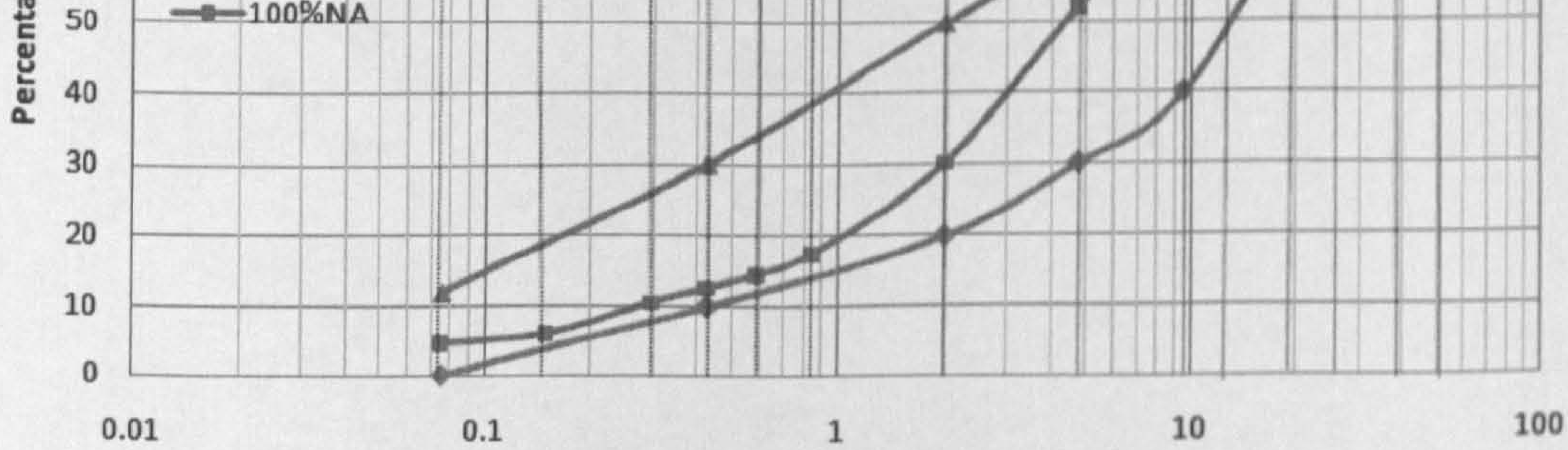


Figure 5.43 Grading curve of 100%NA

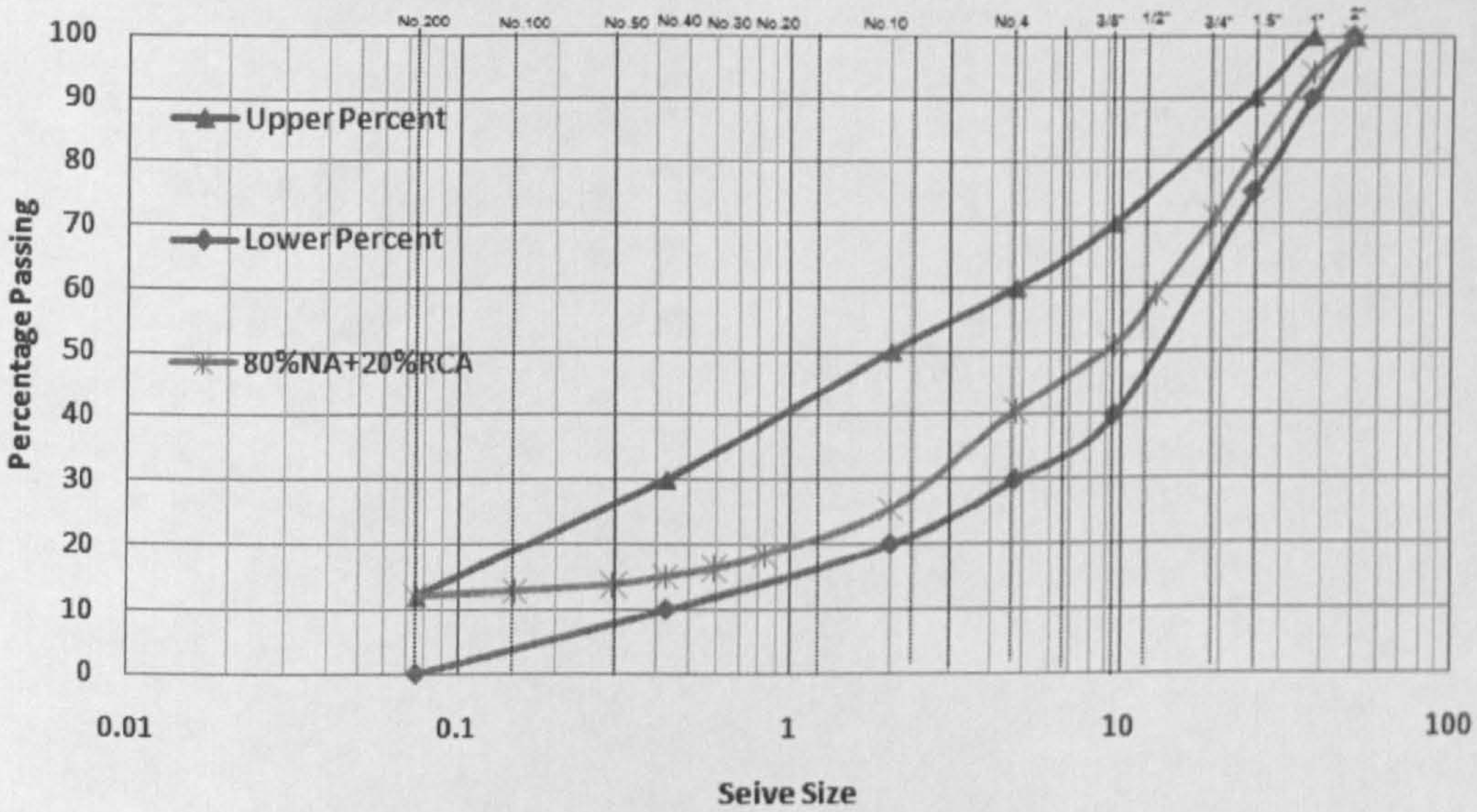


Figure 5.44 Grading curve of 80%NA+20%RCA

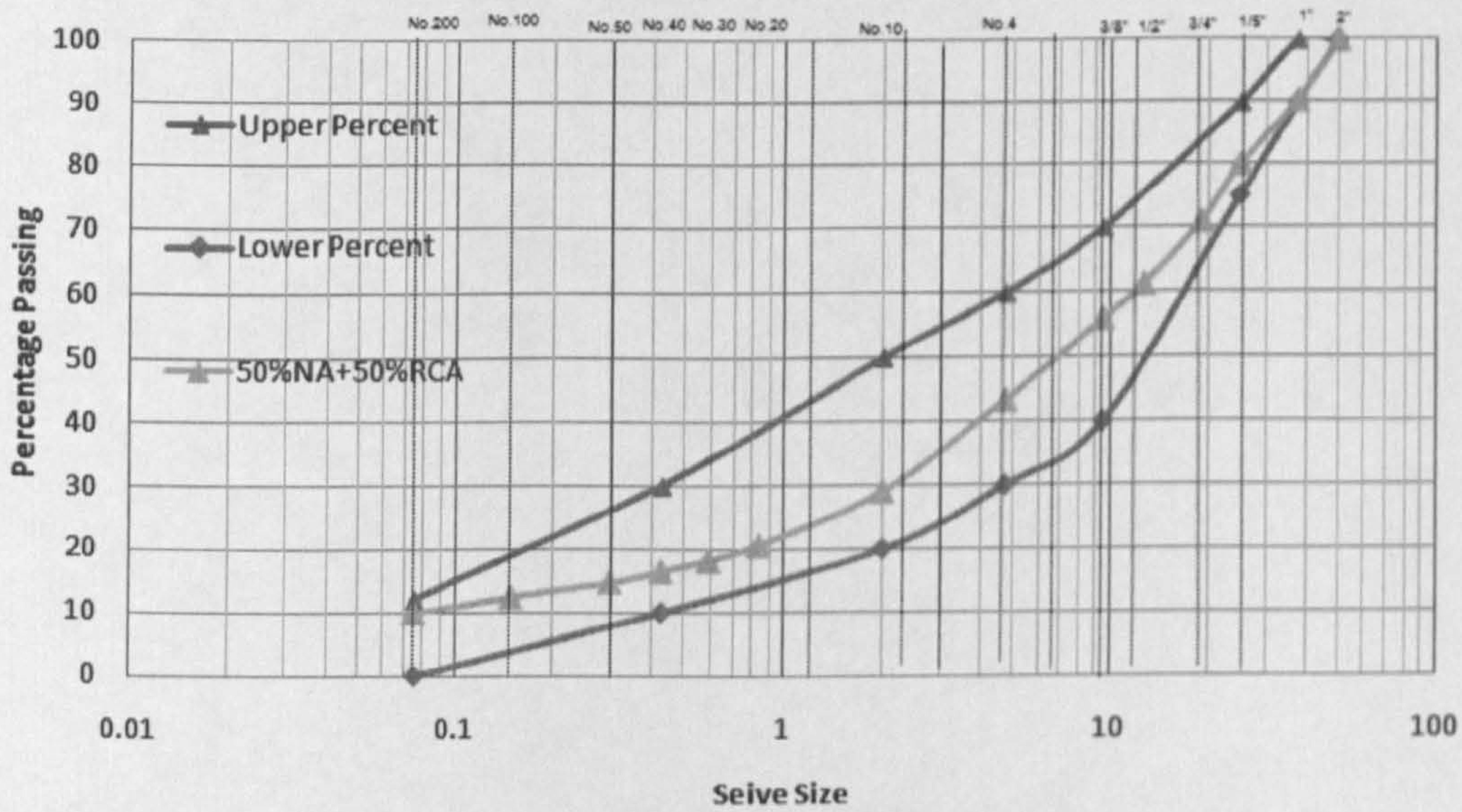


Figure 5.45 Grading curve of 50%NA+50%RCA

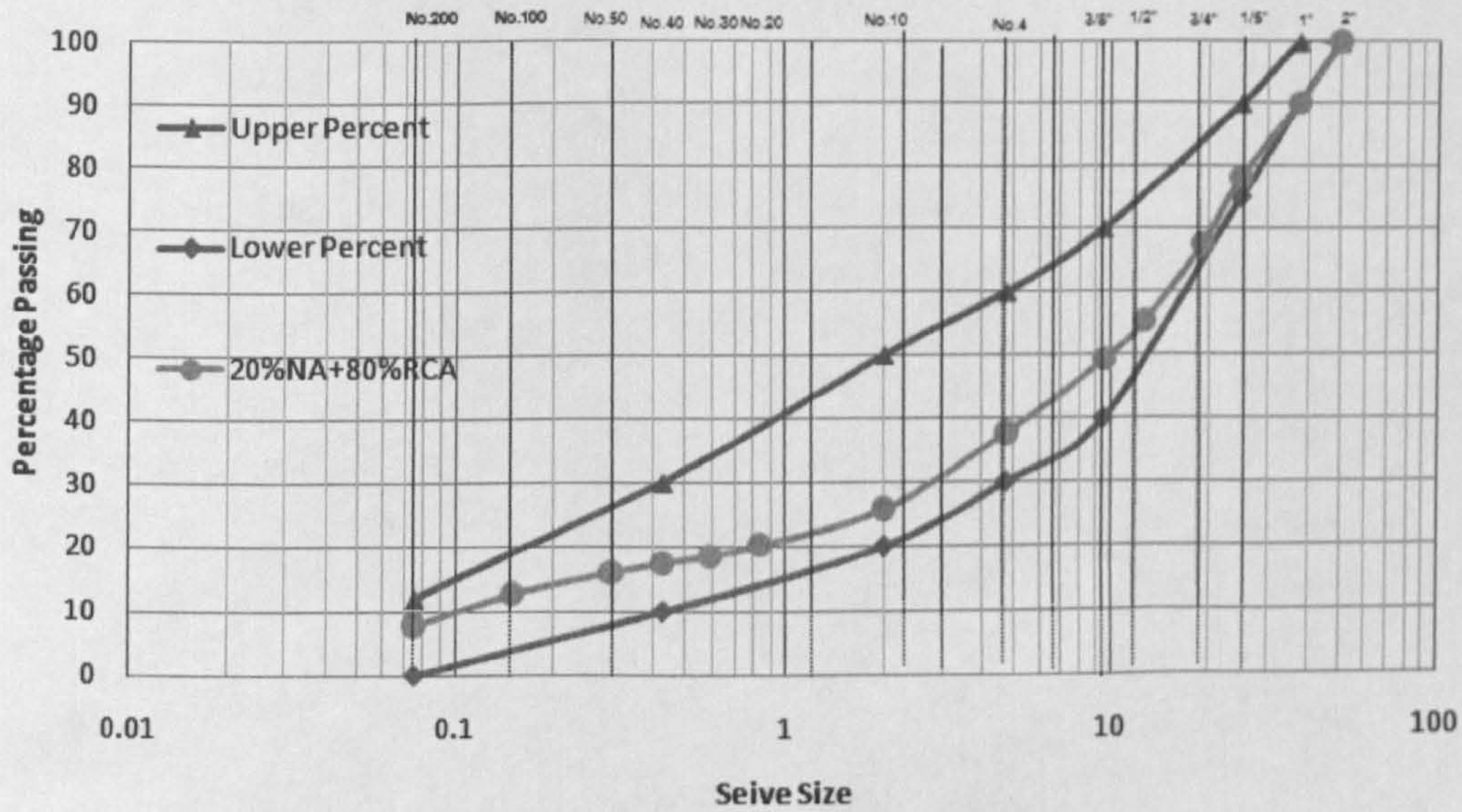


Figure 5.46 Grading curve of 20%NA+80%RCA

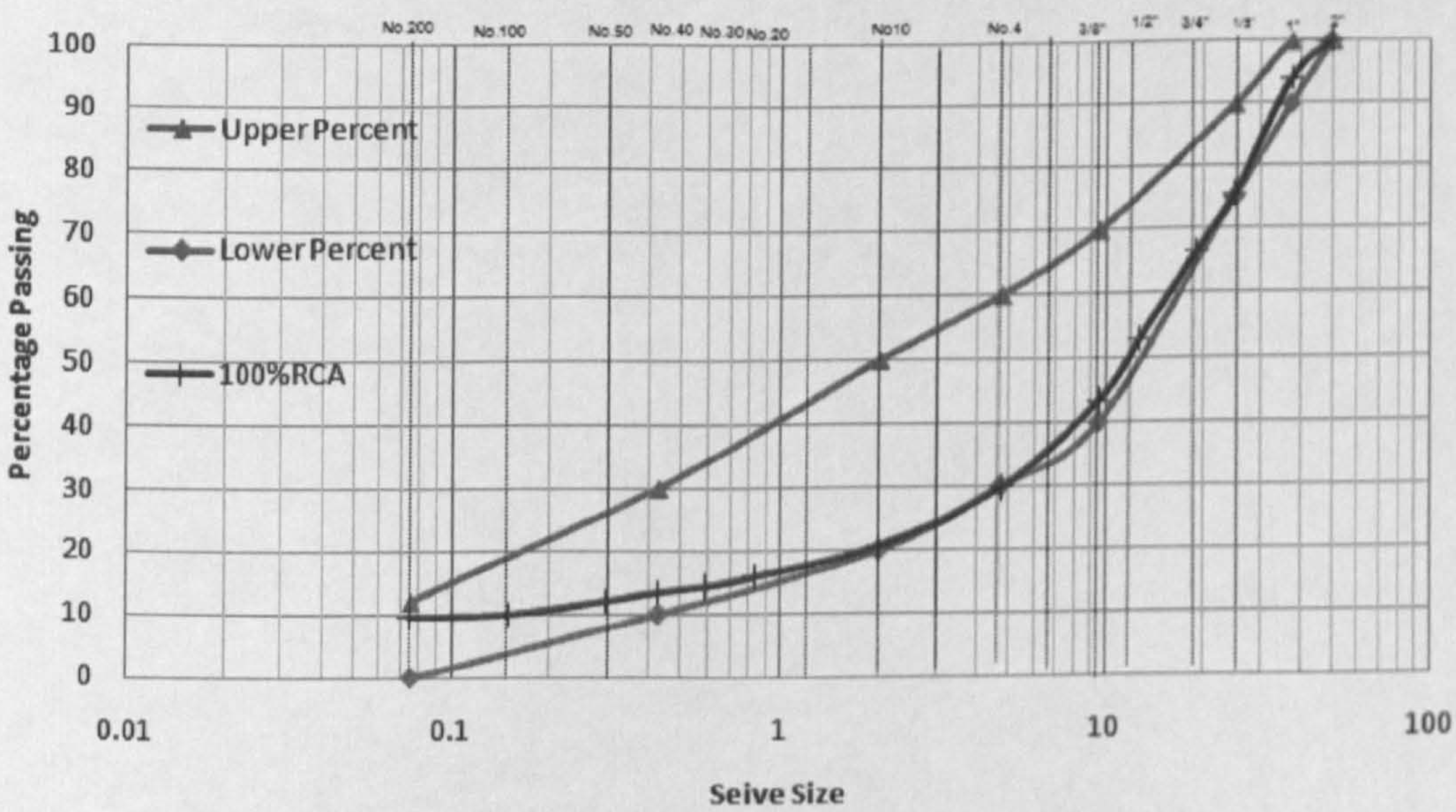


Figure 5.47 Grading curve of 100%RCA

5.16.1 Atterberg limits

This test was included liquid limit (LL) and plastic limit (PL) of mixtures. Liquid limit was performed in accordance with AASHTO T89-02(2007) on the materials passing the 0.425mm sieve with liquid limit device. For 100%RCA and 100%NA the liquid limit in the Casagrande dish method was achieved at low numbers of blows. For 100% RCA and 100%NA, the only trials taken were 10 blows (with 37% moisture) and 16 blows (with 17% moisture) respectively. For other mixtures, the presence of RCA resulted in slow absorption of water by the test portions. To determine plastic limit according to AASHTO T 90-00(2004) the portions crumbled at early stages of rolling, which meant that the plastic limit could not be determined for NA and RCA. As a result of this tests NA and RCA and their mixes were non-plastic (NP).

5.16.2 Sand equivalent value (SE)

Under standard conditions of ASTM designation: D2419-02(2002) the relative proportions of clay-like or plastic fines and dust in fine portion of each mix that passed the 4.75-mm (No. 4) sieve were measured. The test was performed separately with two irrigator tubes and mechanized shaker.The following averages of SE were reported for samples in Table 5.6:

Table 5.6 Sand equivalents of mixes

MATERIAL	SE (%)
100%RCA	76
20%NA+80%RCA	71
50%NA+50%RCA	69
80%NA+20%RCA	60
100%NA	64

The results indicated that RCA had less undesirable fine dust than NA. Undesirable dust can coat aggregate particles and prevent proper subbase particles bonding. All mixes meet the requirement of Iran Highway Asphalt Paving Code which requires a minimum SE value of 30%.

5.16.3 Los Angeles abrasion (LAA)

5.16.3.1 Method Statement

A 10-kg sample of 100%NA and 50%NA+50%RCA prepared according to Grading B and subjected to Los Angeles abrasion test following ASTM C131-06(2006). The test required that each sample be placed with 12 steel spheres inside a metal drum that rotated at 30 revolutions per minute for 1000 revolutions. The weight loss in percentage by abrasion and impact was calculated from the original total mass and the final mass obtained after samples were washed over a No. 12 (1.70mm) sieve and oven dried at 110℃. For other mixtures such as 80%NA+20%RCA, 20%NA+80%RCA and 100%RCA in accordance with ASTM C535-03(2003) the grading designation 2 was chosen because of the available aggregate particles.

5.16.3.2 Test results

Los Angeles abrasion (LAA) losses for each of the materials are largely consistent with the values reported by other researchers (Blankenagel and Geuthrie, 2006 and Taha *et al.*, 2002). The results are summarized below:

Mix type	weight loss in LAA
100%RCA	31%
20%NA+80%RCA and 50%NA+50%RCA	30%
80%NA+20%RCA	28%
100%NA	24%

Therefore, in general the weight loss in LAA decreased as the level of replacement of RCA with NA increased. It is clearly evident that replacing RCA with NA decreased the amount of fine materials produced from stripping of cement paste during LAA testing (Blankenagel and Geuthrie, 2006). In the LAA test, NA particles are more resistant to impact forces of the steel spheres in comparison to RCA. An advantage of less particle breakdown is that the percentage of fine particles is decreased. Consequently the change in gradation of the material, if used as a highway pavement layer, provides enhanced stiffness to support in-service loads. Additionally, such a gradation change results in decreased water absorption potential and improved drainage properties of unbound subbase made with the mix of RCA and NA. As will be seen from Figures 5.57 and 5.58, degradation of NA was much less than that of RCA.

5.16.4 Water absorption

5.16.4.1 Method Statement

AASHTO T 85-91 covers the determination of specific gravity and absorption of coarse aggregate. As the mixtures contained more than 15 percent retained on the 37.5mm (1½") sieve, it was desirable to test coarse aggregates in the following separate size fractions:

a. Size fraction larger than 37.5mm (1½")
b. Size fraction larger than 4.75mm (No.4) and smaller than 37.5mm (1½")

According to the method statements and other requirements of AASHTO T 84, T 85 water absorption tests were carried out on ten kinds of subbase mixtures. The determination of specific gravity and water absorption of fine aggregates was carried out according to the tests described in AASHTO T-84. Size fraction of specimens were chosen from materials passing the 4.75mm (No.4) sieve and saturated surface-dry (SSD) condition was determined using the cone test method within the water absorption test procedure.

The specific gravity tests included bulk specific gravity, bulk specific gravity (SSD), and apparent specific gravity (oven-dry). Bulk specific gravity (G_{sb}) is the characteristic used for calculation of the volume occupied by the aggregate in various mixtures of RCA and NA containing aggregates, cement and coarse and fine fractions of NA. In addition, these solid particles, the voids including permeable and impermeable voids (but not including the voids between particles) must be taken into account to calculate its unit weight. The bulk SSD specific gravity (G_{sb} SSD) was based on aggregates soaked for 15 hours in water to satisfy the absorption requirement. The weight of aggregates and water within the voids filled to the extent achieved in soaking time (but not including the voids between particles) were calculated to find the weight in air of a unit volume of each mixture. Apparent specific gravity (G_{sa}) pertained to the relative density of aggregates not including the permeable pores.
Water absorption (% Abs.) values were used to determine the increase in weight of mixtures due to water in their pores after submerging dry aggregates for 15 hours in water, compared to the dry condition. The dry condition was attained by drying the test samples to a constant weight at a temperature of 110°C. For all mixtures the tests were performed with two specimens of each for all fractions and results reported for the average of specific gravity values or water absorption percentages.

5.16.4.2 Test results

The results of tests performed on each particle size of mixes of RCA and NA are summarized in Figures 5.48-5.55.

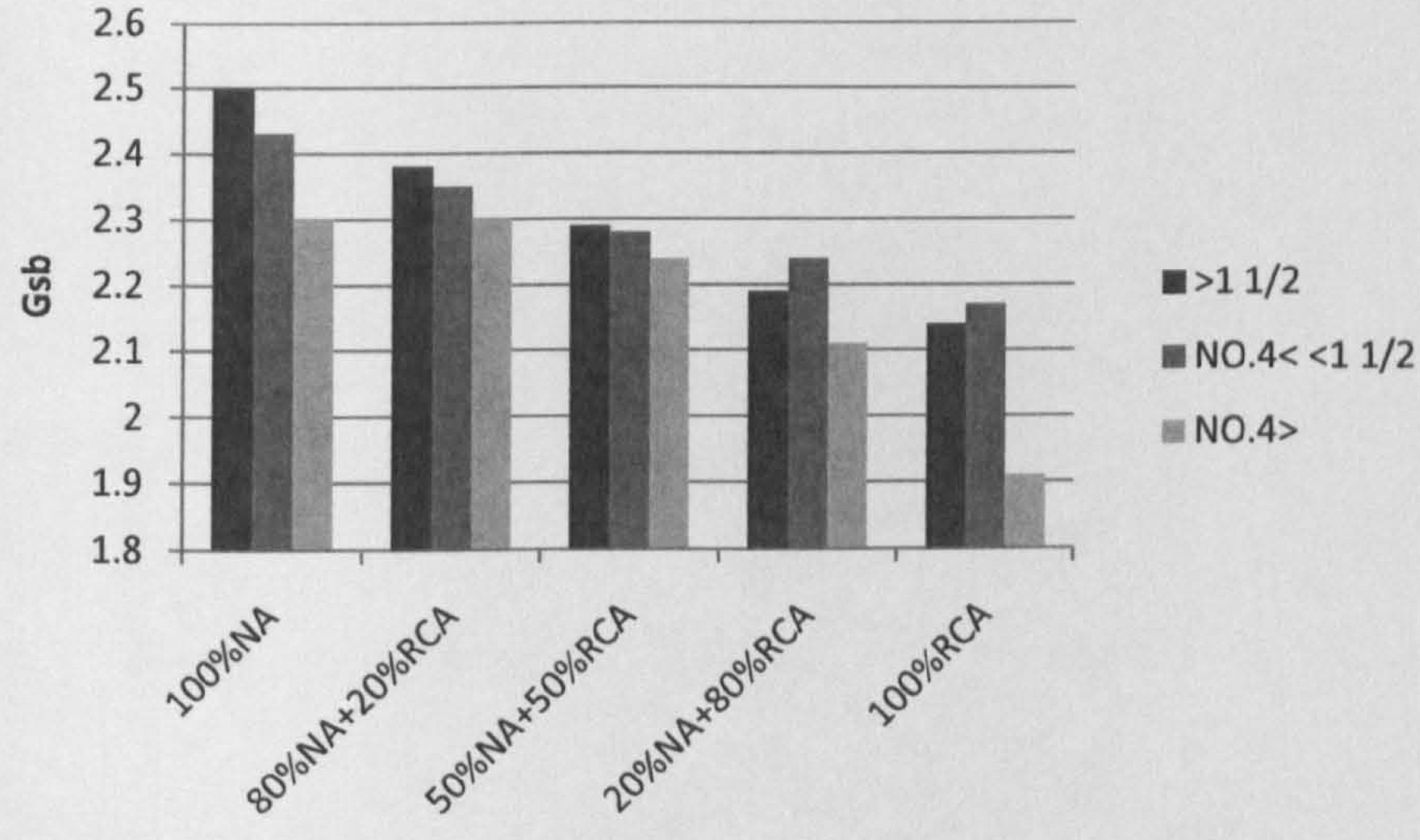


Figure 5.48 Bulk specific gravity (G_{sb}) of RCA and NA mixes

The average value of bulk specific gravity for whole mix including all fractions was calculated from Equation 5.5 and the results were illustrated in Figure 5.49.

$$G_{sb} = \frac{1}{\frac{P_1/100}{G_{sb\,1}} + \frac{P_2/100}{G_{sb\,2}} + \frac{P_3/100}{G_{sb\,3}}} \qquad \textbf{(Equation 5.5)}$$

Where:

G_{sb}= average value of bulk specific gravity

P_1= fraction larger than 37.5mm (1½") in percent

P_2= fraction larger than 4.75mm (No.4) and smaller than 37.5mm (1½") in percent

P_3= fraction finer than 4.75mm (No.4)

$G_{sb\,1}$= bulk specific gravity of fraction larger than 37.5mm (1½")

$G_{sb\,2}$= bulk specific gravity of fraction larger than 4.75mm (No.4) and smaller than 37.5mm (1½")

$G_{sb\,3}$= bulk specific gravity of fraction finer than 4.75mm (No.4)

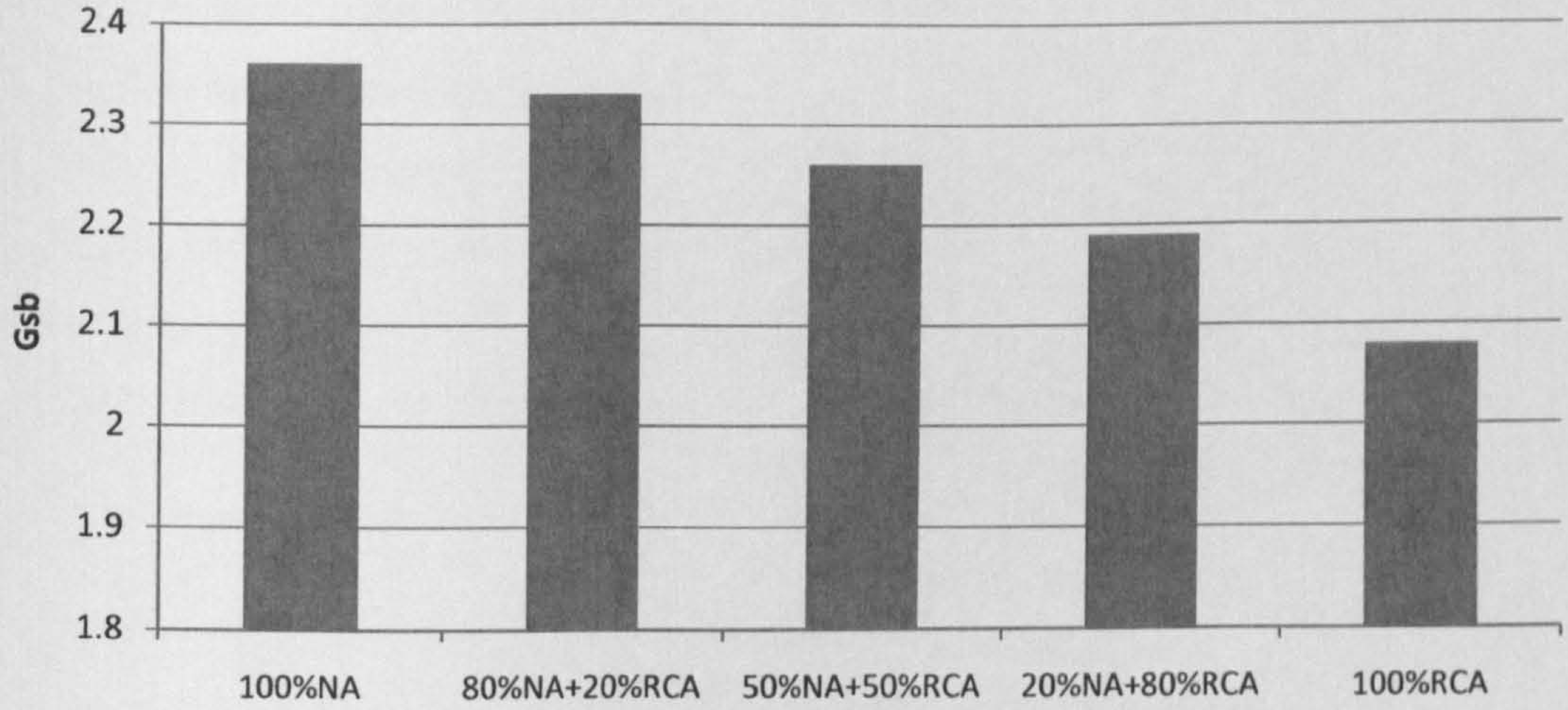


Figure 5.49 Average value of G_{sb} for whole mixes of RCA and NA

The G_{sb} is always the lowest value among the gravities, because the volume calculated includes voids permeable to water. The G_{sb} SSD is the intermediate value, and the G_{sa} is the highest, because the volume calculated includes only the solid aggregate particles which is the impermeable portion of aggregates.

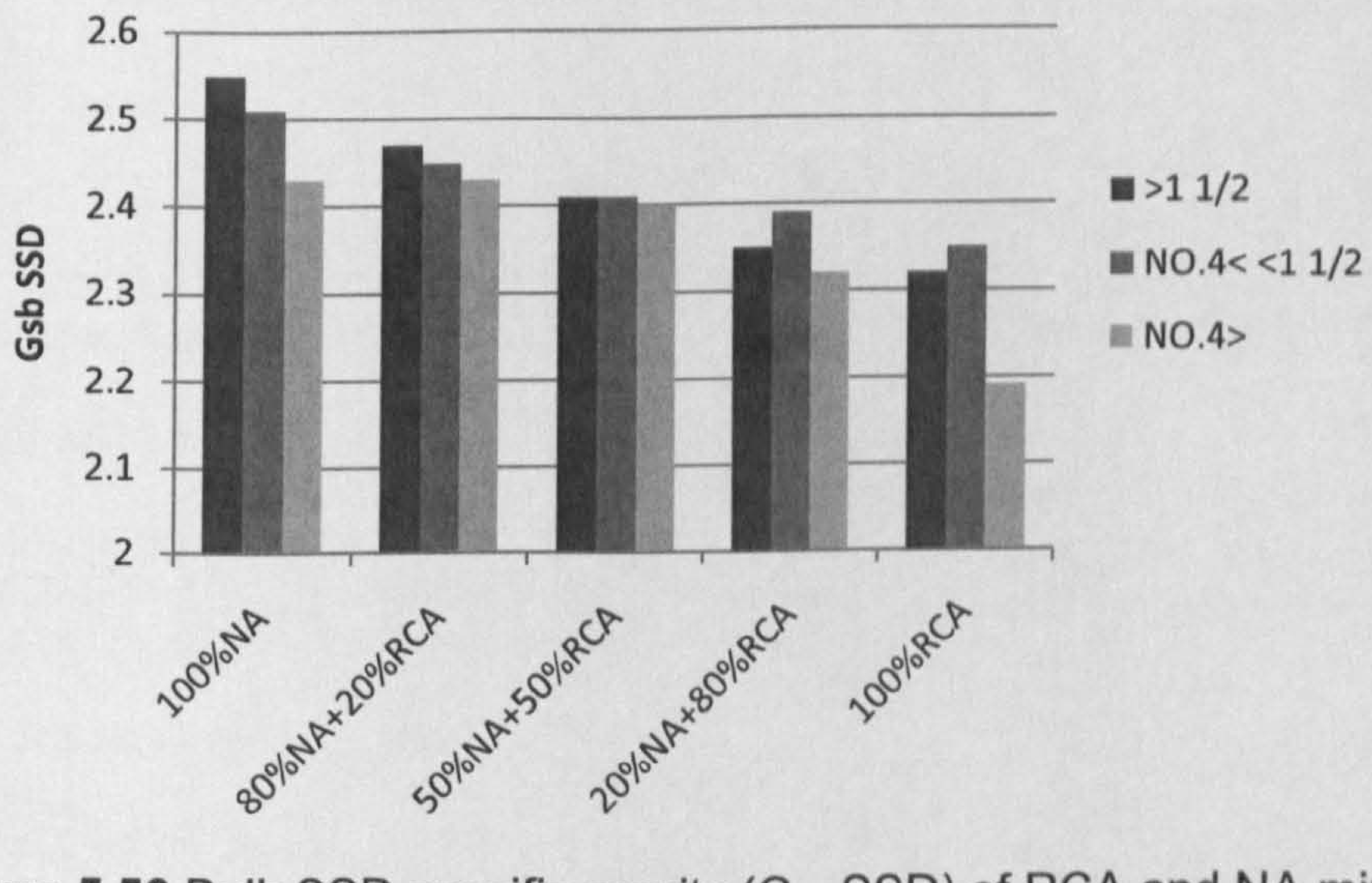


Figure 5.50 Bulk SSD specific gravity (G_{sb} SSD) of RCA and NA mixes

The average value of bulk SSD specific gravity for whole mix including all fractions was calculated from Equation 5.6 and the results illustrated in Figure 5.51.

$$G_{sb}SSD = \frac{1}{\frac{P_1/100}{G_{sb\ 1}SSD}+\frac{P_2/100}{G_{sb\ 2}SSD}+\frac{P_3/100}{G_{sb\ 3}SSD}} \quad \textbf{(Equation 5.6)}$$

Where:

$G_{sb}SSD$= average value of bulk SSD specific gravity

P_1= fraction larger than 37.5mm (1½") in percent

P_2= fraction larger than 4.75mm (No.4) and smaller than 37.5mm (1½") in percent

P_3= fraction finer than 4.75mm (No.4)

$G_{sb\,1}SSD$= bulk SSD specific gravity of fraction larger than 37.5mm (1½")

$G_{sb\,2}SSD$= bulk SSD specific gravity of fraction larger than 4.75mm (No.4) and smaller than 37.5mm (1½")

$G_{sb\,3}SSD$= bulk SSD specific gravity of fraction finer than 4.75mm (No.4)

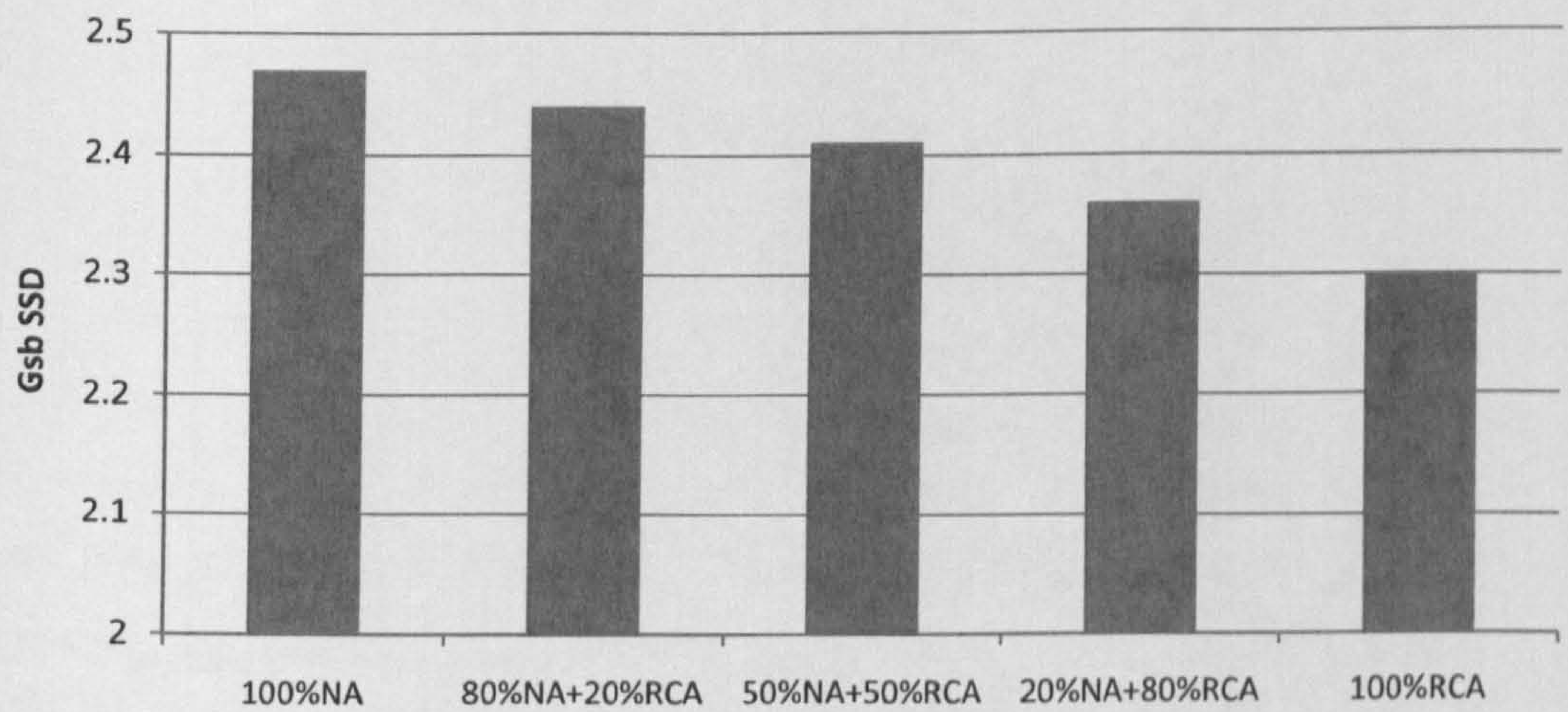


Figure 5.51 Average value of G_{sb} SSD for whole mixes of RCA and NA

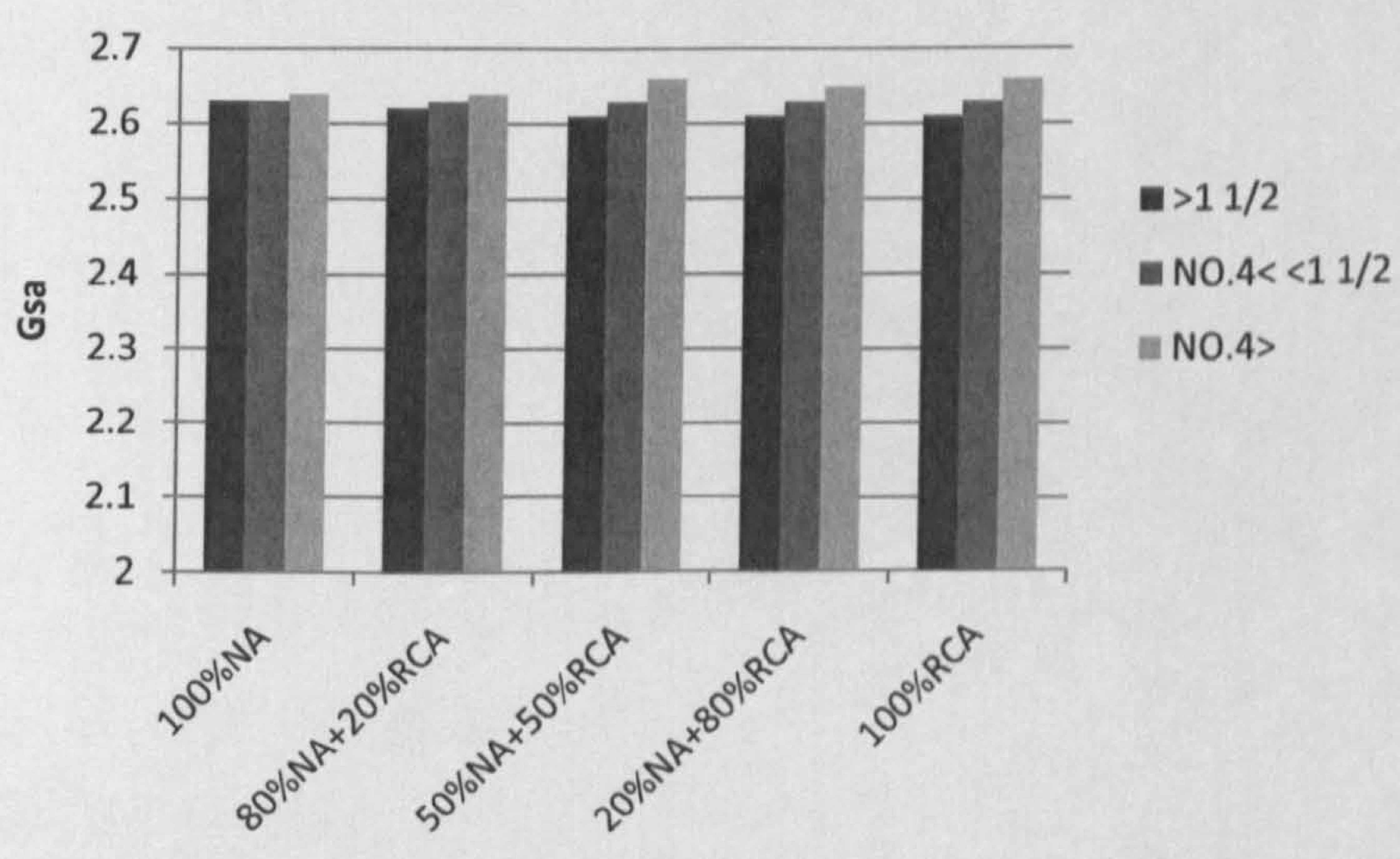


Figure 5.52 Apparent specific gravity (G_{sa}) of RCA and NA mixes

The average value of apparent specific gravity for whole mix including all fractions was calculated from Equation 5.7 and the results illustrated in Figure 5.53.

$$G_{sa} = \frac{1}{\frac{P_1/100}{G_{sa\,1}} + \frac{P_2/100}{G_{sa\,2}} + \frac{P_3/100}{G_{sa\,3}}} \qquad \textbf{(Equation 5.7)}$$

Where:

G_{sa}= average value of apparent specific gravity

P_1= fraction larger than 37.5mm (1½") in percent

P_2= fraction larger than 4.75mm (No.4) and smaller than 37.5mm (1½") in percent

P_3= fraction finer than 4.75mm (No.4)

$G_{sa\,1}$= apparent specific gravity of fraction larger than 37.5mm (1½")

$G_{sa\,2}$= apparent specific gravity of fraction larger than 4.75mm (No.4) and smaller than 37.5mm (1½")

$G_{sa\,3}$= apparent specific gravity of fraction finer than 4.75mm (No.4)

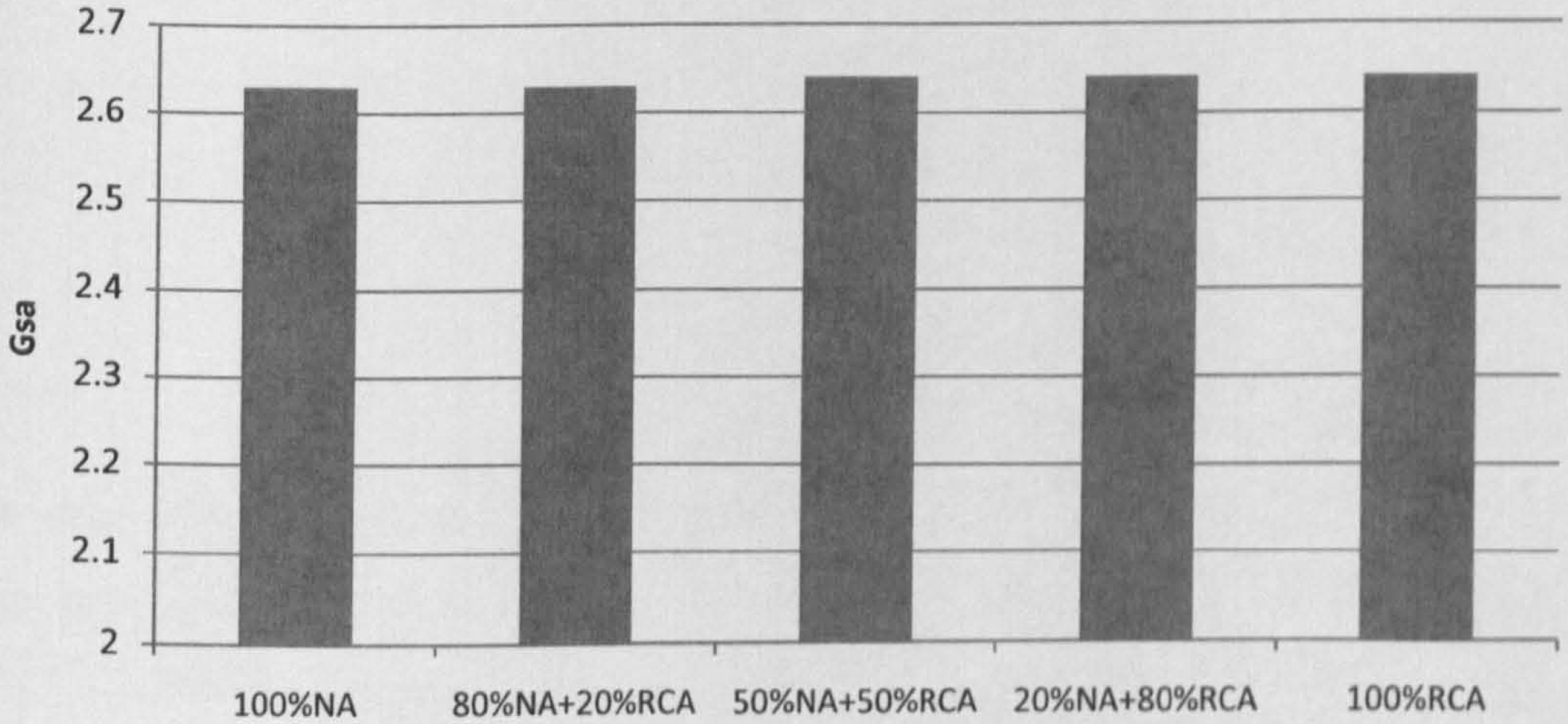


Figure 5.53 Average value of G_{sa} for whole mixes of RCA and NA

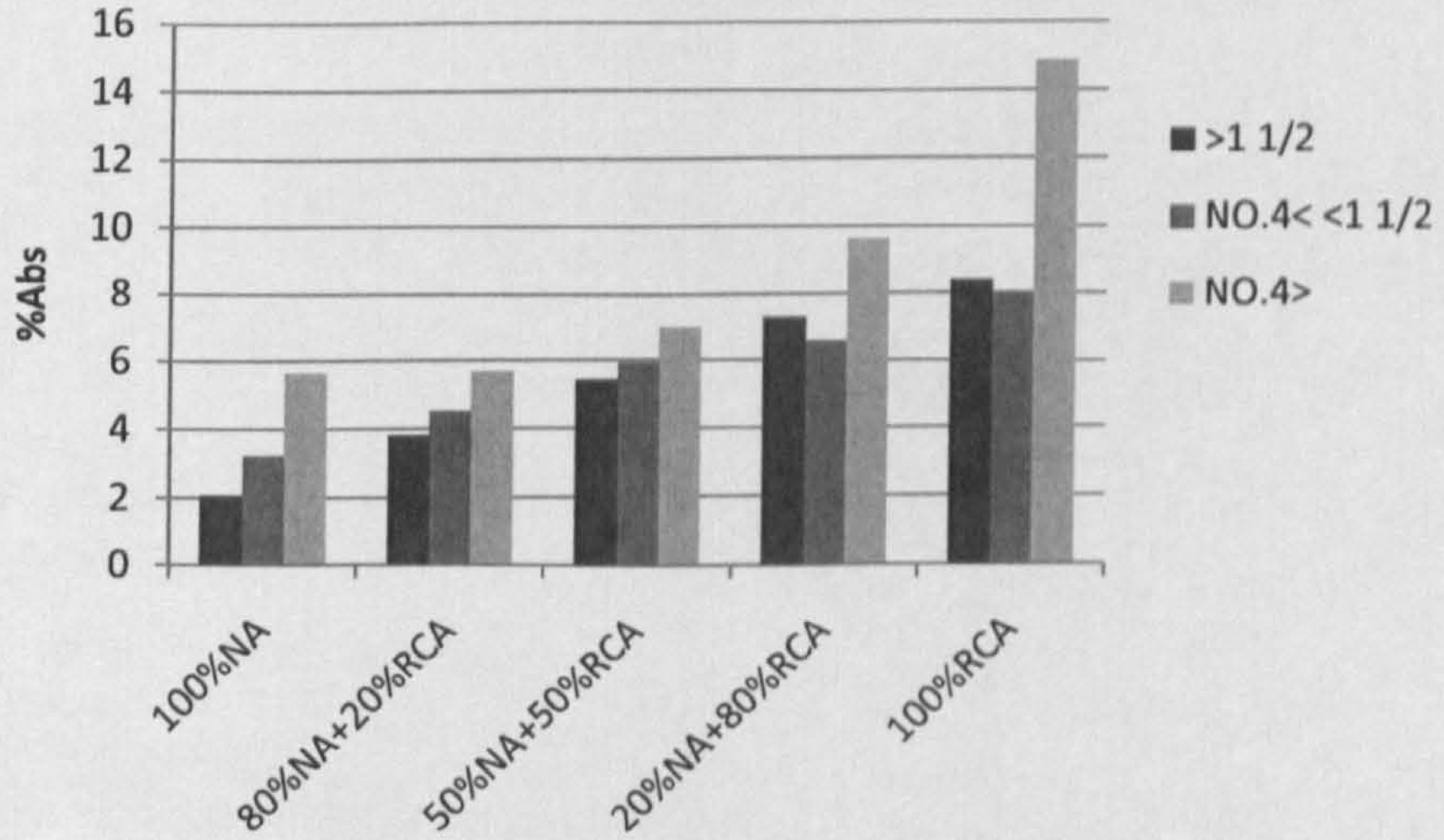


Figure 5.54 Water absorption (%Abs) of RCA and NA mixes

The average value of water absorption for whole mixes including all fractions was calculated from Equation 5.8 and the results illustrated in Figure 5.55.

$$Abs = \left(\frac{P_1 * Abs_1}{100}\right) + \left(\frac{P_2 * Abs_2}{100}\right) + \left(\frac{P_3 * Abs_3}{100}\right) \qquad \textbf{(Equation 5.8)}$$

Where:

Abs= average value of water absorption in percent

P_1= fraction larger than 37.5mm (1½") in percent

P_2= fraction larger than 4.75mm (No.4) and smaller than 37.5mm (1½") in percent

P_3= fraction finer than 4.75mm (No.4)

Abs_1= water absorption of fraction larger than 37.5mm (1½") in percent

Abs_2= water absorption of fraction larger than 4.75mm (No.4) and smaller than 37.5mm (1½") in percent

Abs_3= water absorption of fraction finer than 4.75mm (No.4) in percent

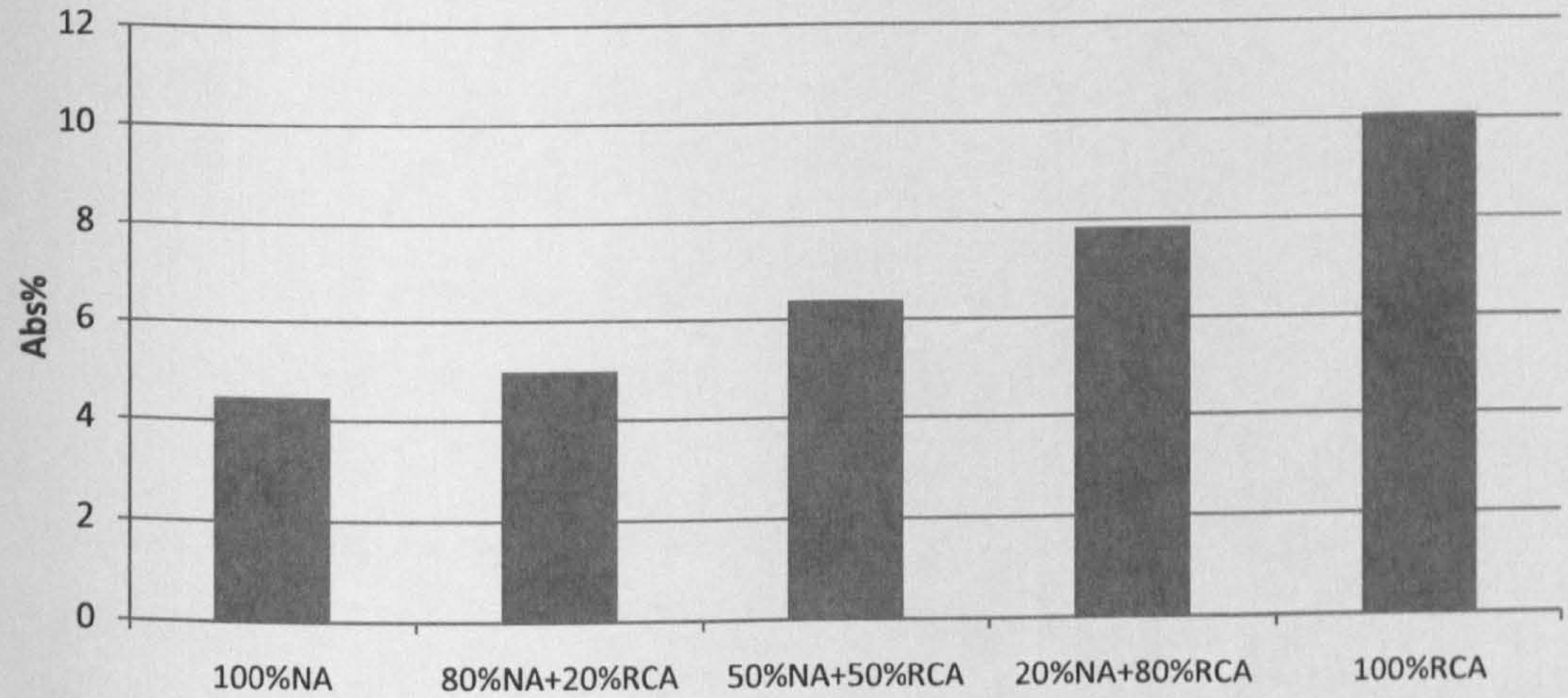


Figure 5.55 Average value of %Abs for whole mixes of RCA and NA

The tests results showed that 100%NA had the highest G_{sb}, G_{sb} SSD and G_{sa} for particles in all size ranges. This is true except for the G_{sa} of the fine fraction (smaller than 4.75mm) of the mixes: {50%NA+50%RCA, 20%NA+80%RCA and 100%RCA}. The G_{sa} of these materials is believed to be influenced by the high percentage of the fine cement particles in a unit volume, thereby affecting the average values of G_{sa} as well (Figures 5.52 and 5.53).

5.16.5 Compaction test

5.16.5.1 Method Statement

Here, the moisture-density relationship for each of the mixes was determined according to the traditional Method D described in AASHTO T 180-01(2004). Aggregates passing the 19 mm sieve were compacted in the standard mold which had a capacity and mass of 2104cc and 6778g, respectively. A 4.54kg mechanical rammer, falling freely through a height of 457mm, was used to compact each specimen in five approximately equal layers with 56 blows to give a total compacted depth of about 125mm.

5.16.5.2 Test results

For the material mixes tested, the curves in Figure 5.56 illustrate the variation of dry density with moisture content.

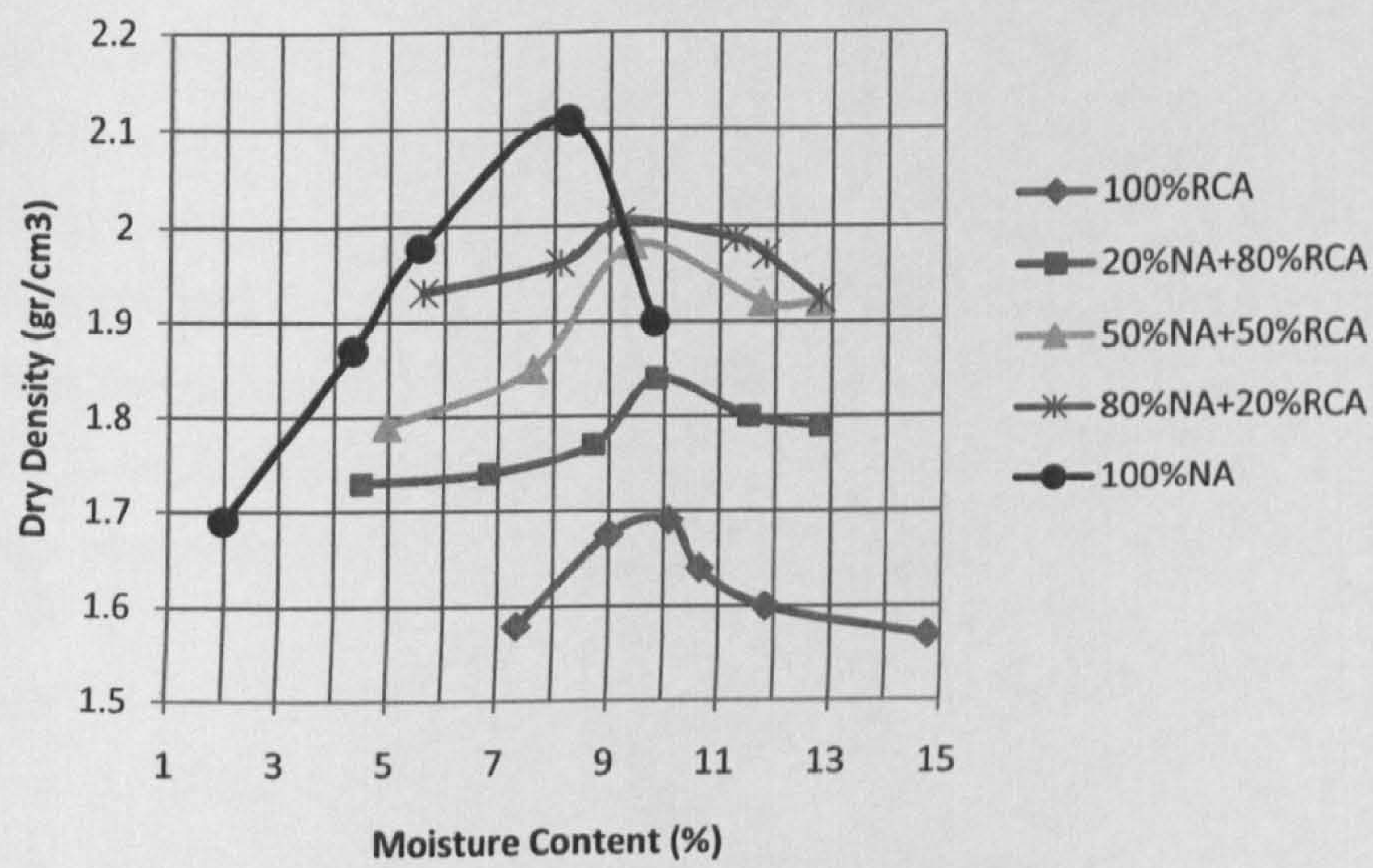


Figure 5.56 Density-Moisture curves

The values of OMC and MDD from the compaction test curves are listed in Table 5.7.

Table 5.7 Summary of OMC and MDD for six subbase materials

MATERIAL	OMC (%)	MDD (g/cm^3)
100%RCA	10.09	1.69
20%NA+80%RCA	9.86	1.84
50%NA+50%RCA	9.41	1.98
80%NA+20%RCA	9.19	2.00
100%NA	8.23	2.11

As seen in Table 5.7, the mixture 100%RCA had the highest OMC while the 100%NA material had the largest MDD. It was found that increased content of RCA in a mixture of RCA+NA resulted in an increase in the moisture content of the material owing to the role of the undydrated cement in the RCA fraction, as previously pointed out by Poon *et al.* (2006). Furthermore, the presence of RCA in the mixtures decreased the MDD, a parameter which relates closely to bulk specific gravity (G_{sb}) of aggregates passing the 4.75mm sieve. The G_{sb} for RCA was less than that of NA so that the overall result was that the greater the RCA percentage in the mix the less was the G_{sb} value. The specific gravity tests indicated that the G_{sb} of 100%RCA and 100%NA was 2.08 and 2.36 respectively.

In order to simulate the crushing of aggregates and consequent change of size under roller passes in the case of real highway pavement, sieve analysis was performed on aggregates less than 3/4"(=19mm) sizes. The grading curves for the RCA and NA materials before and after compaction test are shown in Figs. 5.57 and 5.58 respectively. As can be seen in Fig. 5.57, significant degradation was observed in the coarse size particles of RCA. For example, at size 4.75 mm (sieve No. 4) the difference between the percentage passing before and after compaction was found to be slightly more than 20%. As for the NA material, Fig. 5.58 shows that change in gradation due to compaction was much less significant.

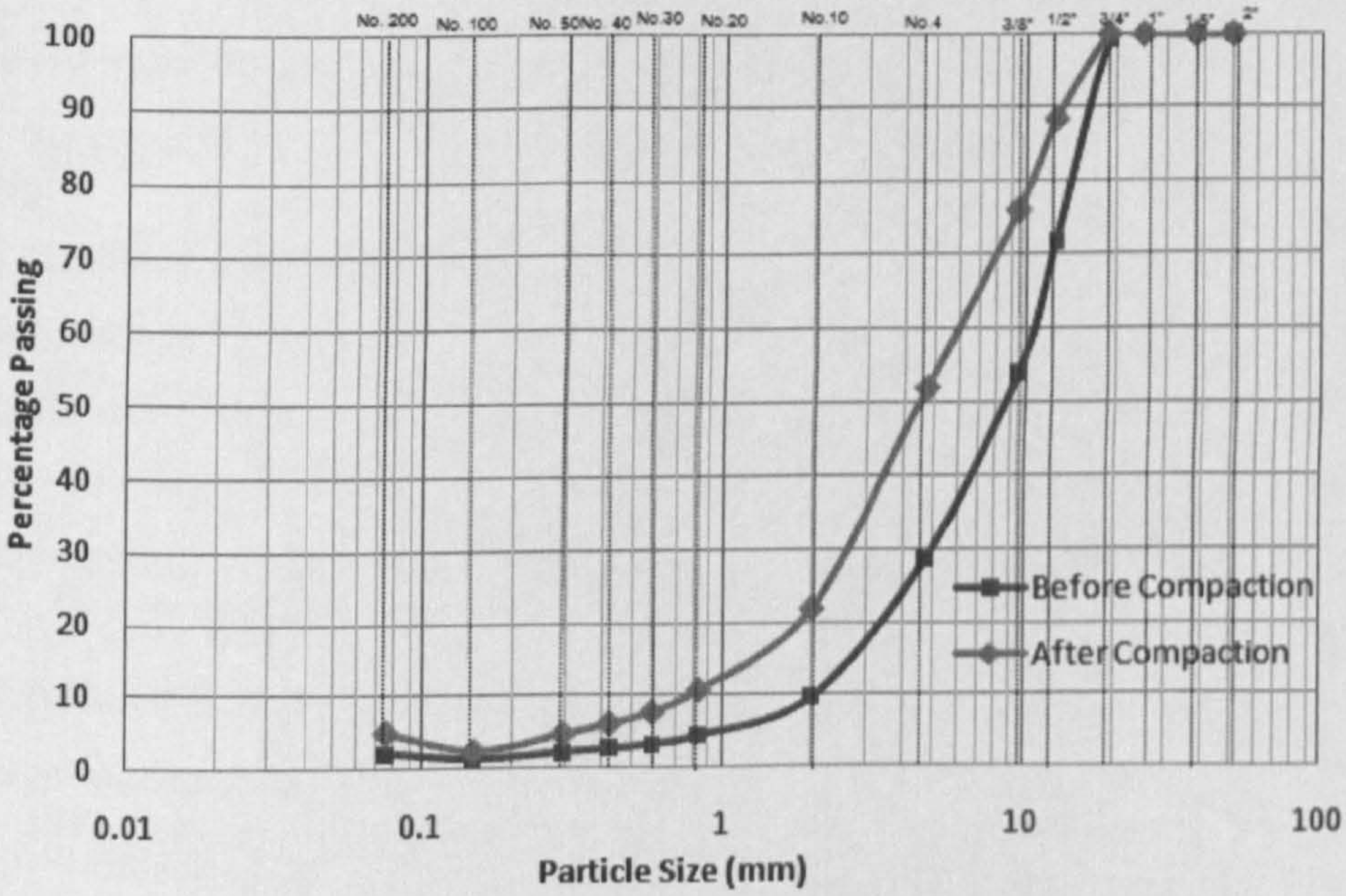


Figure 5.57 Sieve analysis of RCA compacted sample

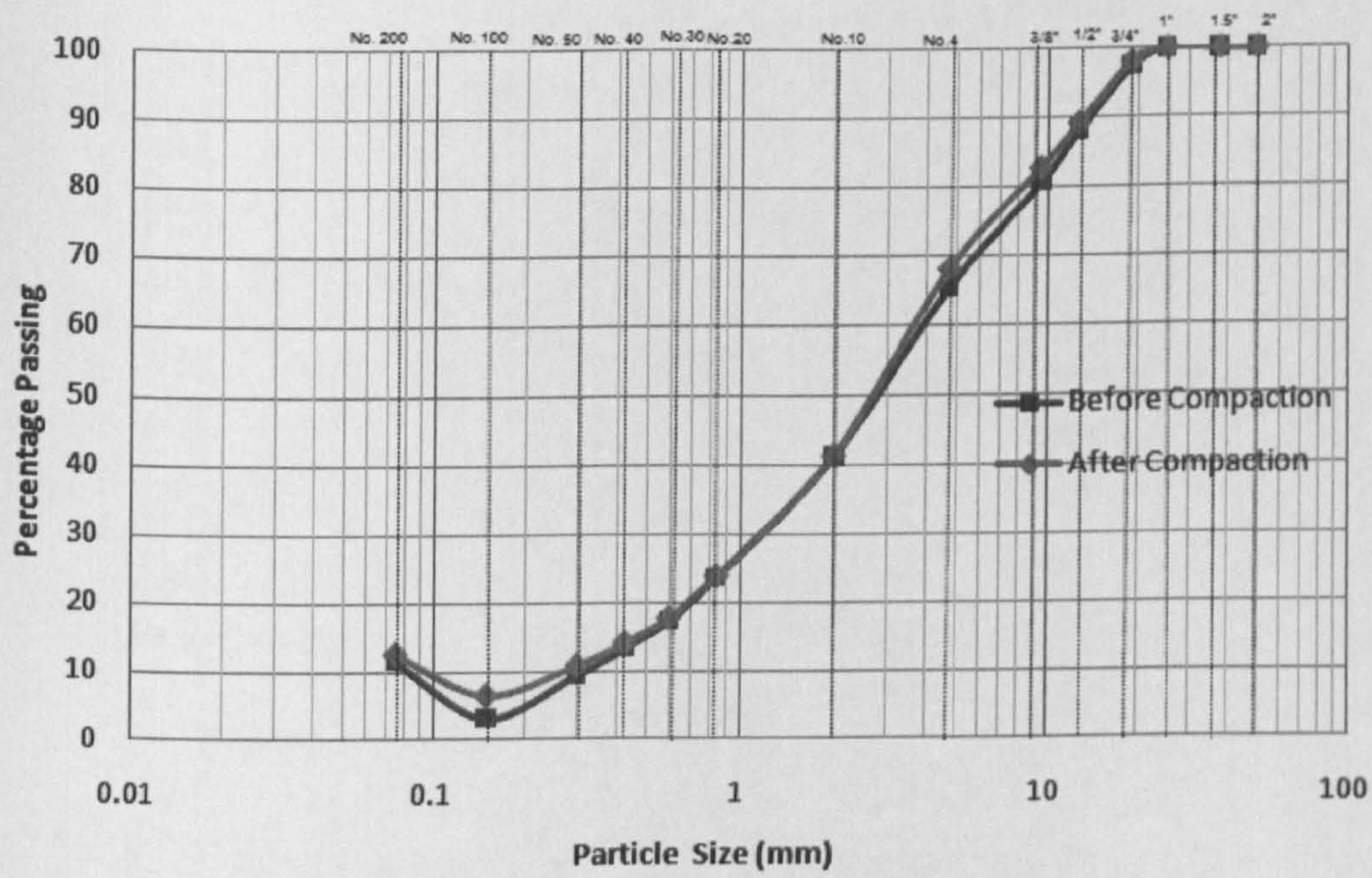


Figure 5.58 Sieve analysis of NA compacted sample

5.16.6 California bearing ratio (CBR)

5.16.6.1 Method Statement

CBR tests were carried out on the following mixes: 100%RCA; 20%NA+80%RCA; 50%NA+50%RCA; 80%NA+20%RCA and 100%NA, at their respective OMC, in accordance with AASHTO Designation T193-99 (2003). For each mix three specimens were prepared from materials passing a 19.0mm (3/4") sieve and then compacted in CBR molds using the procedures given in AASHTO T 180-01(2004). The three specimens from each mix were compacted with the following different number of blows (per layer in 5 layers): 10, 30 and 65. Plunger loads were applied to maintain a uniform rate of penetration of 1.3 mm/min, allowing the force-penetration curves to be plotted for each specimen. From these curves, the corrected load values were determined for each specimen at 2.54 mm and 5.08 mm penetration. Unsoaked and soaked CBR values were obtained by comparing these corrected loads with the standard loads of 1360kg and 2040kg, respectively. To evaluate the swelling properties and their effects on mixtures, CBR values were measured at the end of 96 hours of soaking. Figures 5.59-5.63 show CBR versus dry density relationships for each mixture at the three compactive efforts.

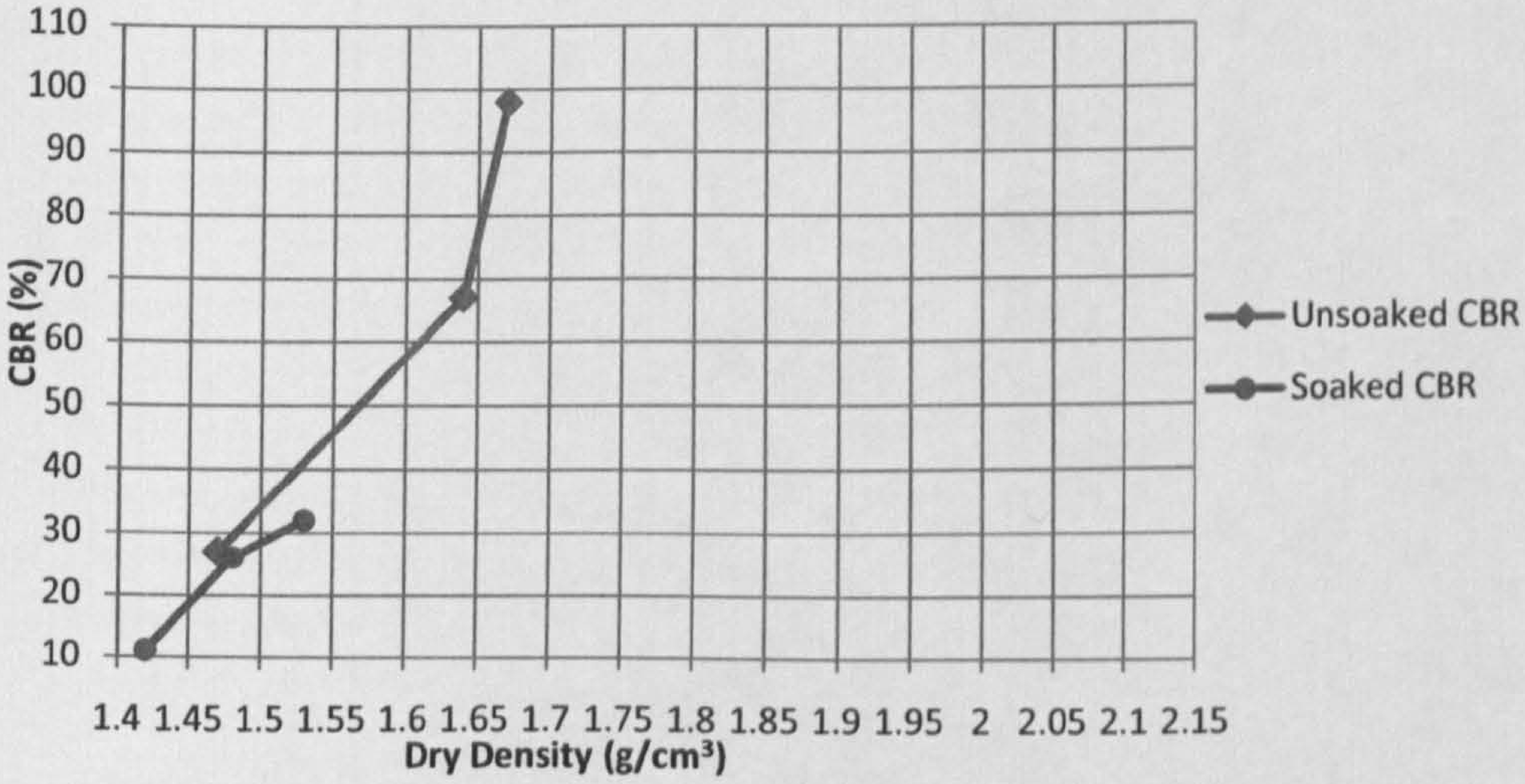


Figure 5.59 CBR-Dry Density relationship for 100%RCA

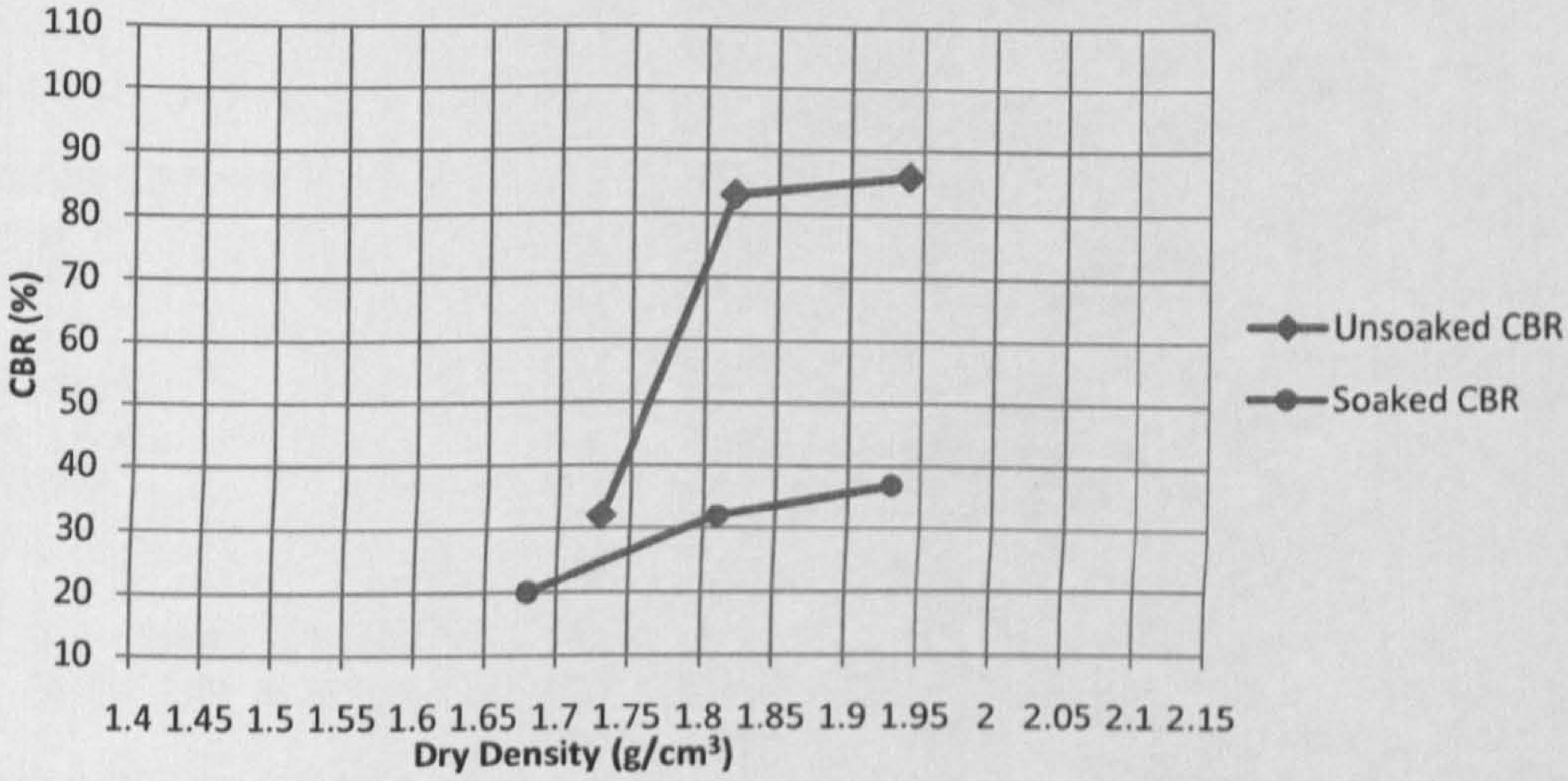


Figure 5.60 CBR-Dry Density relationship for 80%RCA+20%NA

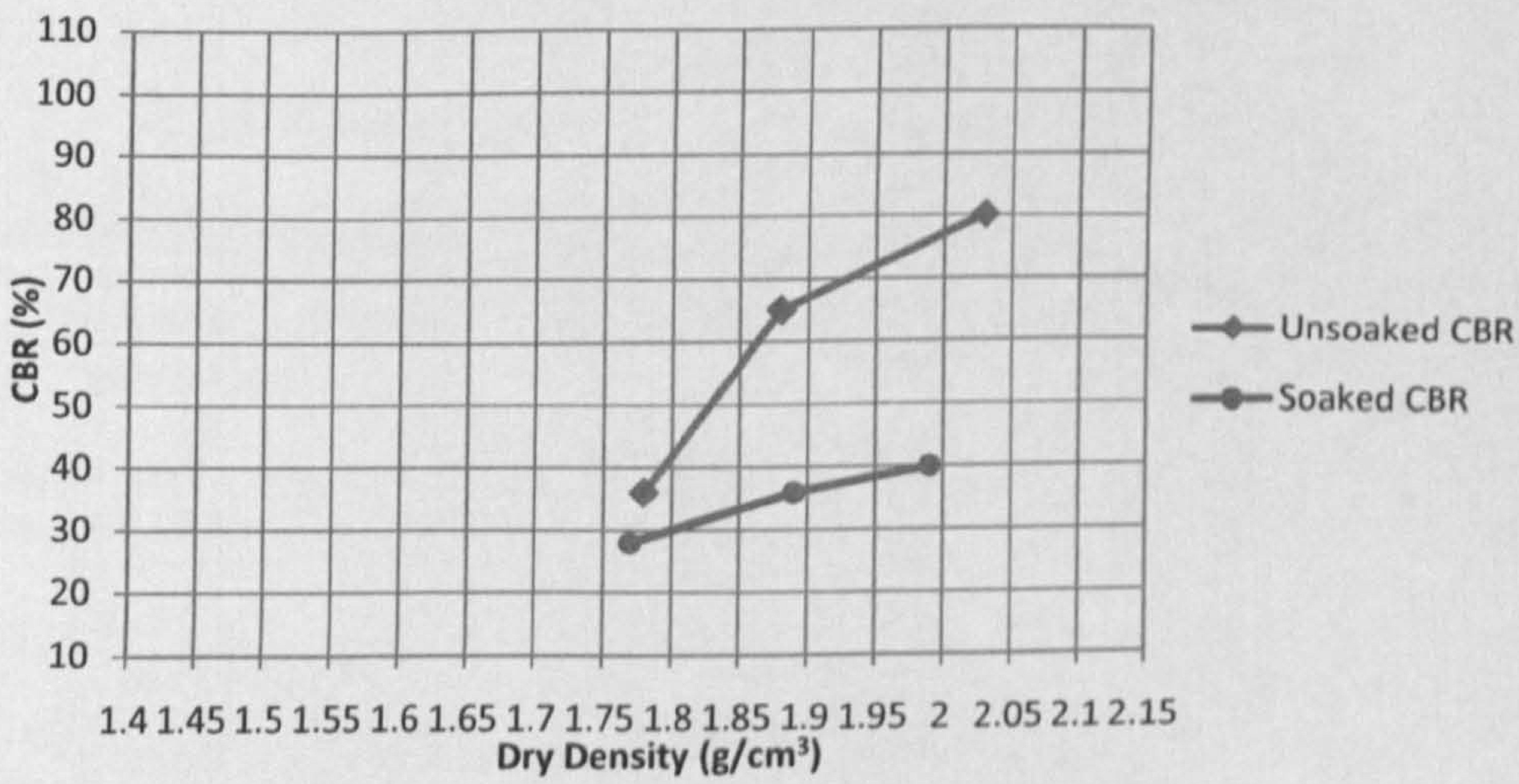


Figure 5.61 CBR-Dry Density relationship for 50%RCA+50%NA

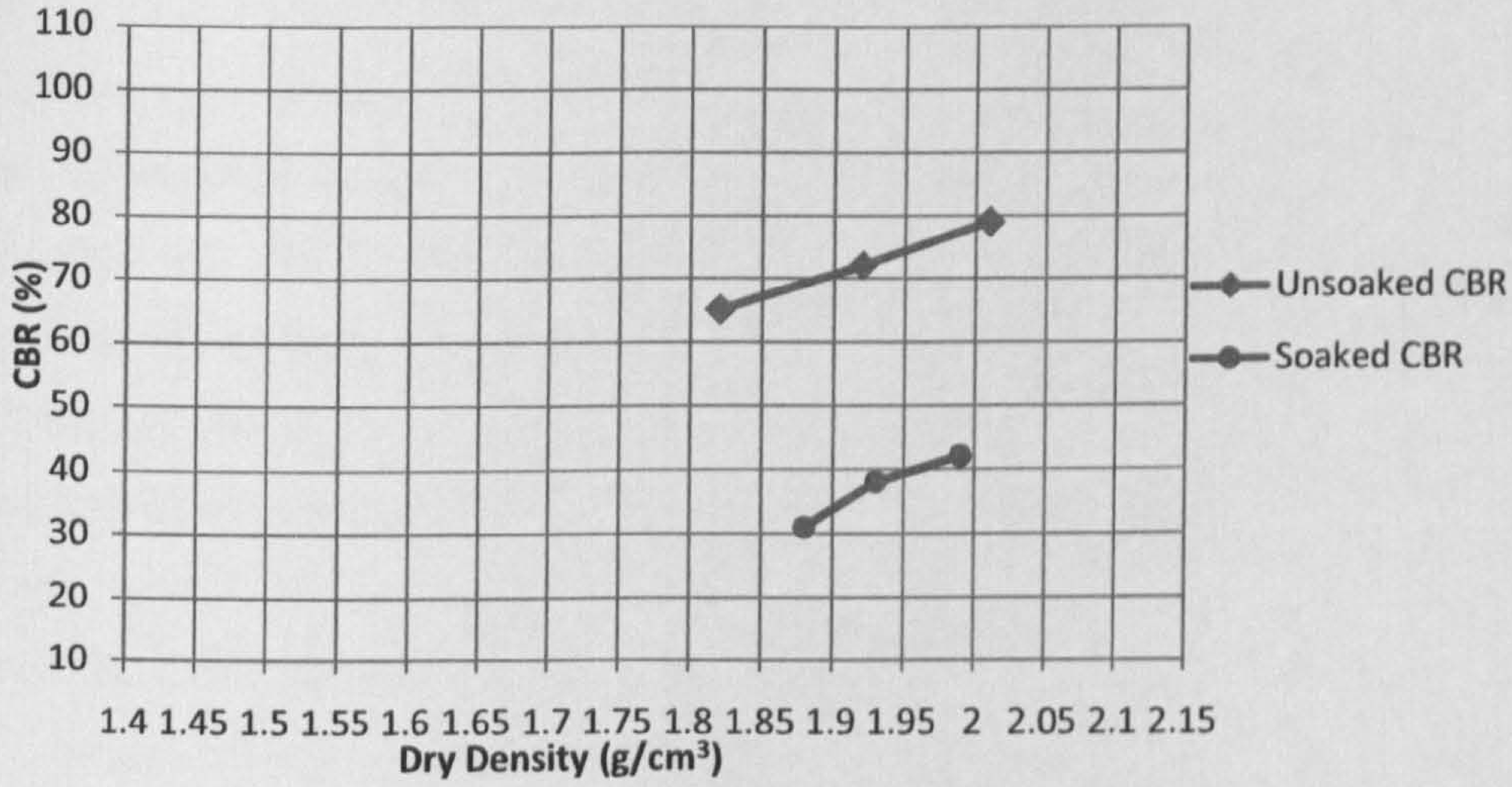


Figure 5.62 CBR-Dry Density relationship for 20%RCA+80%NA

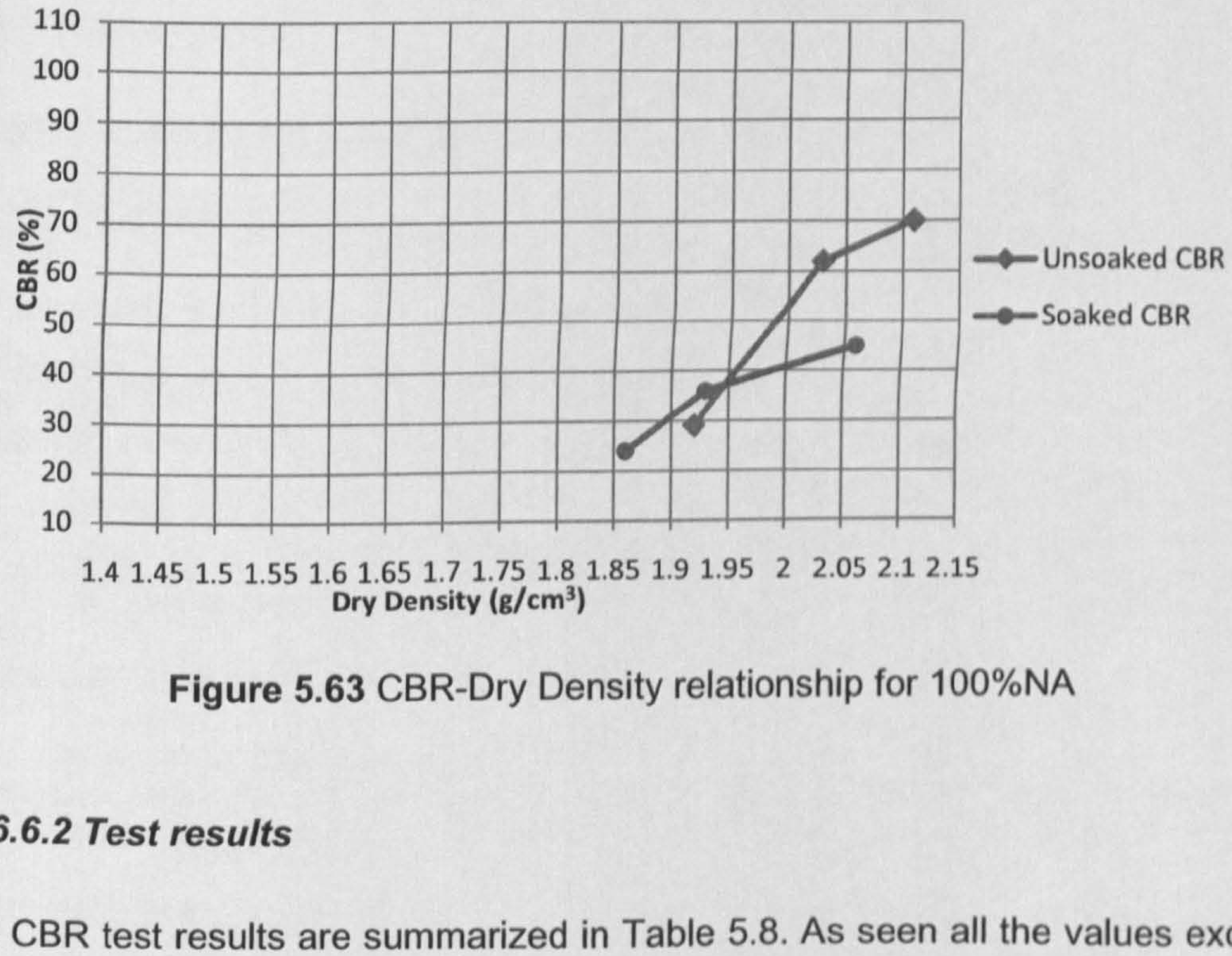


Figure 5.63 CBR-Dry Density relationship for 100%NA

5.16.6.2 Test results

The CBR test results are summarized in Table 5.8. As seen all the values exceed the minimum requirements (25% CBR) of the Iran Highway Asphalt Paving Code (2003).

Table 5.8 Unsoaked and soaked CBR values obtained from samples compacted with 65 blows at OMC

MATERIAL	Unsoaked CBR (%)	Soaked CBR (%)
100%RCA	98	32
20%NA+80%RCA	86	37
50%NA+50%RCA	80	40
80%NA+20%RCA	79	42
100%NA	70	45

Table 5.8 and Figures 5.59 to 5.63 illustrate that in an unsoaked condition, the 100%RCA material had the highest CBR value (98%). The CBR value decreased as the NA content increased, so that, the CBR of 100%NA was 70%. A possible explanation for this behavior could be the tendency of the rounded-shape NA particles to roll and slip over each other under the penetration of the CBR plunger. This behavior leads to a decrease in the overall bearing strength of the materials. Decreased CBR values in the unsoaked state can also be due to weak interlocking between particles for example due to high voids content. It is also seen from Table 5.8 that the CBR decrease due to soaking was much larger (66%) for the 100% RCA material than for the 100% NA material (25%). It is thought that this behavior could be attributed to the high water absorption (and consequent weakening) of the materials containing high proportions of RCA. So moisture content has more detrimental effect on RCA.

5.16.7 Soundness of aggregate using sodium/ magnesium sulphate tests

5.16.7.1 Method Statement

To determine the resistance of aggregates to disintegration by saturated sodium sulphate/ magnesium sulphate solution this test was performed in accordance with AASHTO T104-99(2003). The test was carried out by immersing the coarse and fine fractions separately in sodium sulphate and magnesium sulphate in five cycles. Table 5.10 lists the particle size ranges for the fine and coarse particles tested.

Table 5.10 Required sizes for soundness test

Coarse Fractions	Fine Fractions
2 ½"- 2" (63mm-50mm)	No.4-No.8 (4.75mm-2.36mm)
2"-1 ½"(50mm-37.5mm)	No.8-No.16(2.36mm-1.18mm)
1 ½"-1"(37.5mm-25mm)	No.16-No.30(1.18mm-0.59mm)
1"-3/4"(25mm-19.1mm)	No.30-No.50(0.59mm-0.297mm)
3/4"-1/2"(19.1mm-12.7mm)	No.50-No.100(0.297mm-0.149mm)
½"-3/8"(12.7mm-9.5mm)	
3/8"-No.4(9.5mm-4.75mm)	

Each cycle of testing consisted of 16-18 hrs of immersion in the solution, removing the aggregate sample from the solution, draining for 15 min and drying to constant mass at a temperature of 110℃. The weighted averag e soundness values were calculated from the percentage losses for each fraction, based on the percentage compositions of the mixes in Table 5.10 to give the soundness results for the NA and RCA.

5.16.7.2 Test results

Figure 5.64 and 5.65 present bar charts showing the results for sodium sulphate and magnesium sulphate soundness values respectively. Results are shown for the fine, coarse and whole particle size range for 100%RCA and 100%NA materials. The soundness for whole mix is sum of the soundness of fine and coarse portions aggregates.

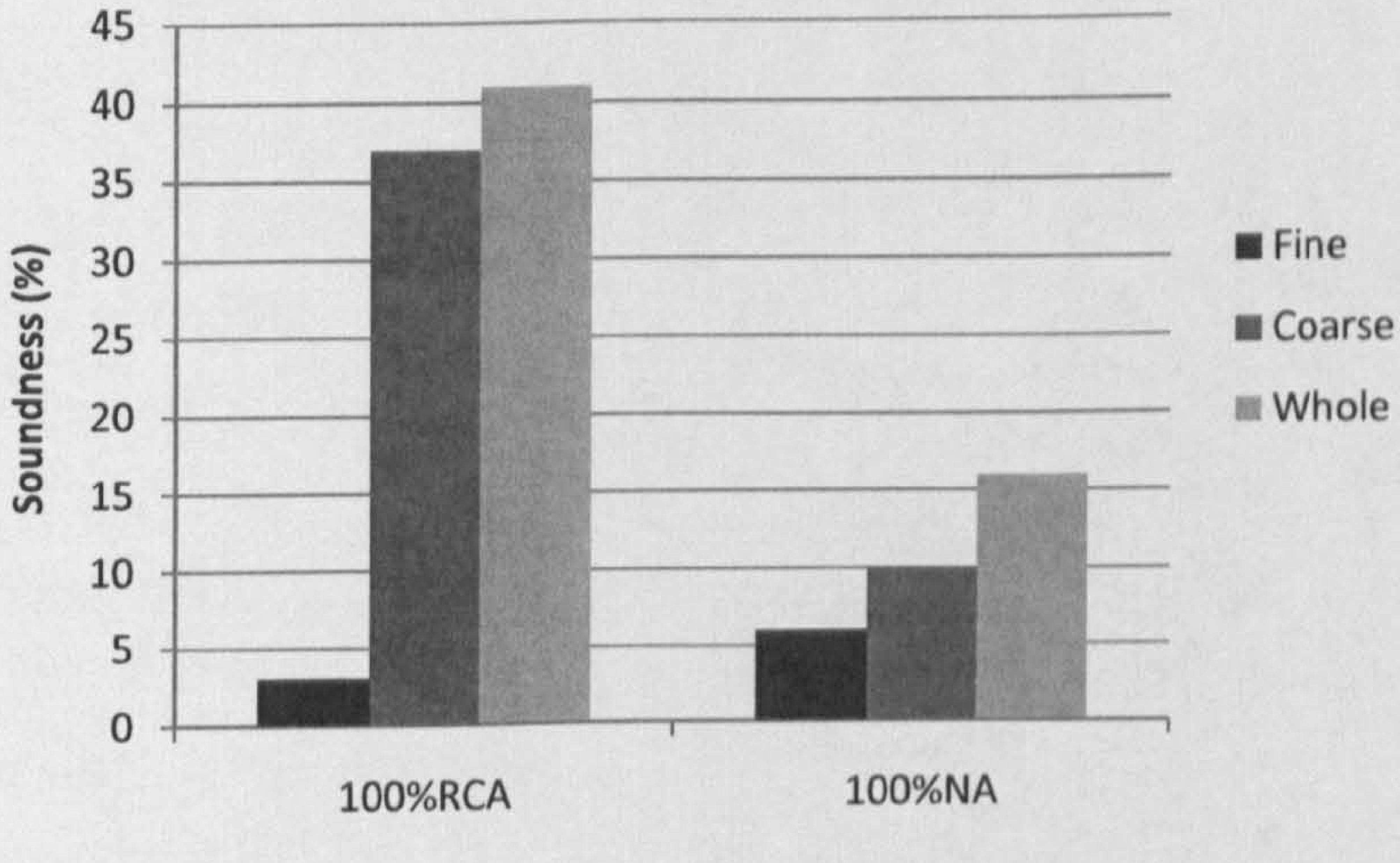


Figure 5.64 Comparison of results of soundness with sodium sulphate

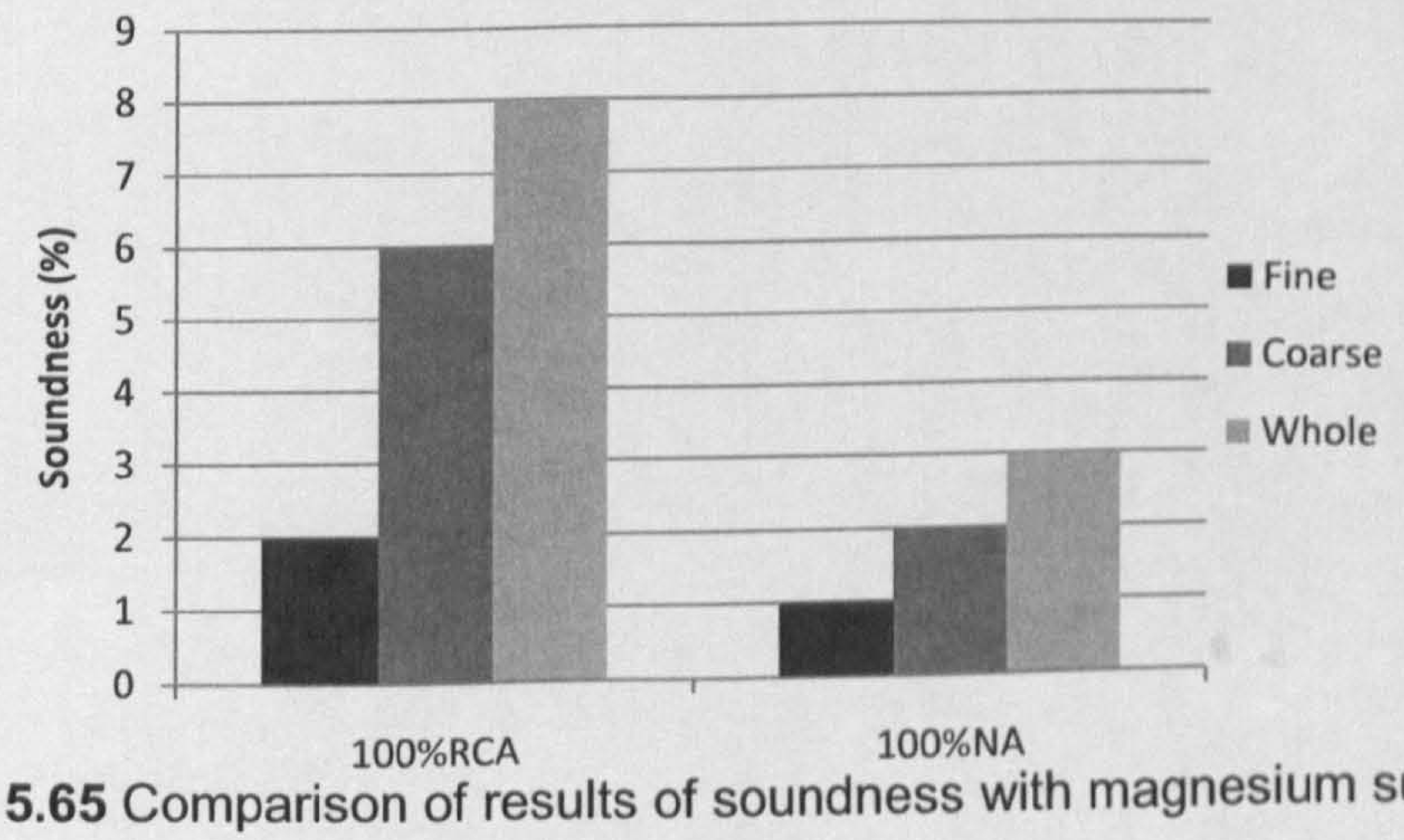


Figure 5.65 Comparison of results of soundness with magnesium sulphate

As seen in Figs 5.64 and 5.65, the 100%NA was more resistant to disintegration by sodium sulphate solution and magnesium sulphate with whole loss of 16% and 3% respectively. It means that 100%NA is more resistant than 100%RCA particles against internal expansive force, generated from the rehydration of the salt during re-immersion. It can be judged that the coarse fraction of 100%RCA had a freeze-thaw performance since it displayed losses 37% and 6% in sodium and magnesium sulphate solutions respectively. The reason for this behavior of the coarse fraction of 100%RCA can be attributed to the separation of aggregates from cement mortar due to the chemical reaction between sulphates and cement. In contrast, the fine fraction of 100%RCA performs better than those of NA in freeze-thaw conditions subject to weathering action. The loss values for the fine fraction of 100%NA in sodium sulphate test was higher than those of RCA hence reflecting their poor performance when subjected to freezing and thawing actions. Given the high values of sodium sulphate soundness losses observed in the 100%NA and 100%RCA, these materials do not fulfill the requirements of the Iran Highway Asphalt Paving Code, which sets a limit of 12% loss for unrestricted use. The reaction among the cement mortar attached to the RCA particles and sodium sulphate contributes to increasing the loss in the sodium sulphate test (Chini *et al.* 2001). However different combination of RCA and RAP will give more sound mixes. This is why additional soundness tests were performed using magnesium sulphate solutions for more reliability and to aid performance assessment later in Chapter 6.

Viewed from left to right, Fig. 5.66 shows a cracked particle of size 63mm-50mm, a split particle of initial size 63mm-50mm and a split and two cracked particles of size e 50mm-37.5mm for 100%NA immersed in sodium sulphate. The number of cracked and split particles in magnesium sulphate was very low and negligible.

Figure 5.66 Cracking and splitting of particles of 100%NA materials

5.16.8 Determination of Portland-Cement Content of RCA

The content of soluble silica in the RCA material, which was obtained from demolition of the Amir Kabir tunnel in Tehran (Iran), was measured using the procedure in ASTM C 1084-02 (2002). For this purpose the RCA samples were crushed to sizes finer than 4.75mm (No.4) sieve. Chemical analysis of representative sub-samples showed that soluble silica content in the crushed concrete was 3.12% (Figure 5.67). The cement percentage (C_s) was calculated by dividing the percentage of silica (SiO_2) in the RCA by the percentage of silica SiO_2 in fresh cement and multiplying by 100. The cement silica value was assumed 21.0% based on the recommendations in ASTM C 1084-02 (2002). As the dry density of concrete was 2.12 g/cm^3, the cement content of concrete according to this analysis was obtained as 315 Kg/m^3. There is no correlation between cement content and properties of RCA.

Figure 5.67 Triple analysis of solube silica in RCA

5.17 Mixes of Reclaimed Asphalt Pavement (RAP) with Recycled Concrete Aggregates (RCA)

In this study, a portion of reclaimed asphalt pavement was replaced by recycled concrete aggregate and investigates as a subbase material. The following mix combinations were studied: (i) 100%RAP as control mix, (ii) 80%RAP+20%RCA, (iii) 50%RAP+50%RCA, (iv) 20%RAP+80%RCA and (v) 100%RCA. The particle size distributions of these mixtures were within standard Type-2 subbase material grading curves which are illustrated in Figures 5.68 to 5.71. The grading curve of RCA was previously shown in Figure 5.47.

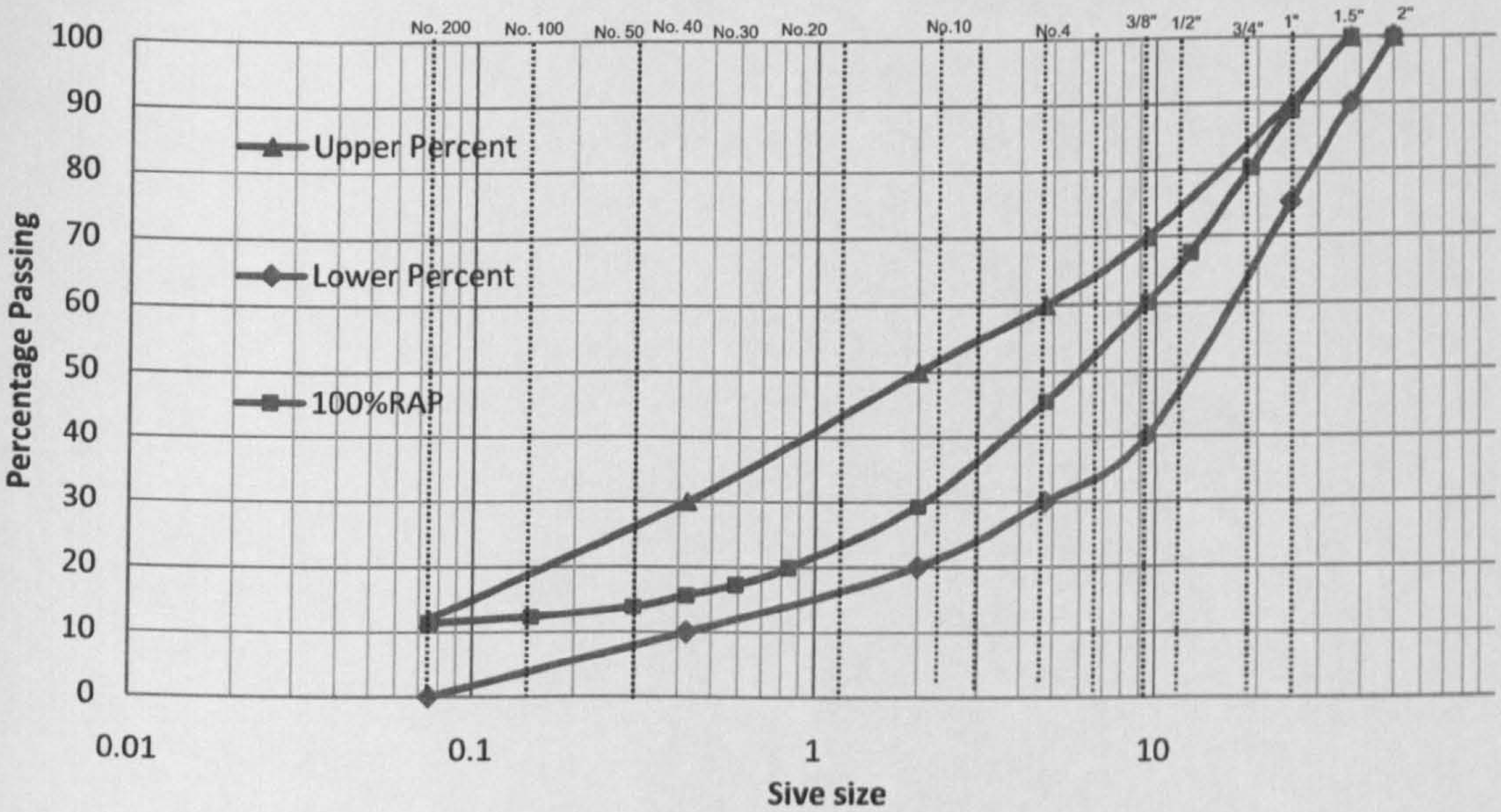


Figure 5.68 Grading curve of 100%RAP

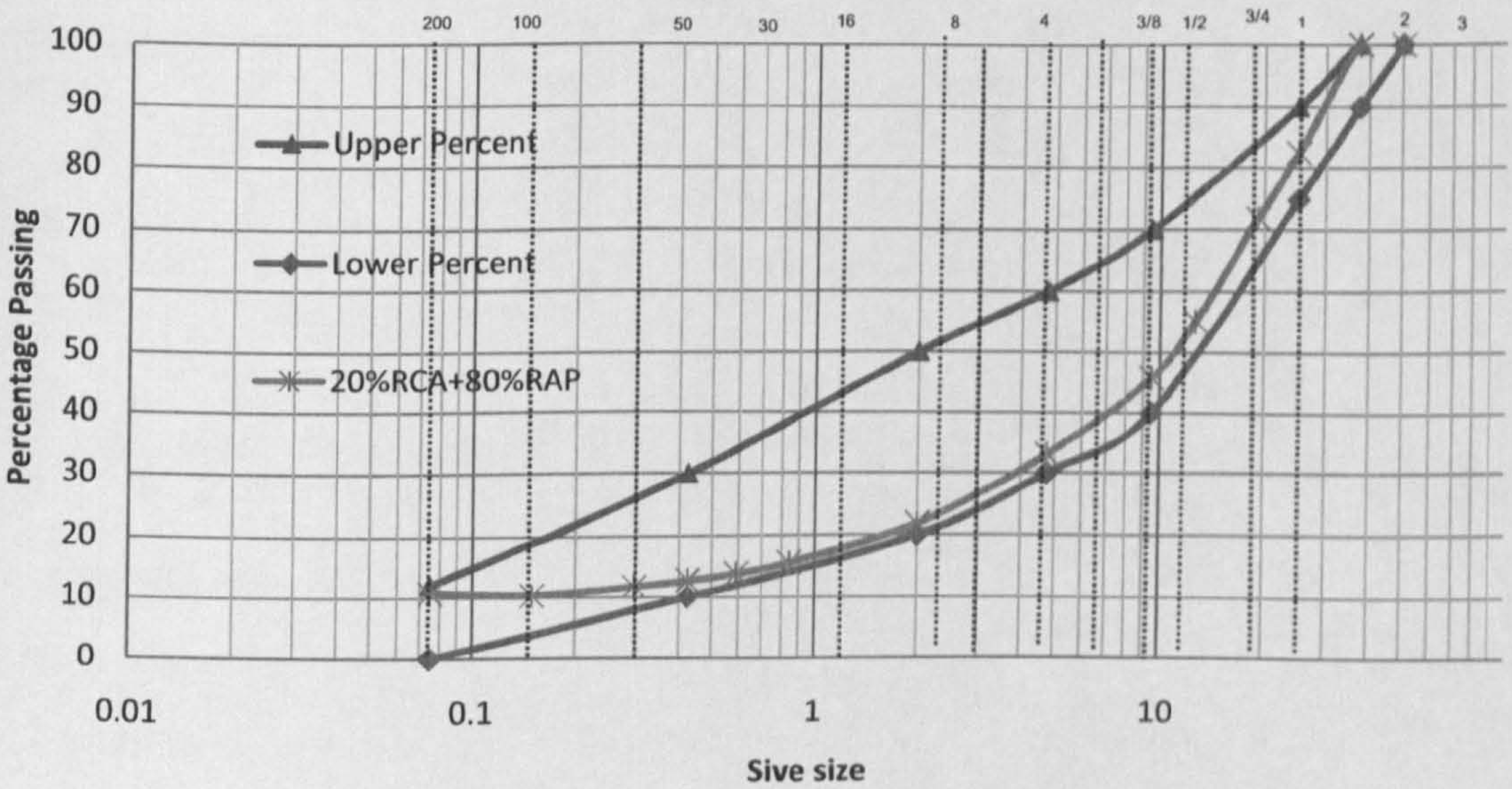


Figure 5.69 Grading curve of 80%RAP+20%RCA

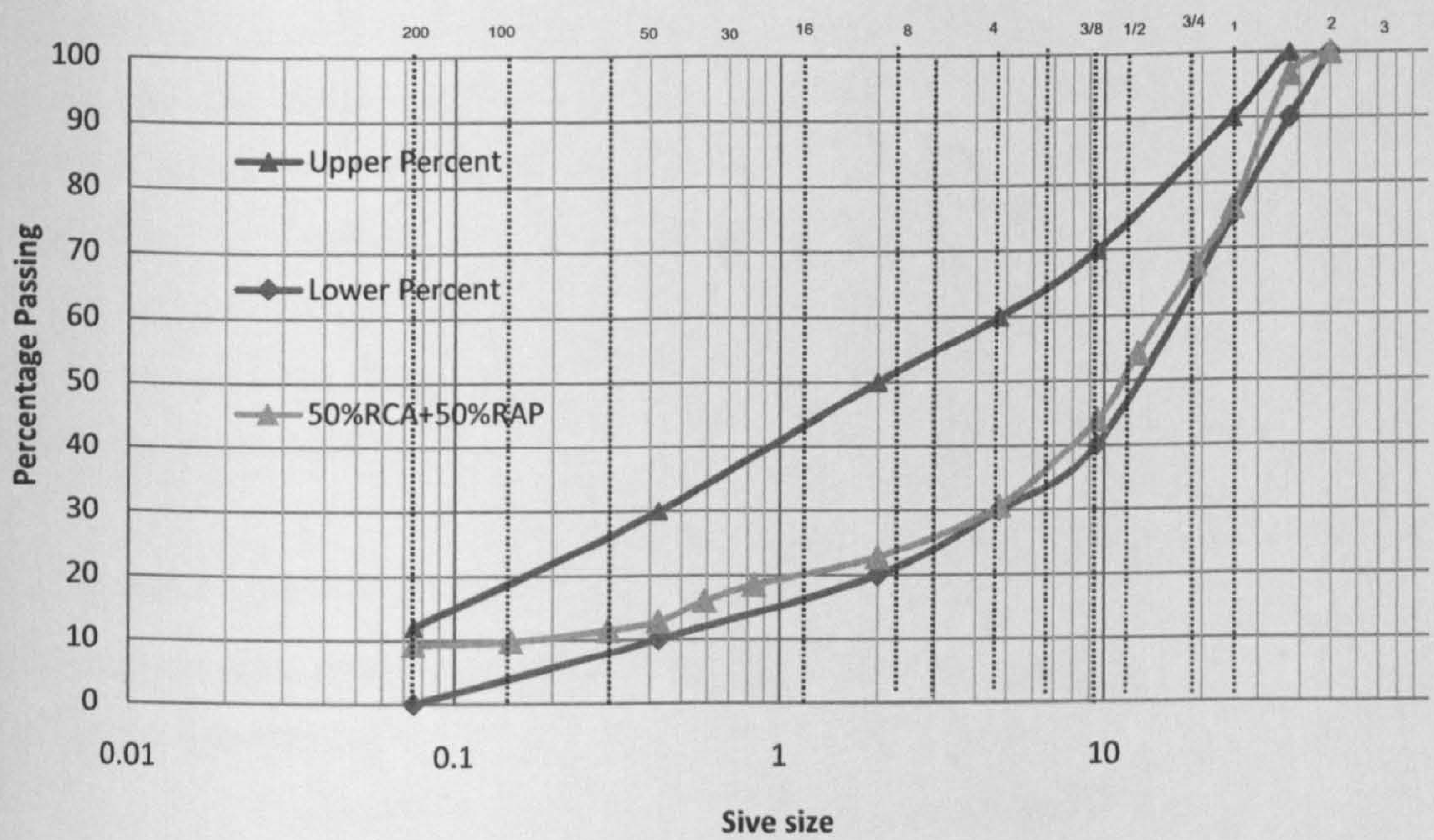


Figure 5.70 Grading curve of 50%RAP+50%RCA

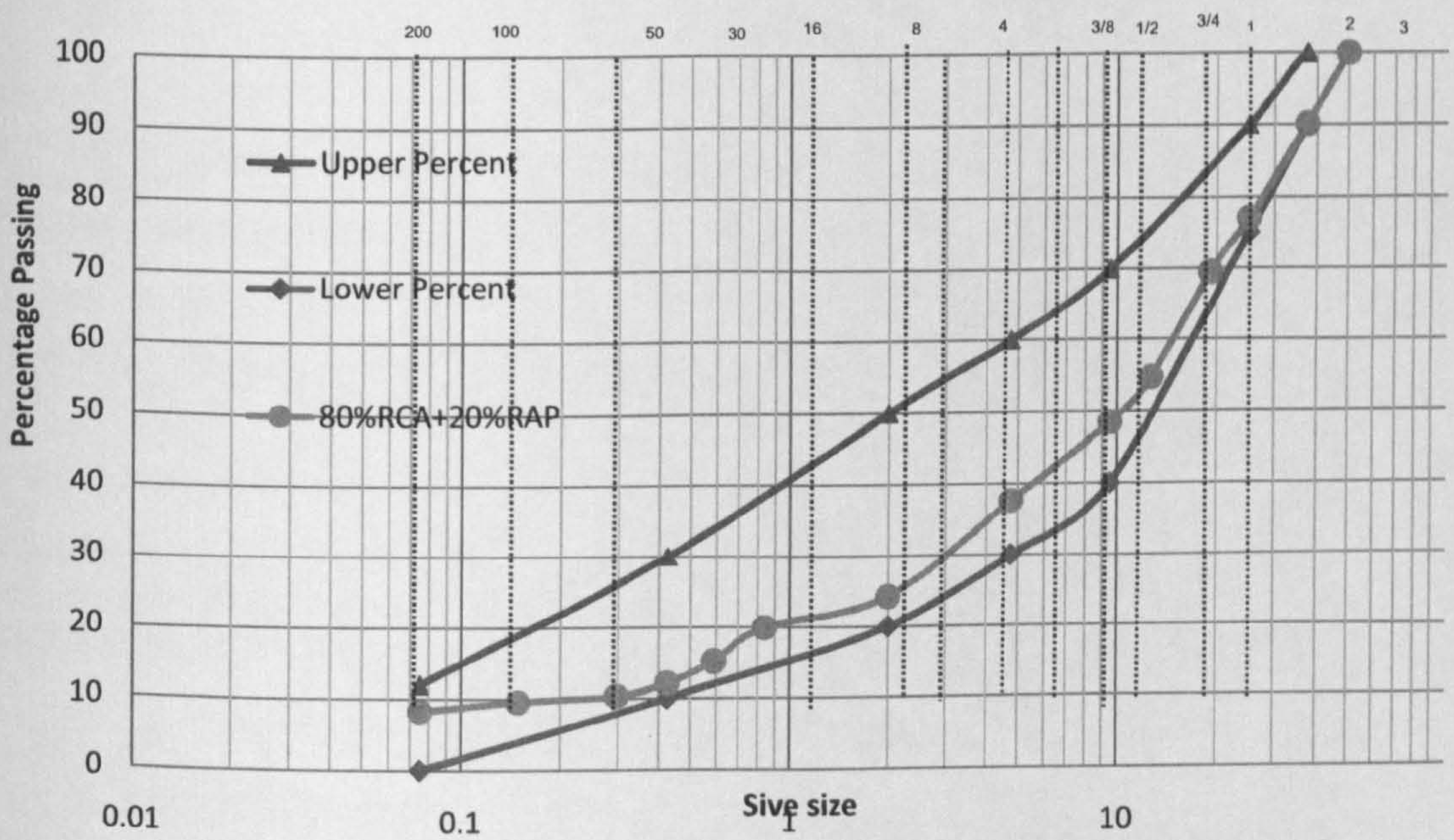


Figure 5.71 Grading curve of 20%RAP+80%RCA

5.17.1 Consistency limits (Atterberg limits) tests

Consistency limit tests were carried out on the materials following the procedures presented earlier in Section 5.16.1. It was observed that, at all moisture contents in the Casagrande dish method of liquid limit (LL) test, the 100%RAP material tended to slide on the surface of the cup rather than flow towards the grove. This implied that the Casagrande method was not applicable to this material and so the

LL could not be determined. This behavior was also exhibited, albeit to a lesser extent, by the other mixtures because the presence of bitumen within RAP inhibited absorption of water thereby making it difficult to increase moisture content. In the determination of the plastic limit (PL) according to AASHTO T 90-00 the test portions crumbled in the first step of rolling, meaning that the PL could not be determined. Therefore it was concluded that all the five RAP and RCA mixes were non-plastic.

5.17.2 Sand equivalent value (SE)

Based upon the method already described in Section 5.16.2, the following average SE values were found:

Mixture	Sand equivalent (SE) %
100%RCA	76%
50%RCA+50%RAP	82%
80%RCA+20%RAP, 20%RCA +80%RAP and 100%RAP	81%

As seen above, apart from 100%RCA the other mixes were clean with less undesirable fine dust. Undesirable dust can coat aggregate particles and prevent proper subbase particles bonding. Finally, all mixes were found to satisfy the minimum requirement of SE=30% specified in the Iran Highway Asphalt Paving Code (2003).

5.17.3 Los Angeles Abrasion (LAA) tests

5.17.3.1 Method Statement

From the 100%RCA and 50%RCA+50%RAP materials, 10 kg samples were sieved off to achieve grading 2 specified in the ASTM C535-03. For the other mixtures (80%RCA+20%RAP, 20%RCA+80%RAP and 100%RAP) LAA test samples were prepared to grading designation 3 of ASTM C535-03. This was necessary due to the sizes of the available aggregate particles in the materials. The samples were then subjected to LAA testing following the procedure given in ASTM C535-03. The test required each sample to be placed with 12 standard steel spheres inside a metal drum, which was then rotated 1000 times at a rate of 30 rpm. The weight loss of the material (in percent) due to abrasion and impact effects was calculated from the original total mass and the final mass measured after washing the samples over a No. 12 sieve followed by and oven drying at 110 ℃.

5.17.3.2 Test results

The results of the LAA losses for each of the materials were found to be generally consistent with the values reported by other researchers (Blankenagel, 2005, Blankenagel and Geuthrie 2006).The 100%RCA material showed a weight loss of 31%. It was also found that weight losses in LAA tests decreased as the level of replacement of RAP with RCA increased. The 80%RCA+20%RAP and 50%RCA+50%RAP had weight losses of 29% while the 20%RCA+80%RAP and 100%RAP had losses of 28%. From the results of the test, it is clearly indicated that replacing RCA with RAP decreased fine produced from stripping of cement paste (Blankenagel, 2005). The mechanism can be explained by the fact that RAP particles were more flexible against impacts by steel spheres because of the elasticity of the constituent bitumen. An advantage of less particle breakdown is that the percentage of fine particles is decreased. Consequently the change in gradation of the material, if used as a highway pavement layer, provides enhanced stiffness to support in-service loads. Additionally, such a gradation change results in decreased water absorption potential and improved drainage properties of unbound subbase made with the mix of RCA and RAP. As previously shown in Figure 5.57 and later in Figure 5.83, the degradation of RAP was less than that of RCA.

5.17.4 Water absorption

The procedure used for the determination of water absorption has already been described in Section 5.16.4.1. The test results for each particle sizes ranges of the mixes of RCA and RAP are summarized in Figures 5.72- 5.80.

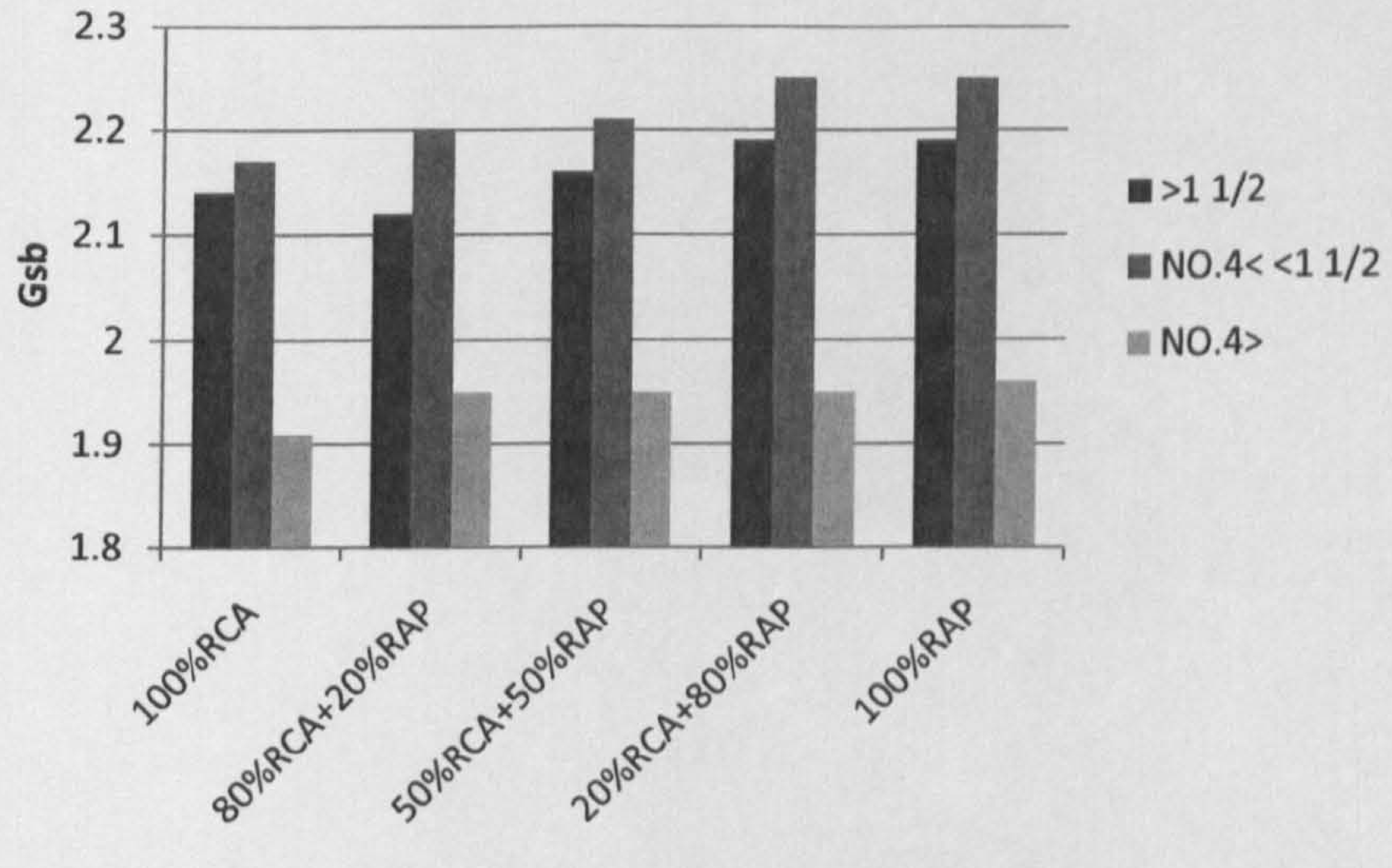


Figure 5.72 Bulk specific gravity (G_{sb}) of RCA and RAP mixes

The average value of bulk specific gravity for whole mix including all fractions was calculated from Equation 5.5 and the results are illustrated in Figure 5.73.

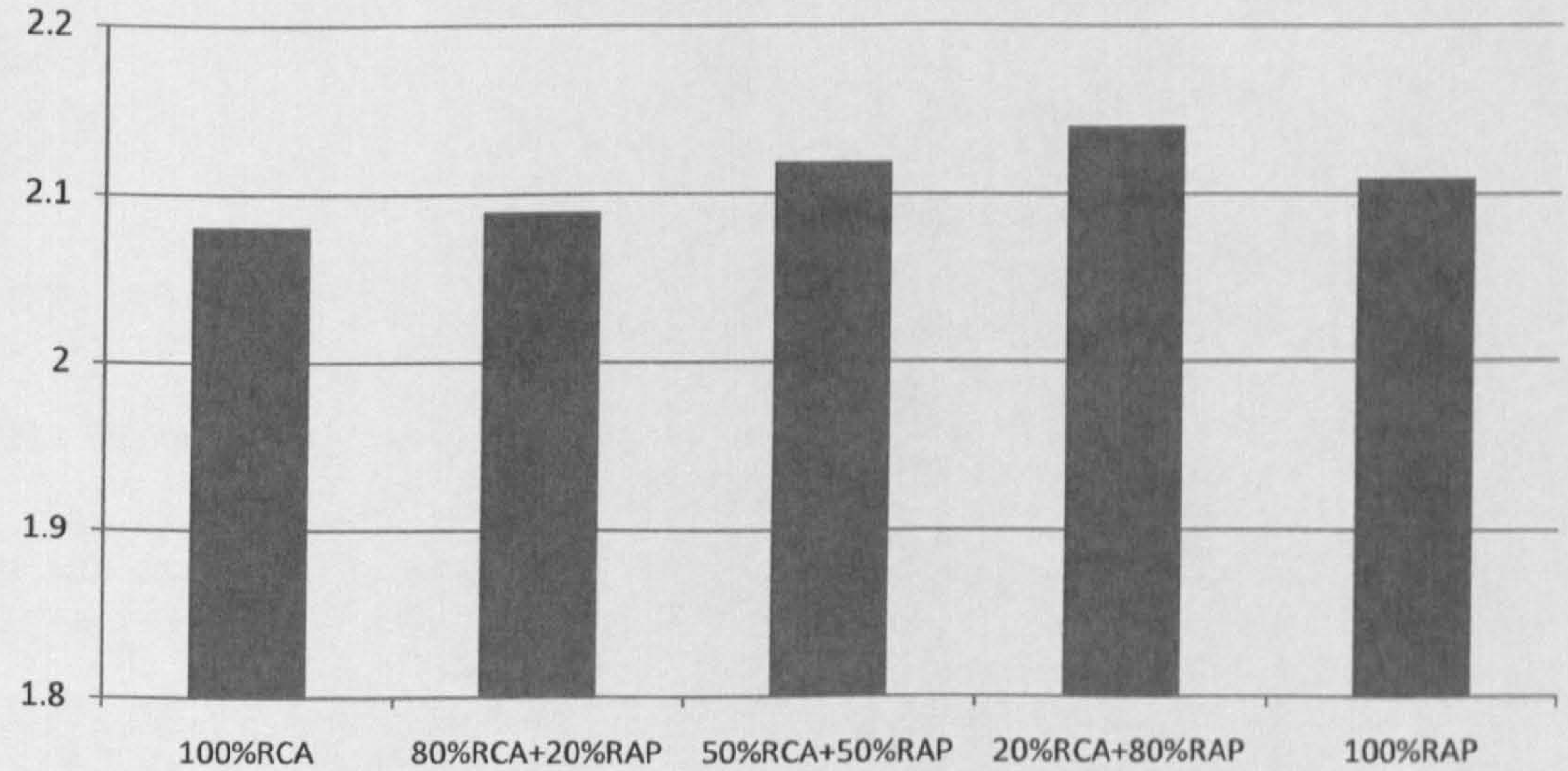


Figure 5.73 Average value of G_{sb} for mixes of RCA and RAP

The quantity of the coarse fraction lying in the range 4.75-37.5 mm in the mixes of 50%RAP+50%RCA and 80%RAP+20%RCA is more than twice that of the fine fraction (Figures 5.69 to 5.70). As was shown in Figure 5.72, these coarse particles have the highest G_{sb} among all fractions and hence the average value of G_{sb} for these two mixes is higher than for the other mixes. (Figure 5.73)

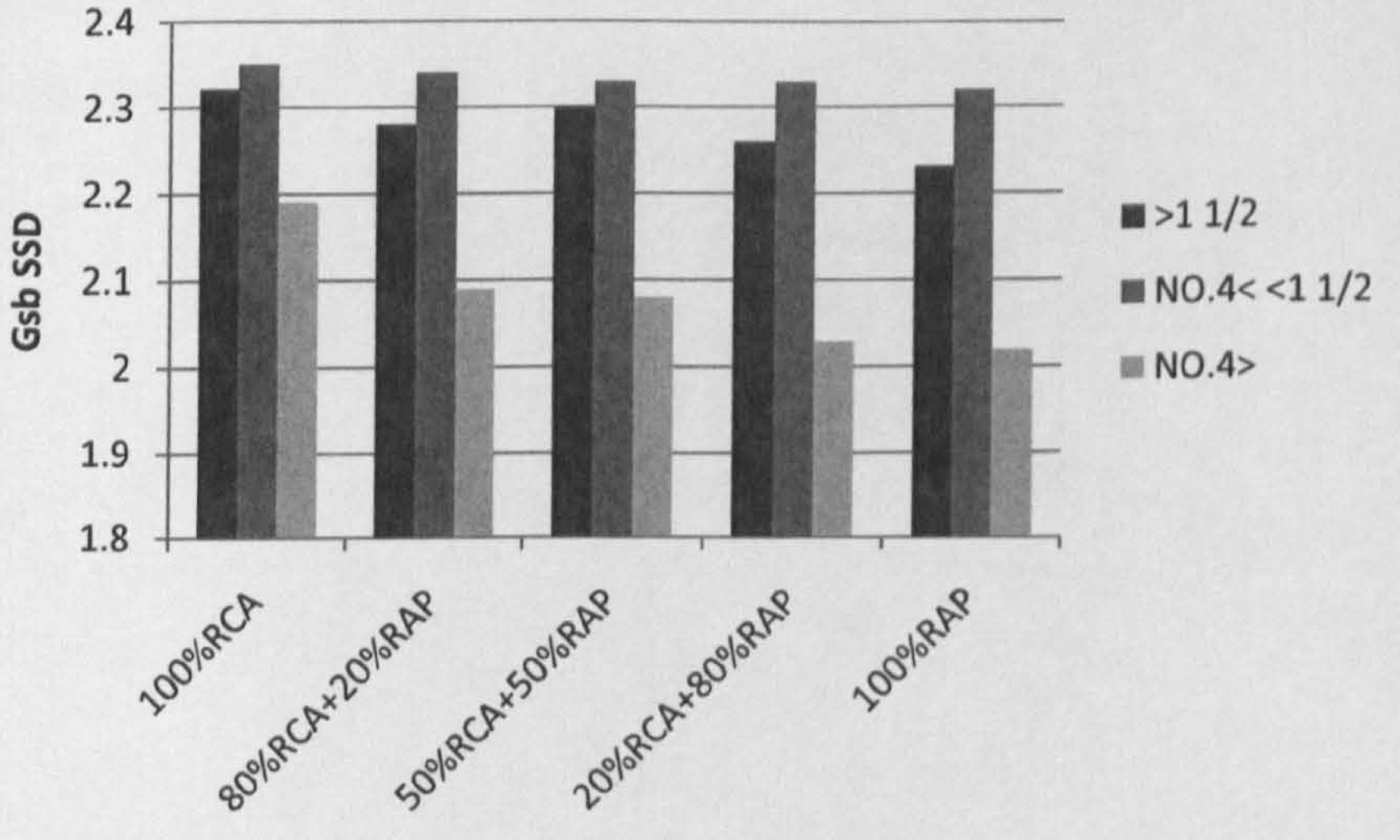


Figure 5.74 G_{sb} SSD of RCA and RAP mixes

The average value of G_{sb} SSD for whole mix including all fractions was calculated from Equation 5.6 and the results are illustrated in Figure 5.75.

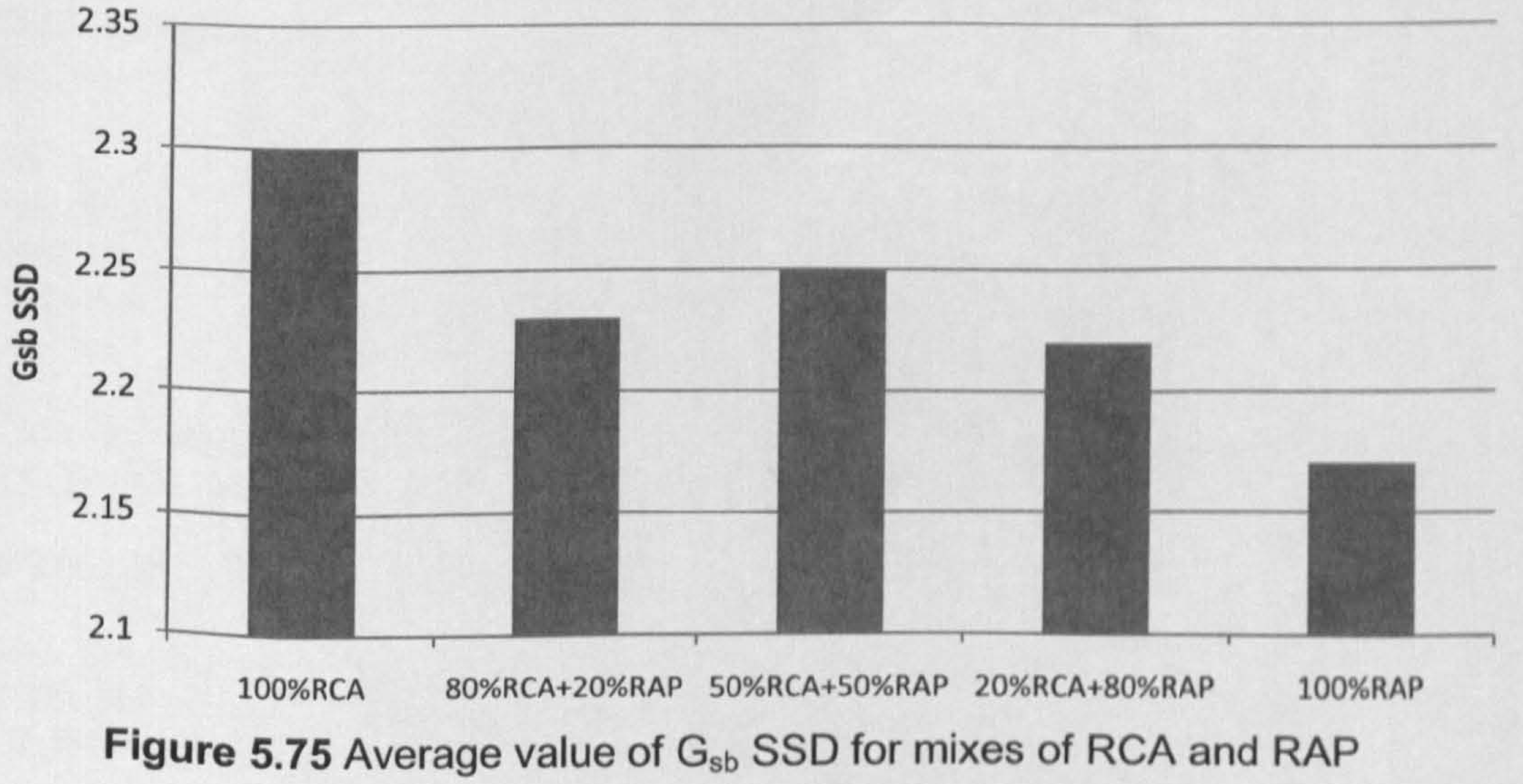


Figure 5.75 Average value of G_{sb} SSD for mixes of RCA and RAP

Figure 5.75 indicates that G_{sb} SSD of 50%RAP+50%RCA was more than that of 20%RAP+80%RCA. This was because of the higher G_{sb} SSD of 50%RAP+50%RCA in the coarse particles larger than 37.5mm. As shown in Figure 5.76 the number of RCA coarse particles in the 50%RAP+50%RCA mix was more than RAP coarse particles and this led to more water being absorbed during 15 hours of immersion. For this reason the G_{sb} SSD of the 50%RAP+50%RCA mix was found to be greater than that of the 20%RAP+80%RCA mix. Based on the foregoing findings, it can be concluded that in practice, in order to control the water absorption rates account must be taken of the precise proportions of RCA and RAP present in the three required particle size ranges (fine, medium and coarse) of the stockpile material.

Figure 5.76 Water absorption specimen for the coarse fraction of 50%RAP+50%RCA

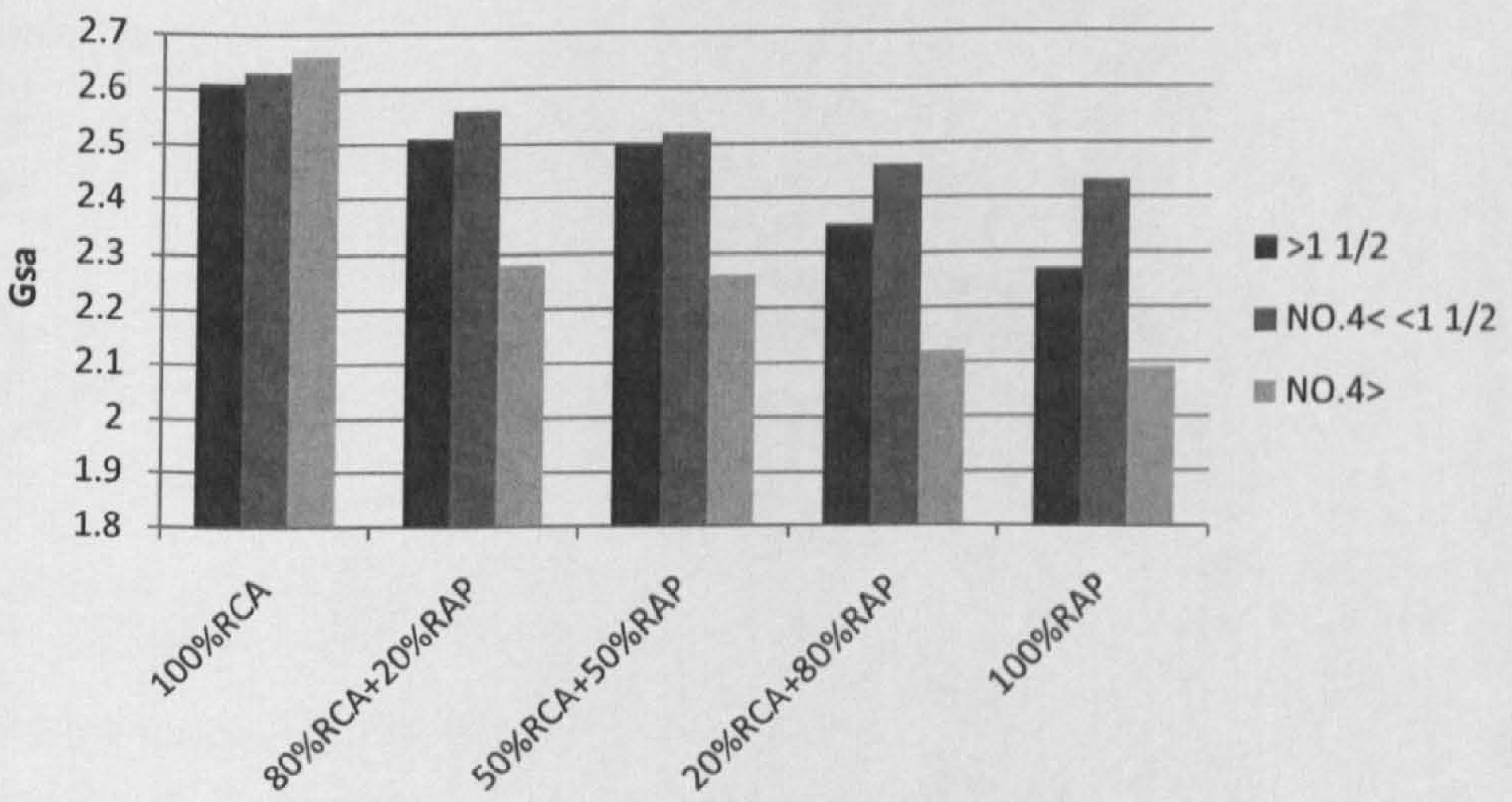


Figure 5.77 G_{sa} of RCA and RAP mixes

The average value of G_{sa} for whole mixes including all fractions was calculated from Equation 5.7 and the results are illustrated in Figure 5.78.

Figure 5.78 Average value of Gsa for mixes of RCA and RAP

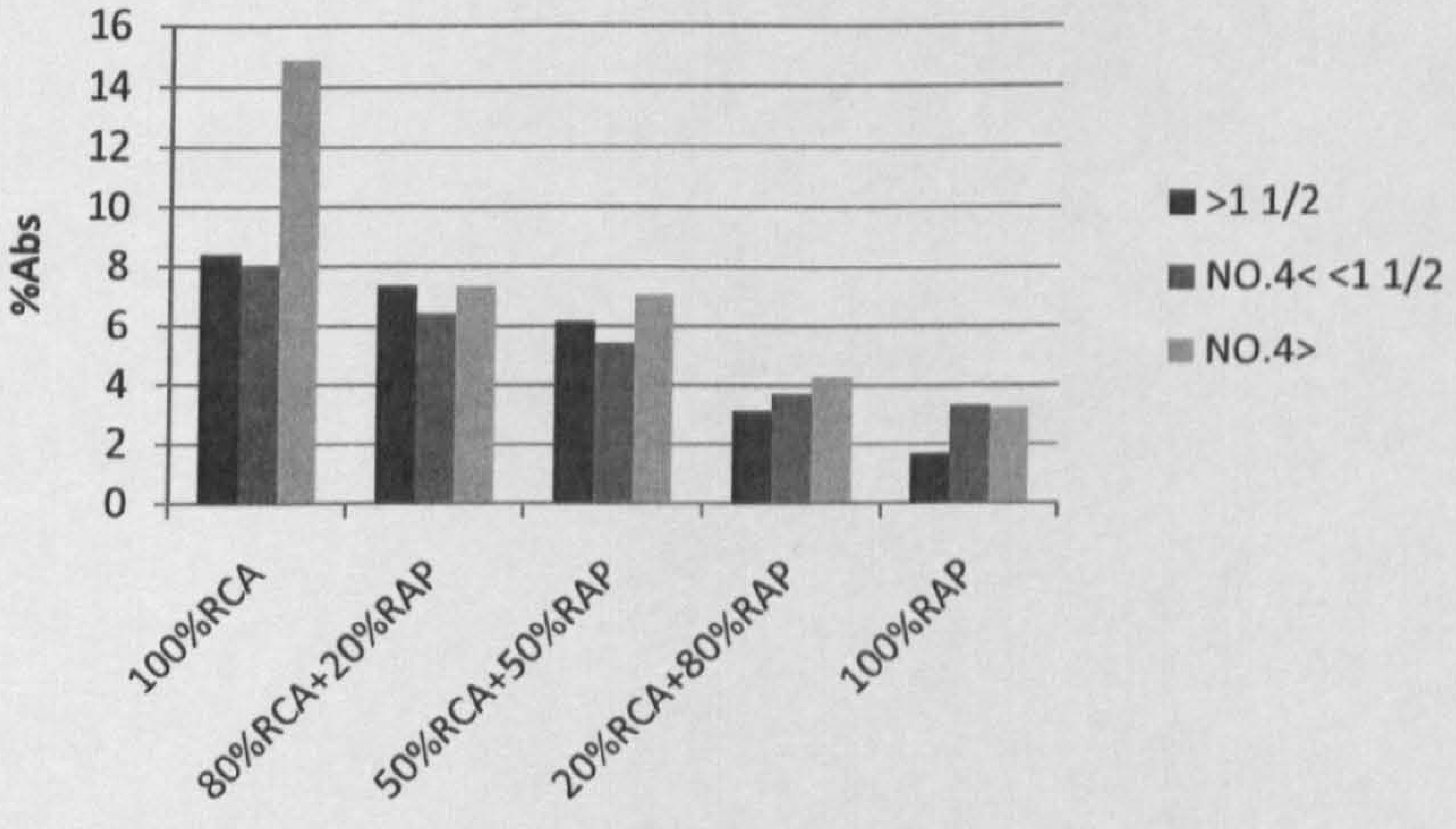


Figure 5.79 Water absorption (%Abs) of RCA and RAP mixes

The average value of water absorption for whole mixes including all fractions was calculated from Equation 5.8 and the results are illustrated in Figure 5.80.

Figure 5.80 Average value of %Abs for mixes of RCA and RAP

The difference between G_{sa} and G_{sb} for *100%RCA suggests that the volume of* permeable pores in this material is more than that in NA and RAP. A possible reason for this may be the separation of cement from particles during crushing of concrete to extract RCA with the consequence that the water absorption potential is increased.

As the density of cement mortar (approximately 1.0–1.6 Mg/m^3) (Tam and Le 2007a) is less than that of stone particles of about 2.60 Mg/m^3, the smaller the specific gravity, the higher the cement mortar adhering to the RCA. Due to the high porosity and low density of the cement paste coating RCA particles, the specific gravities of RCA *were lower than those of* NA.

As seen *in* Figs. *5.55 and 5.80, the RCA materials had the highest water absorption, followed by* NA *and* RAP. Any existing cement mortar adhering to the *RCA stone particles make the material weaker, more porous and results in a* cracked layer (Tam *et al.* 2007b). *This decreases the specific gravity of the RCA* relative to NA. It seems that these porous spaces besides unhydrated cement *led* to increase in the water absorption rate. Furthermore, specific gravities of coarse aggregate are larger than those of fine aggregate, implying that a higher amount of cement mortar was attached to the fine aggregate. This behavior affects the properties of RCA in the sense that the water absorption rate of the fine fraction is increased as supported by the findings by Tam and Le (2007a).

The calculations showed that the average absorption values for 100%RCA, 100%NA and 100%RAP were *10.08%, 4.45% and 3.30%, respectively*. Obviously bitumen coated particles in RAP limited the absorption of water *by the particles*, thereby decreasing the water absorption potential of the mix. In the same way weather affects road surfacing materials in the long term, the bond between the coarse aggregates of RAP and bitumen was observed to weaken resulting in the particles separating after 15 hours of submersion in water and drying to constant mass (Figure 5.81).

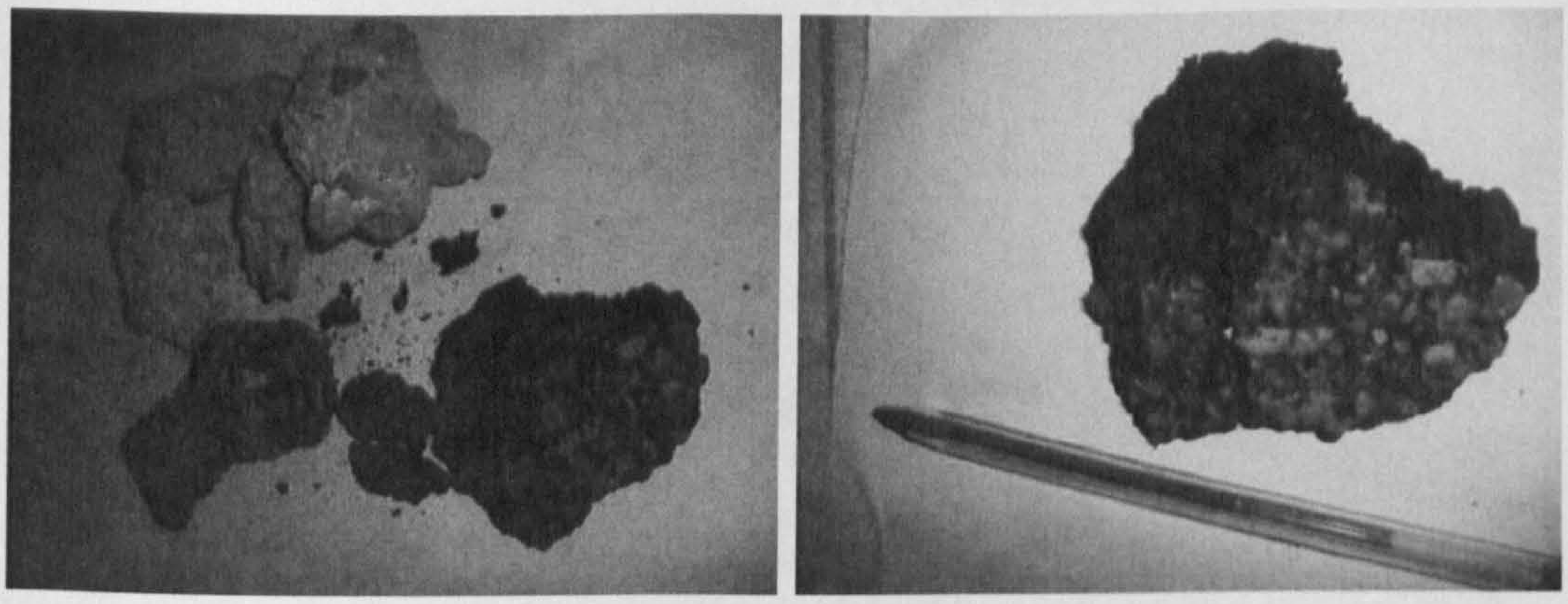


Figure 5.81 Cracked and split RAP aggregates due to submersion in water followed by drying to constant mass

5.17.5 Compaction test

On completion of the compaction test as explained in Section 5.16.5.1, Figure 5.82 illustrates dry density versus moisture content curves for each mixture.

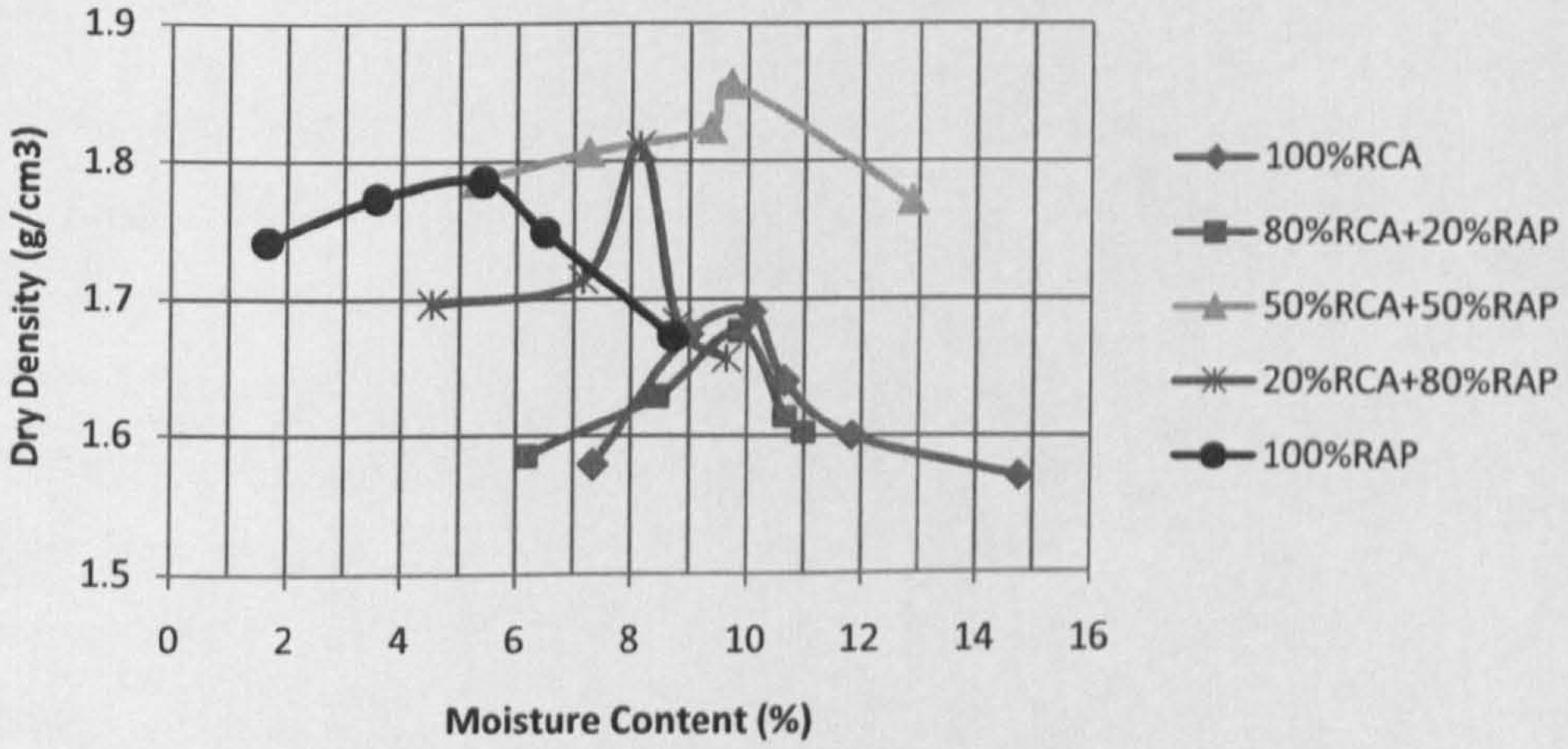


Figure 5.82 Density-Moisture curves

From Fig. 5.82 the OMC and MDD of the mixtures were obtained and listed in Table 5.11.

Table 5.11 Summary of OMC and MDD for six subbase materials

MATERIAL	OMC (%)	MDD (g/cm^3)
100%RCA	10.09	1.69
80%RCA+20%RAP	9.86	1.68
50%RCA+50%RAP	9.71	1.86
20%RCA+80%RAP	8.15	1.81
100%RAP	5.41	1.79

As seen in Table 5.11 the 50%RCA+50%RAP mix had the highest MDD. It is also seen that there was a general decrease of OMC with increasing RAP content in the mixes. This was due to the low water absorption of bitumen coated particles as compared to RCA. Furthermore, the results also show a general trend of increase of MDD with increasing RAP content, a result which is associated with the bulk specific gravities (G_{sb}) of aggregates passing the 4.75mm sieve obtained from specific gravity and absorption test (AASHTO T 85-91, 2007 and AASHTO T 84-00, 2007) . As previously discussed the G_{sb} of RAP was more than that of RCA so that the greater the percentage of RAP in a mix the higher is the G_{sb}. Also, as found from the specific gravity and absorption tests, the G_{sb} of the 50%RCA+50%RAP was 2.12 and exceeded values for the other mixes. As a consequence, the MDD of 50%RCA+50%RAP was revealed to be greater than those of the other mixes.

To evaluate the crushing of the aggregates and changes in their sizes under roller passes, Figs. 5.57and 5.83 were plotted from the results of sieve analyses performed on aggregates passing the 3/4" (19mm) sieve before and after the compaction test. Less degradation of RAP in LAA test was confirmed by Figures 5.57 and 5.83. RCA particles showed significant degradation by more than 20%, based on the mass of particles retained on No.4 sieve. In contrast, RAP is much less susceptible to degradation under roller passes, in real pavements. The above observations, coupled with the results from LAA tests suggest that a high percentage of RAP in an unbound subbase enhances the toughness of the pavement.

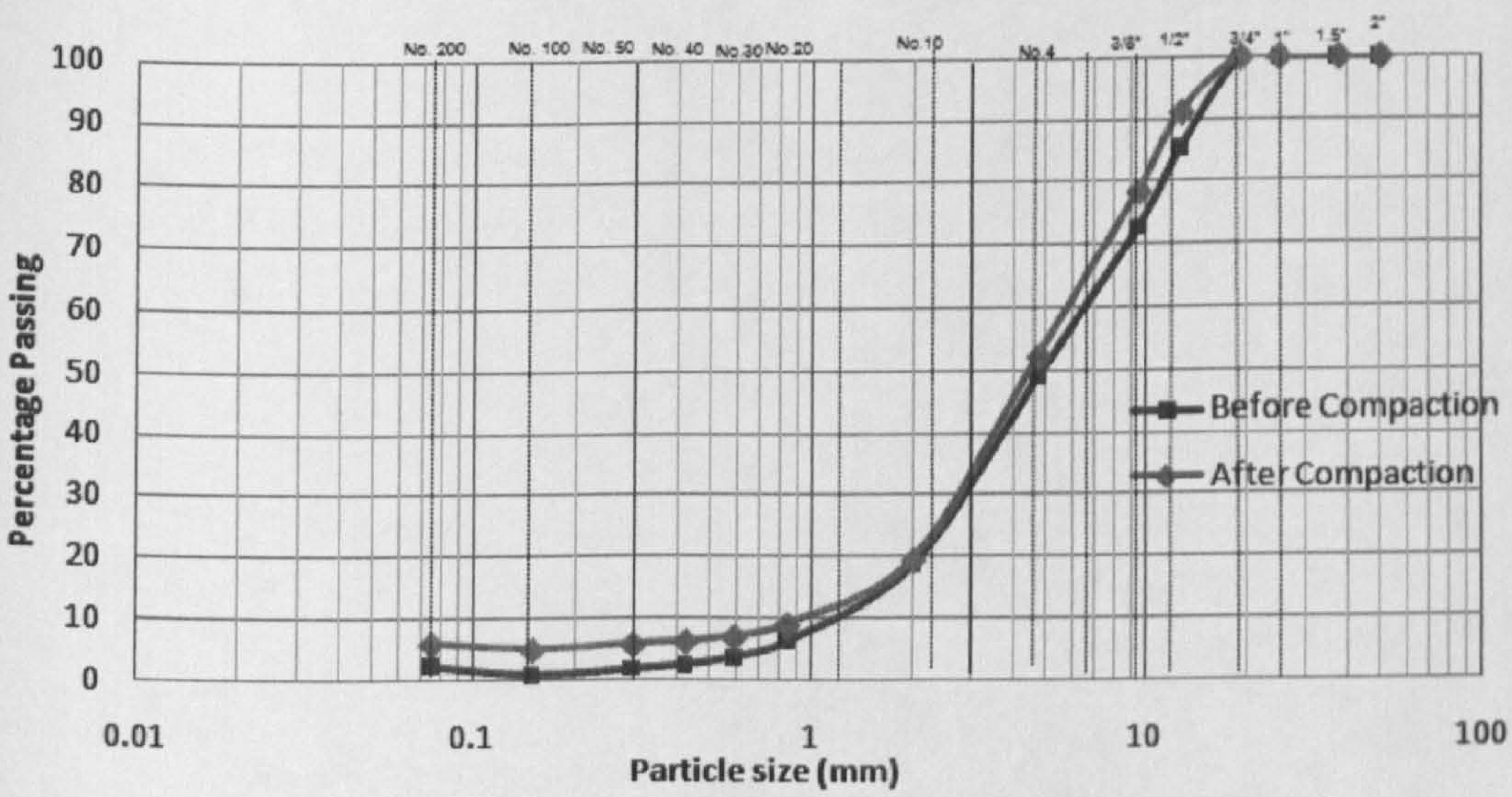


Figure 5.83 Sieve analysis of RAP compacted sample

5.17.6 California bearing ratio test (CBR)

From the test procedure described in Section 5.16.6.1, the CBR-Dry density relationships obtained for RAP and RCA are plotted in Figs. 5.84 to 5.87 for three compaction efforts. To allow quick comparison with 100%RCA the reader is referred to a previous Fig. 5.59.

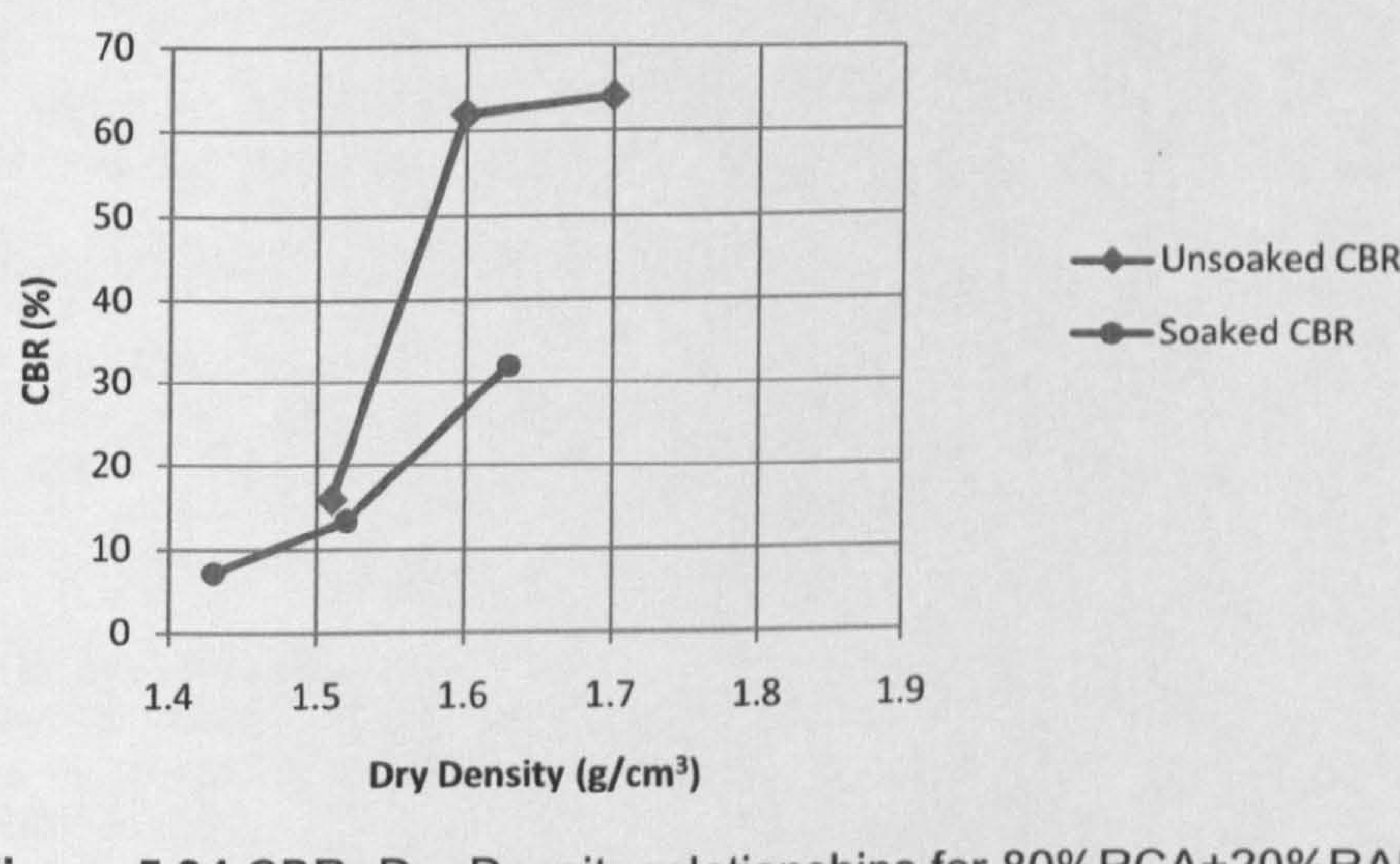


Figure 5.84 CBR- Dry Density relationships for 80%RCA+20%RAP

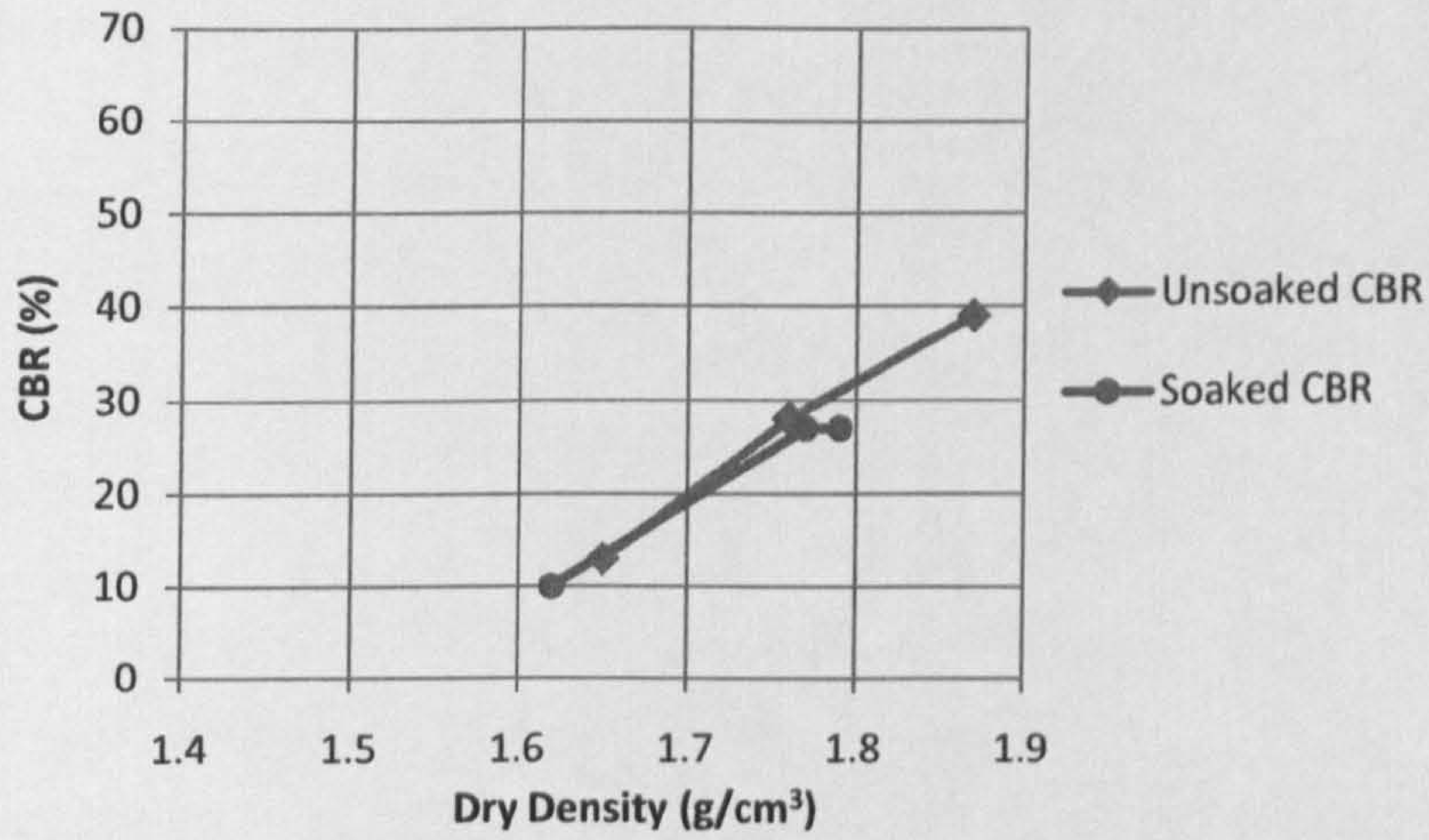


Figure 5.85 CBR- Dry Density relationships for 50%RCA+50%RAP

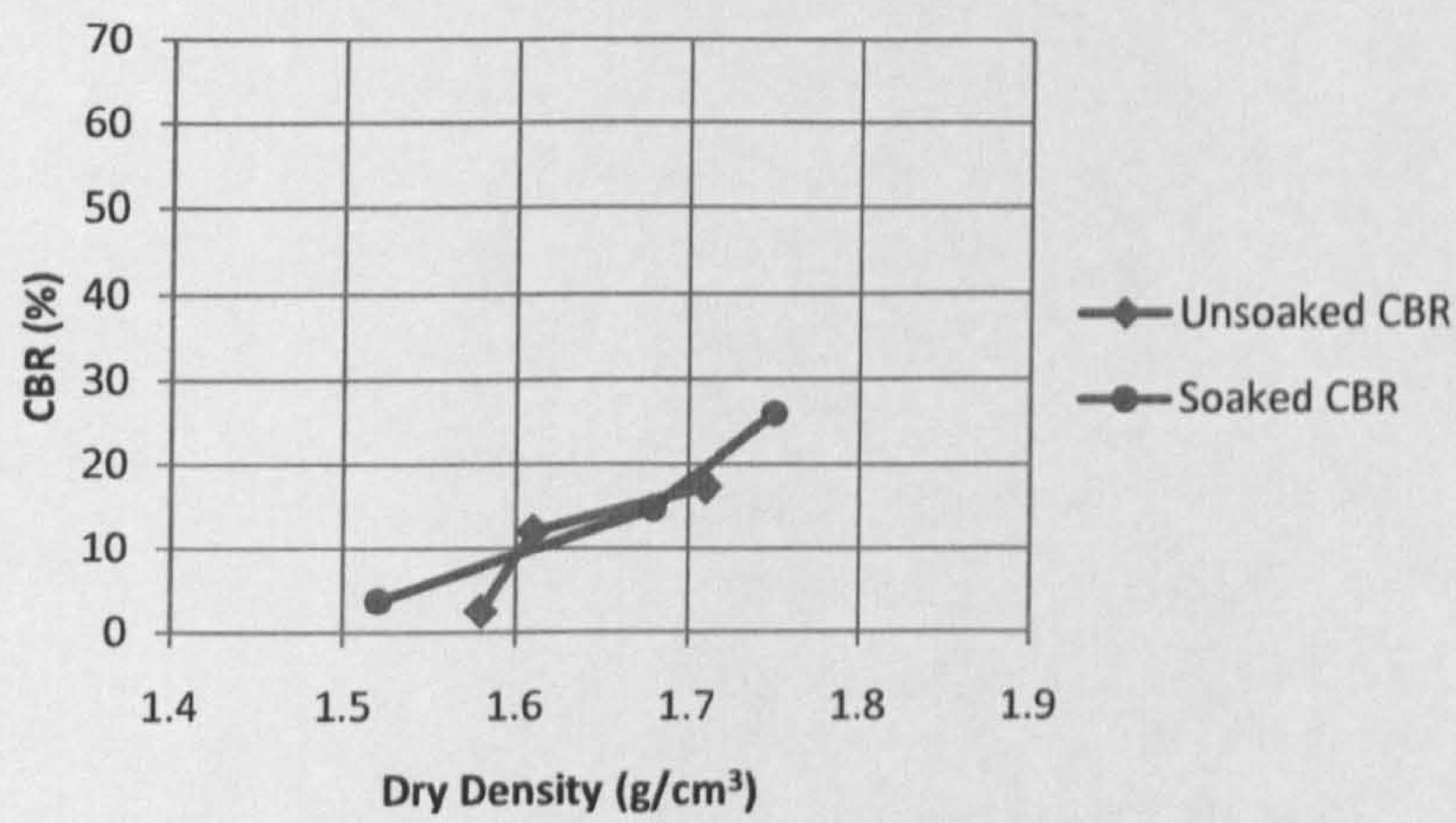


Figure 5.86 CBR- Dry Density relationships for 20%RCA+80%RAP

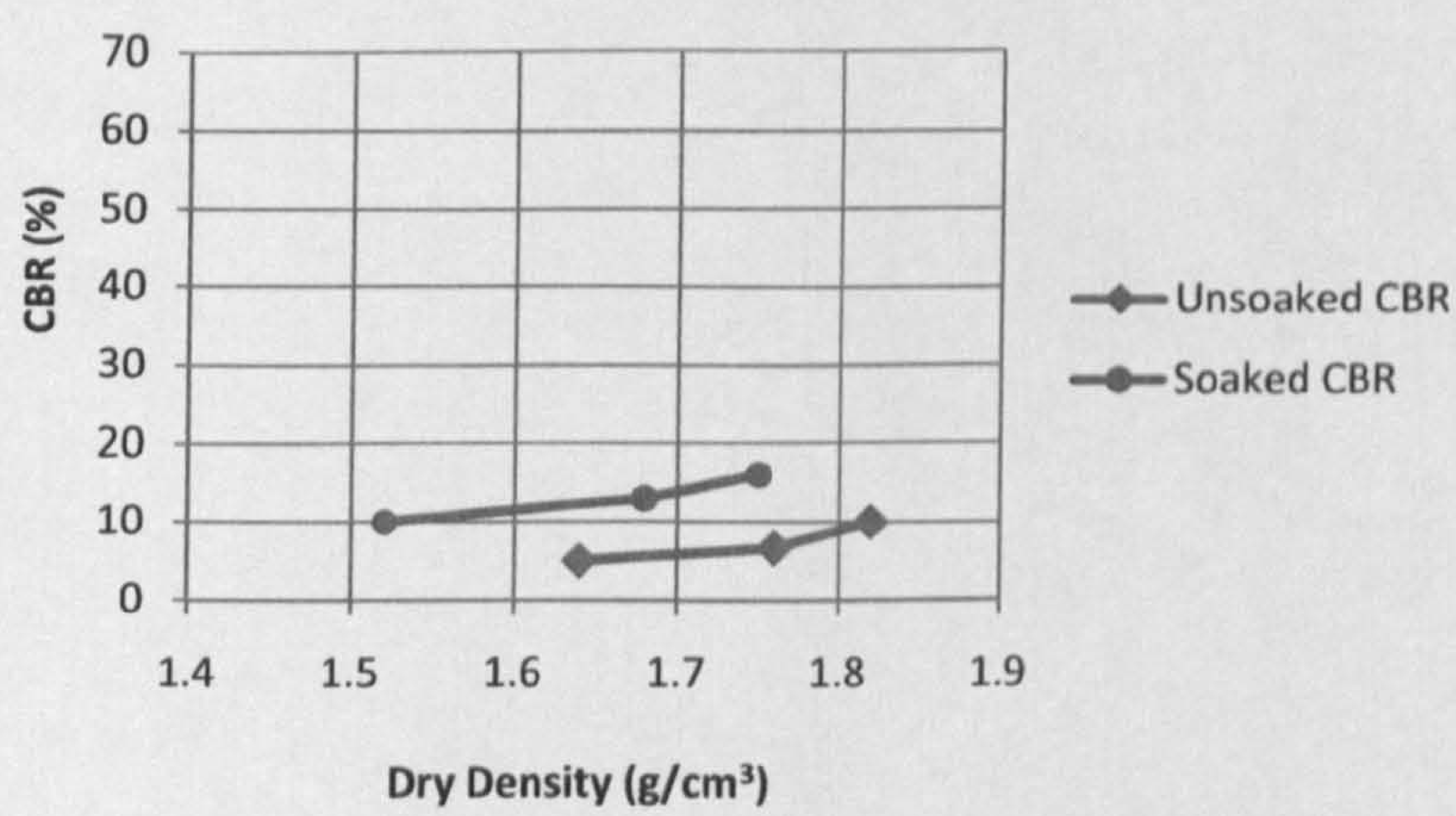


Figure 5.87 CBR- Dry Density relationships for 100%RAP

From the above graphs, a summary of the CBR test results, for unsoaked and soaked samples, are given in Table 5.12:

Table 5.12 Unsoaked and soaked CBR values obtained from samples compacted with 65 blows at OMC

MATERIAL	Unsoaked CBR (%)	Soaked CBR (%)
100%RCA	98	32
80%RCA+20%RAP	64	32
50%RCA+50%RAP	38	27
20%RCA+80%RAP	17	26
100%RAP	10	16

As seen from Table 5.12, in the unsoaked state the 100%RCA material had the highest CBR value (98%). With increasing RAP content up to the 100% level, the CBR drastically decreased to 10%. This result can be attributed to sliding of the bitumen coated aggregates over each other under the penetration of the CBR piston. This led to a decrease in the overall bearing strength of the materials. Although the highest dry density was displayed by the 50%RAP+50%RCA mix, blending of these materials apparently compromised the effectiveness of interlocking between particles thereby resulting in reduced CBR as shown in Table 5.12. Furthermore, it can be seen from Table 5.12 that the 4-day soaked period decreased the CBR values of 100%RCA but increased those of 100%RAP. Figures 5.84 to 5.87 confirmed that replacing RCA by RAP in increasing percentages gradually moved the curve for soaked CBR above the unsoaked CBR curve. Overall all the CBR results for 100%RCA, 80%RCA+20%RAP and 50%RCA+50%RAP satisfied the requirement CBR≥25% by the Iran Highways Asphalt Paving Code (2003).

5.17.7 Soundness of aggregate using sodium sulphate

Figures 5.88 and 5.89 present the soundness loss values obtained from the sodium sulphate/ magnesium sulphate immersion test cycles described in Section 5.16.7.2.

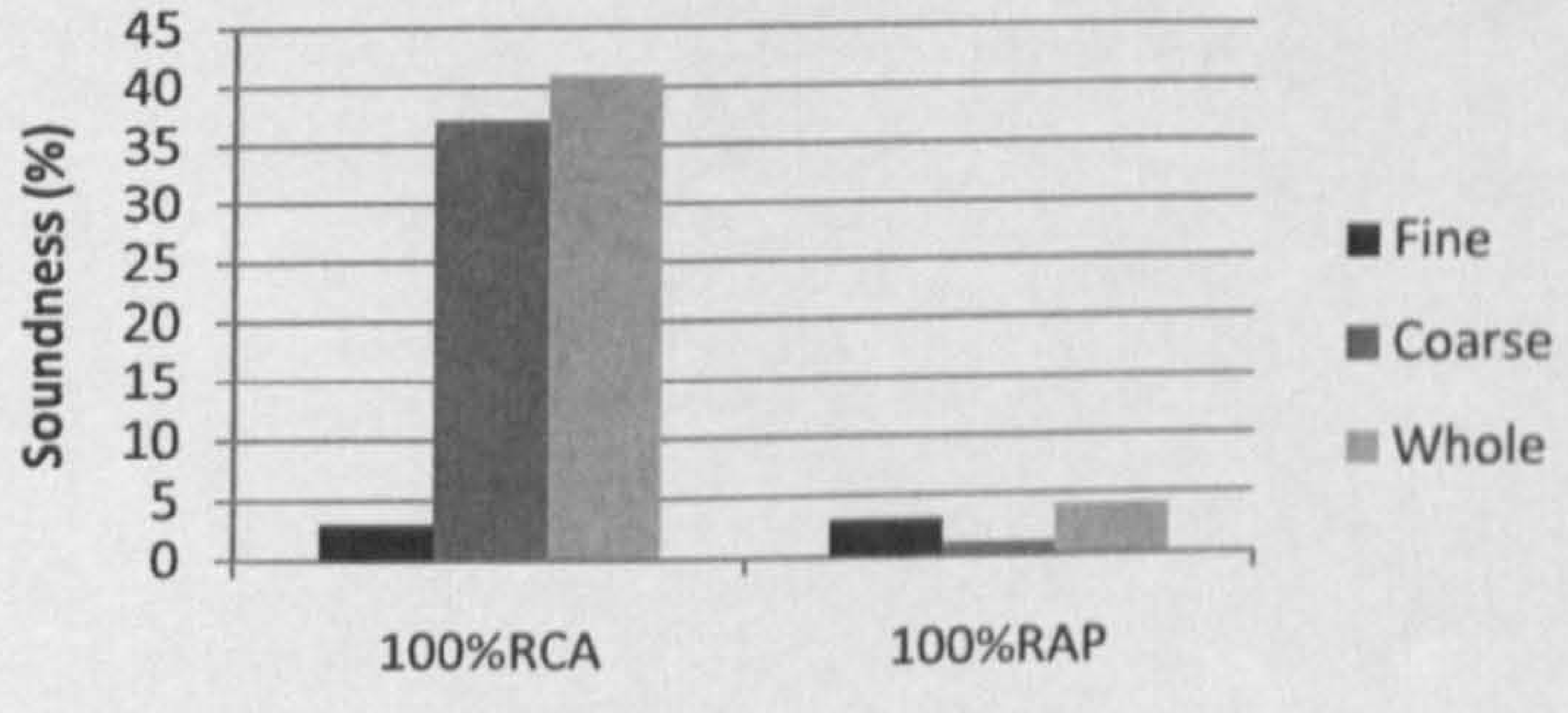


Figure 5.88 Comparison of results from durability test of soundness with sodium sulphate

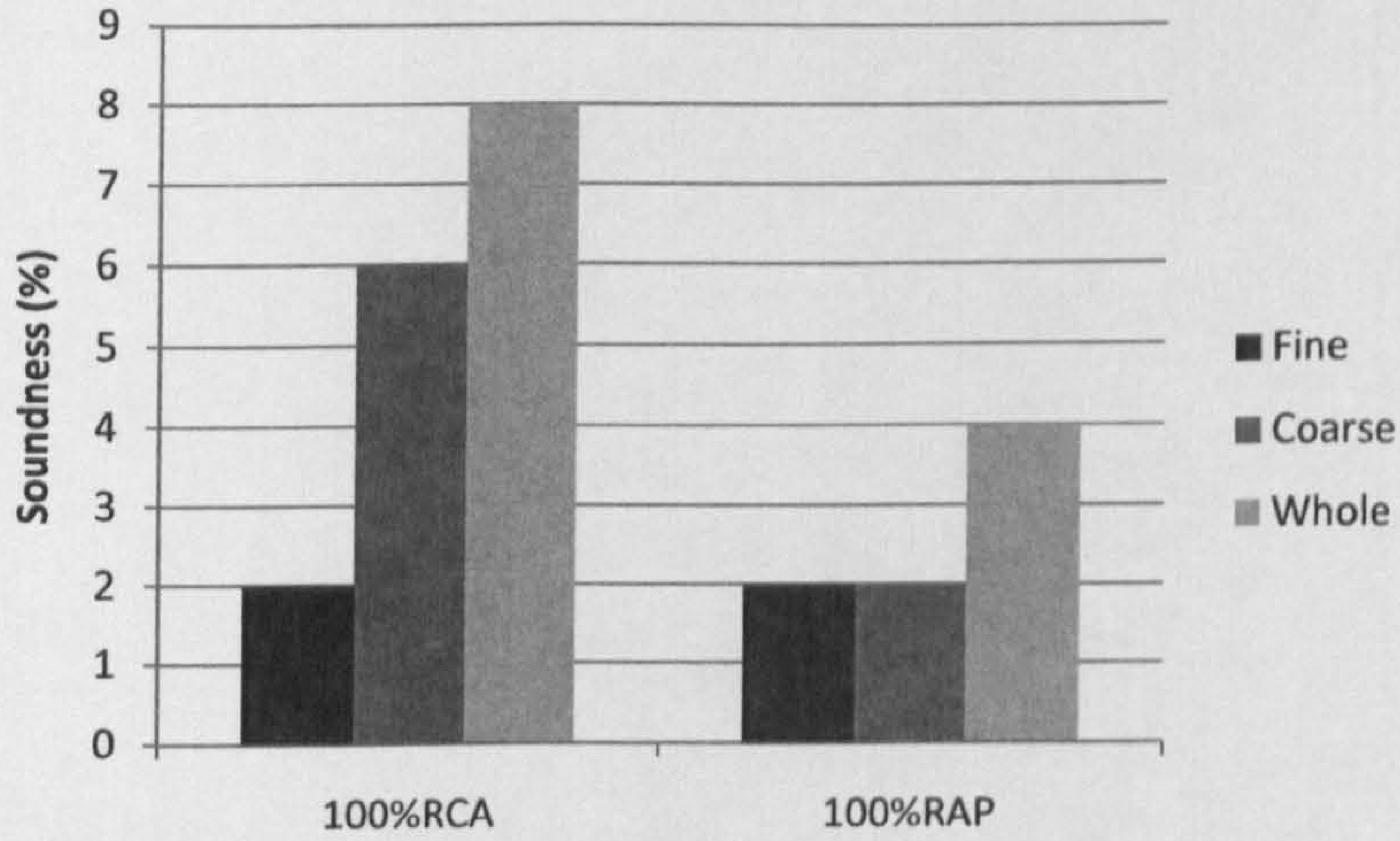


Figure 5.89 Comparison of results from durability test of soundness with magnesium sulphate

As demonstrated in Figs. 5.88 and 5.89 the 100%RAP was found to be more resistant to disintegration by saturated solutions of sodium sulphate and magnesium sulphate. The measured weight loss, for the whole particle size range, was only 4% in both cases. Thus RAP is better capable of resisting the internal expansive forces generated from the rehydration of both magnesium and sodium sulphate salts during re-immersion. Given the high weight loss of 37% in sodium sulphate as shown in Fig. 5.88, the coarse fraction of 100%RCA is judged to have relatively less durability when subjected to repeated cycles of freezing and thawing. The separation of stones and mortar and cement can be the reason of this phenomenon. As regards the fine fractions of RCA and RAP, Figs. 5.88 and 5.89 show no discernible differences between the soundness test results, both from magnesium and sodium sulphate methods. For the coarse fractions, both Figs. 5.88 and 5.89 indicate that the loss values for the 100%RCA was higher than those of the RAP. It is also noted that the 100%RCA (coarse and whole particle size ranges) did not fulfill the Iran Highway Asphalt Paving Code limit of 12% loss (for pavements of unrestricted use) while the 100%RAP met these requirements.

5.17.8 Determination of asphalt binder in RAP

To determine the percentage of bitumen in the RAP the Centrifuge extractor method described in AASHTO T 164-06 was used. The results showed that the asphalt content in the RAP, which was extracted from the wearing course, was approximately 4.9%. For various sizes, the percentages of extracted aggregates are listed below:

Sieve Size	Passing Percent (%)
1/2"	87
3/8"	87
NO.4	58
NO.8	33
NO.30	33
NO.50	7
NO.100	7
NO.200	0

There is no correlation between bitumen content and properties of RCA.

CHAPTER 6 AGGREGATES PERFORMANCE ASSESSMENT

6.0 Preamble

The performance of aggregates was evaluated based on the test specifications given in the NCHRP Reports 453 and 598. This document was selected for benchmarking because was the most comprehensive one in the world, for the use of recycled aggregates. The performance related assessment is carried out on the basis of the information already presented in Tables 4.4 and 4.8 in chapter 4. There are three main factors involved here; i.e. (a) traffic loading, (b) moisture levels in highway pavements and (c) the temperature conditions. The NCHRP Reports 453 and 598 defines four significance levels (numbered 1-4 in order of increasing significance to aggregate performance potential) combining the effects of traffic loading, moisture and temperature conditions as shown previously in Table 4.3.

The performance of mixes are assessed in this chapter separately for toughness, durability, stiffness, frost susceptibility and shear strength according to test results from chapter 5 and will be classified according to Table 4.3 as a design aid.

6.1 Performance Potential Based on Toughness

The toughness of T-1, T-2 and T-3 was assessed by Micro-Deval test (MD), Los Angeles Abrasion (LAA), Aggregate Crushing Value (ACV), Aggregate Impact Value (AIV). But toughness testing carried out in Iran utilized LAA and degradation after compaction. In Table 4.7 the MD test was rated as "good", but other tests were ranked as "fair". It should be recognized that the degradation of compacted samples is a simplified and practical simulation of the gyratory degradation test described in NCHRP 453 for natural aggregates. Paranavithana and Mohajerani (2003) also used the gyratory degradation test in their research relating to RCA. As was illustrated in Tables 4.4 and 4.8, the MD test is the recommended method for assessing the toughness/abrasion potential of natural and recycled aggregates. As such the results of other tests, if done instead of MD test, should be transformed into the equivalent MD test value before using Tables 4.4 and 4.8. However, this is not easily achieved because at present there are few published correlations between, say LAA and MD values, and also these correlations suffer from low reliability. As an example, a scatter diagram of LAA versus MD values reported by Rangaraju and Edlinski (2008) is illustrated in Figure 6.1. This chart was plotted from a study of some aggregates used in hot mix asphalt (HMA). As will be seen shortly in Table 6.1, the author's tests showed that all aggregates with LA abrasion of less than 35% had the MD of less than 23.5%. These limits are annotated in Fig. 6.1 to have an approximate way of using LAA values to predict what the MD values of the materials tested in Iran would have been.

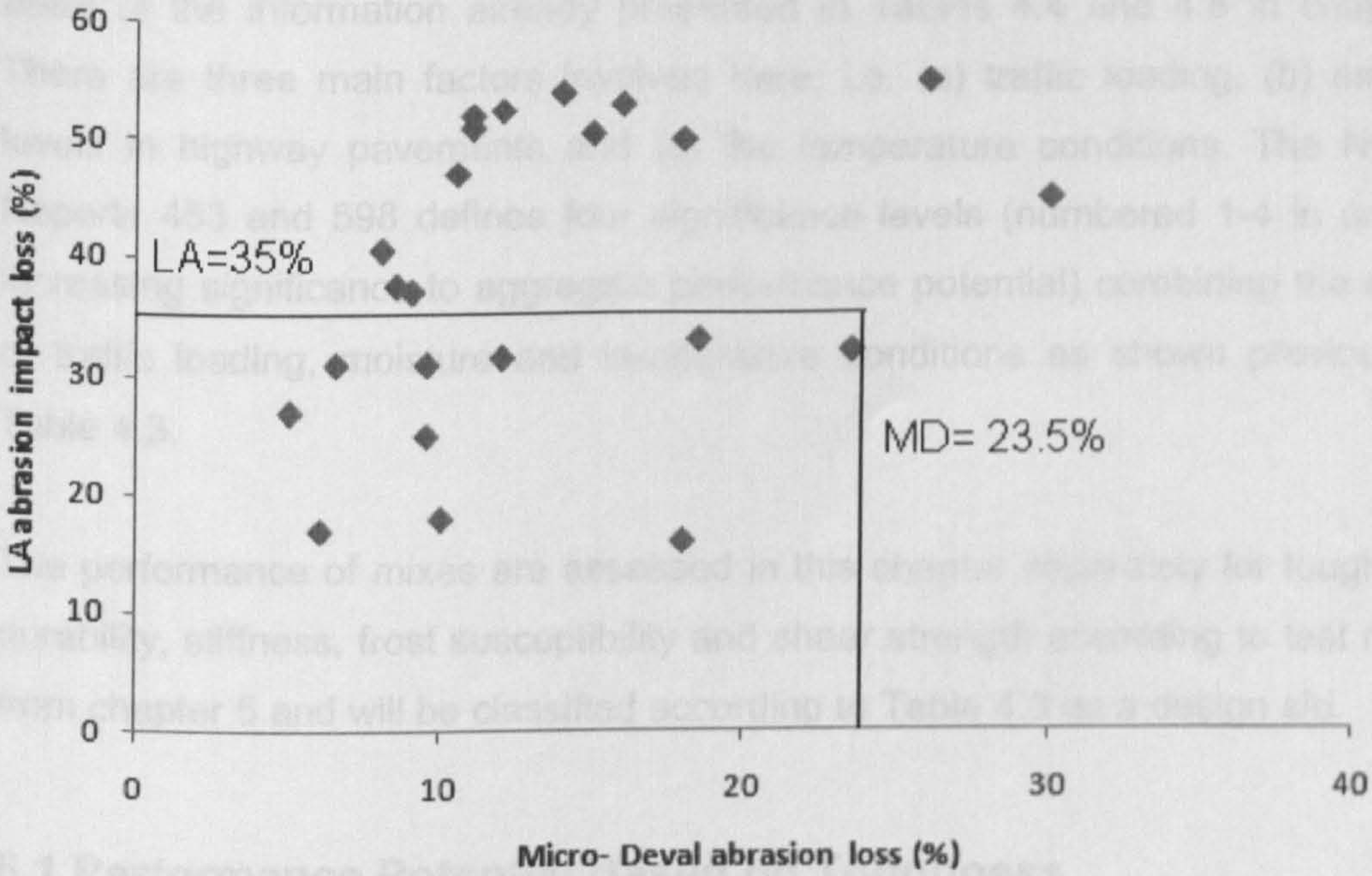


Figure 6.1 Chart showing the likely range of MD values corresponding to LAA< 35% (After Rangaraju and Edlinski, 2008)

Williamson (2005) presented a chart for correlating LAA and MD values for natural and recycled aggregates. The chart, which is shown in Figure 6.2, indicates a less degree of scatter (i.e. a better correlation) as compared to the data points in Fig. 6.1. Remarkably for LAA=35% (the maximum value observed in the author's data in Table 6.1) the corresponding MD=23%, which is consistent with the author's suggestion.

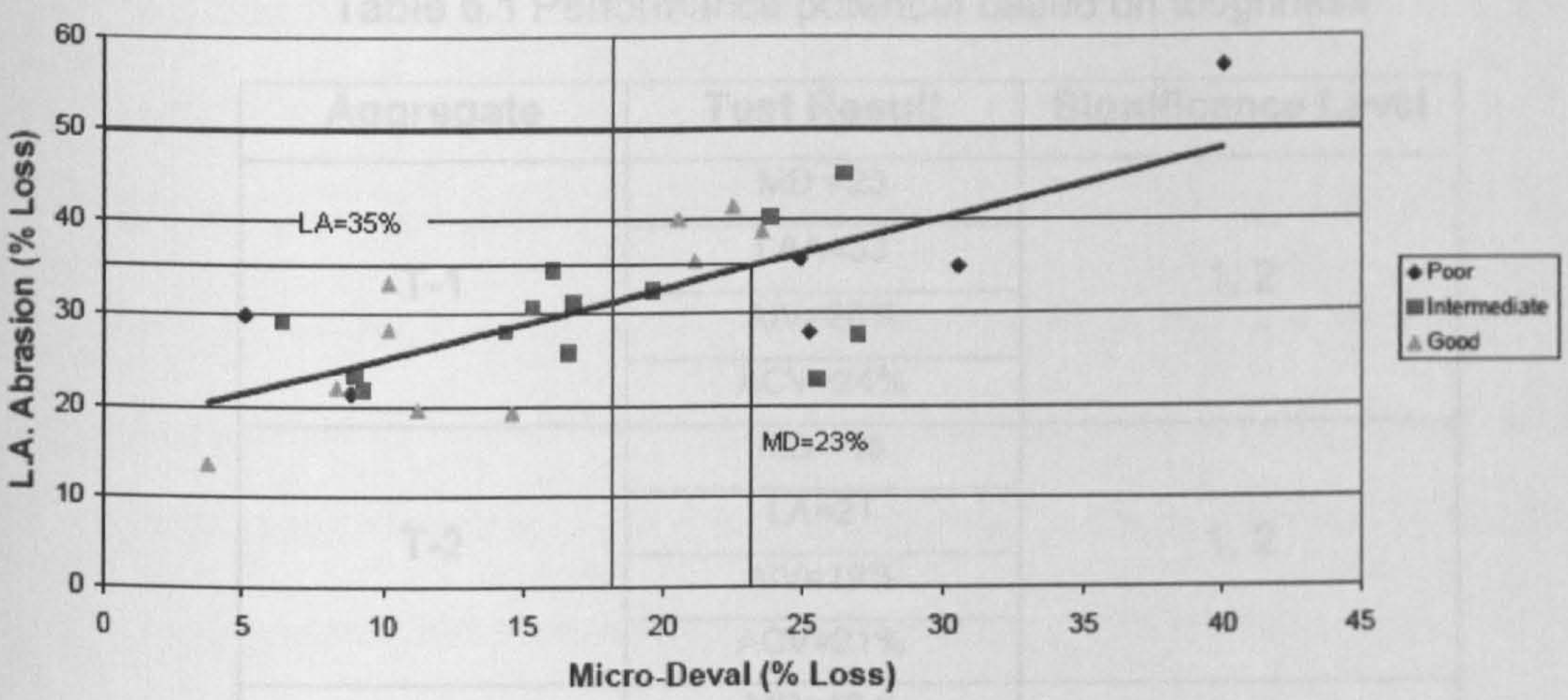


Figure 6.2 L.A. Abrasion vs. Micro-Deval (After Williamson, 2005)

All aggregates tested in Iran had LAA of less than 31% apart from the 100%RCA which had LAA=31%. Based upon the author's suggested correlation from Figs. 6.1 and 6.2 the corresponding MD is judged to be less than 30%, which meets the requirements of significance levels of 1,2.

Aggregate strength is important for unbound aggregate where the subbase is subjected to the loads during construction and in service. Williamson (2005) suggested that Micro-Deval test should be used in conjunction with the ACV test to characterize the strength of aggregates. The researcher also demonstrated that LAA and ACV test results are closely related. As cited by Wu *et al.* (1998), ACV, and not AIV, can be used to assess the suitability of an aggregate for application in bitumen macadam for roadbases. In another citation in this research, it was suggested that a combination of impact, crushing and abrasion resistance should be used to assess aggregate toughness.

Table 6.1 Performance potential based on toughness

Aggregate	Test Result	Significance Level
T-1	MD =23 LAA=33 AIV=28% ACV=24%	1, 2
T-2	MD =18 LA=21 AIV=19% ACV=21%	1, 2
T-3	MD=18.6 LAA=22.4 AIV=16% ACV=16%	1, 2
100%NA	LAA=24% Degradation=Medium	1, 2
80%NA+20%RCA	LAA=28%	1, 2
50%NA+50%RCA	LAA=30%	1, 2
20%NA+80%RCA	LAA=30%	1, 2
100%RCA	LAA=31% Degradation=High	1, 2
20%RAP+80%RCA	LAA=29%	1, 2
50%RAP+50%RCA	LAA=29%	1, 2
80%RAP+20%RCA	LAA=28%	1, 2
100%RAP	LAA=28% Degradation=Low	1, 2

As has been discussed, all the publications examined are in agreement that Micro-Deval is the most appropriate test for assessing toughness of aggregates. In addition, for assessment of the overall performance of recycled aggregates other supporting tests are also necessary. This is why the present research has included a wide range of testing to include: LAA, AIV, ACV and degradation of aggregates to supplement MD test values. The overall results of toughness properties showed that:

- Low degradation of 100%RAP caused the mixes containing RAP to have lower AIV and ACV.
- Degradation test results were consistent with those of AIV and ACV, and hence justifying the reasonableness of the results of LAA and MD tests.
- Degradation of 100%RCA is increased by the process of extraction of RCA, which may mean that cement mortar attached to the aggregates is crushed or partly detached from the aggregates causing weakness in RCA. This is manifested in the results of the other toughness tests such as LAA and MD.
- In summary, it is seen in Table 6.1 that for all tested materials, obtained from different sources, the significance level was always either 1 or 2. This implies that from the viewpoint of toughness, the materials investigated here would be appropriate for unbound subbases for medium and low traffic situations (subject to other criteria being satisfied).

6.2 Performance Potential Based on Stiffness

Figure 6.3 shows the correlation chart for estimating the resilient modulus of granular subbases from CBR values, as extracted from the well-known "Van Till chart" available in text books such as Huang (2004).

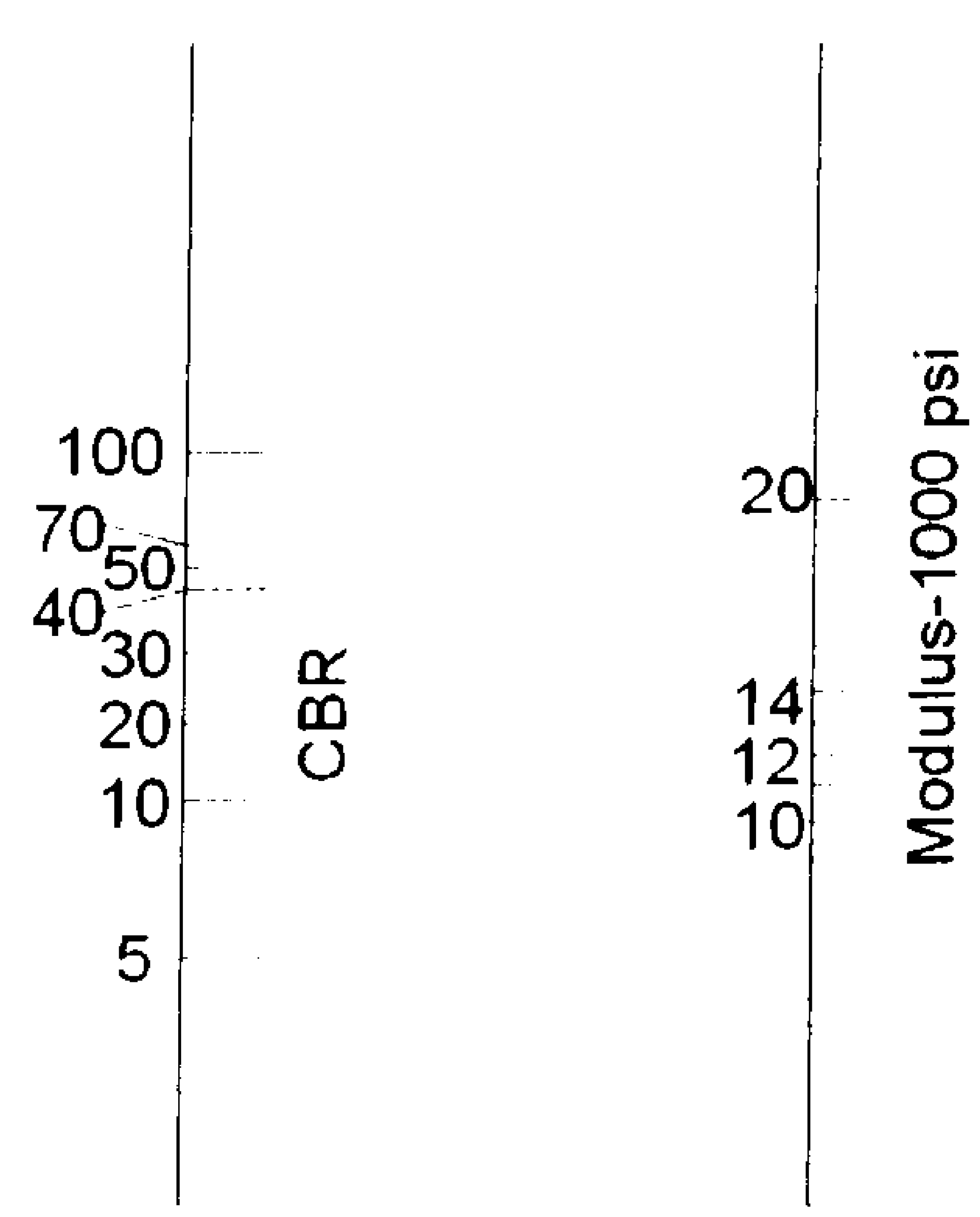


Figure 6.3 Correlation chart for estimating resilient modulus of subbase(After Huang, 2004)

The resilient moduli of mixes were correlated from their CBR and are given in Table 6.2. It was found that all mixes, except unsoaked CBR of 100%RCA and 80%RCA+20%NA, had resilient moduli below 20 ksi. Therefore according to Table 4.8, it seems that the tested materials other than 100%RCA and 80%RCA +20%NA may only be appropriate for low traffic, low moisture and non-freezing conditions. This is because the correlated resilient modulus of the materials falls below 20ksi. Having said that, the author feels inclined to suggest that direct measurements of resilient modulus, rather than empirical correlation, may more accurate values and hence change the significance level.

Table 6.2 Performance potential based on stiffness

Aggregate	Test Result	Resilient Modulus (psi)	Significance Level
T- 1	Unsoaked CBR=50.15%	17500	1
	Soaked CBR=31%	14800	
T- 2	Unsoaked CBR=71.14%	18750	1
	Soaked CBR=42%	17250	
T- 3	Unsoaked CBR=28.59%	14700	1
	Soaked CBR=25%	14000	
100%NA	Unsoaked CBR=70%	18750	1
	Soaked CBR=45%	17250	
80%NA+20%RCA	Unsoaked CBR=79%	20000	1
	Soaked CBR=42%	17000	
50%NA+50%RCA	Unsoaked CBR=80%	20000	1
	Soaked CBR=40%	17000	
20%NA+80%RCA	Unsoaked CBR=86%	+20000	1
	Soaked CBR=37%	16250	
100%RCA	Unsoaked CBR=98%	+20000	1
	Soaked CBR=32%	15000	
20%RAP+80%RCA	Unsoaked CBR=64%	18000	1
	Soaked CBR=32%	15000	
50%RAP+50%RCA	Unsoaked CBR=38%	16250	1
	Soaked CBR=27%	14200	
80%RAP+20%RCA	Unsoaked CBR=17%	12000	1
	Soaked CBR=26%	14000	
100%RAP	Unsoaked CBR=10%	10500	1
	Soaked CBR=16%	12000	

6.3 Performance Potential Based on Durability

Aggregate degradation effects can be caused by the processing, transportation, construction and traffic loading subsequent to construction of a highway pavement. In general terms, long-term degradation forces are measured though durability tests such as sulphate soundness tests (sodium sulphate and magnesium sulphate) and frost/heave resistance tests.

According to Williamson (2005) sodium sulphate soundness (SS) test is not appropriate for evaluation of RCA due to chemical reactions between the sulphate and mortar. The researcher argues that the chemical reactions result in high mass looses thereby not being representative of the true soundness characteristics of the aggregate. From this argument, it seems that there are demerits of using RCA for base or subbase materials in areas of high soil sulphate content. As an alternative to sodium sulphate, magnesium sulphate (MS) test may be used. Nevertheless, the results should be treated with caution and correct judgment because it has been reported by Williamson (2005) that MS soundness values were generally about half of SS soundness values. Other authors such as Chini *et al.* (2001) suggested that for RCA an alternative soundness test method involving wetting and drying should be adopted.

According to AASHTO Designation M319-02 (2006) some RCAs yield high soundness loss values when tested with conventional sulphate soundness testing methods. Therefore these methods may not be appropriate for soundness testing of RCAs. In this case, the "No-Test" approach outlined in AASHTO Designation M319-02 (2006) waives the requirement for soundness testing and approves the use of RCA provided it meets the other quality tests. Kue *et al.* (2001) proposed that, for RCA, a sodium sulphate soundness loss of less than 50% can be taken as suitable for unbound layer application. The limit of 50% for RCA is equivalent to the Florida DOT specification (Kue *et al.* 2001) of 15% for natural aggregates. As regards the test the coarse fraction of RCA materials tested in Iran as part of this research, the sodium sulphate soundness test results obtained are well within the limit of 50% stated above.

With reference to Table 4.8, the author has identified that durability tests are not specified for recycled aggregates and yet strikingly Table 4.4 does explicitly recommend the magnesium sulphate soundness test as a durability test to be complied with. Accordingly, Table 6.3 has been prapared for summarising durability performance based only on sulphate soundness testing results. The criteria mentioned in Table 4.4 shows that 100%NA, 100%RCA and 100%RAP will have the requirement of all significance levels: 1, 2, 3 and 4.

Table 6.3 Performance potential based on durability

Aggregate	Test Result	Significance Level
T-1	MS =7%	1, 2,3,4
T-2	MS =3%	1, 2,3,4
T-3	MS =6%	1, 2,3,4
100%NA	SS=16% MS =3%	1, 2,3,4
80%NA+20%RCA	16%<SS<44% 3%<MS<8%	1, 2,3,4
50%NA+50%RCA	16%<SS<44% 3%<MS<8%	1, 2,3,4
20%NA+80%RCA	16%<SS<44% 3%<MS<8%	1, 2,3,4
100%RCA	SS=44% MS =8%	1, 2,3,4
20%RAP+80%RCA	4%<SS<44% 4%<MS<8%	1, 2,3,4
50%RAP+50%RCA	4%<SS<44% 4%<MS<8%	1, 2,3,4
80%RAP+20%RCA	4%<SS<44% 4%<MS<8%	1, 2,3,4
100%RAP	SS=4% MS =4%	1, 2,3,4

Since 100%RCA was found to be non-plastic and had the greatest sand equivalence of 76%, the material is considered to have high durability and that the high weight loss values observed in the coarse RCA may be attributed to sulphate attacks. From the above points, it may be suggested that:

(a) If the sulphate levels in a subgrade soil and surface water are not of concern then 100%RCA can be used for significance levels: 1, 2, 3 and 4 from durability performance point of view.

(b) The mixes of RCA with NA and RCA and RAP can achieve significance levels: 1, 2, 3 and 4 as relates to durability. This is because the soundness values from magnesium sulphate were less than 13%.

6.4 Performance Potential Based on Frost Susceptibility

In Table 4.8 the Tube Suction Test (TST) was suggested for the evaluation of frost susceptibility of mixes. However, since this test is not commonly used in UK practice, the present research focused on other related properties of aggregates to assess frost susceptibility. The Federal Aviation Administration (FAA) of the United States Department of Transportation (DOT) developed a method for correlating the degree of frost susceptibility with the amount of material finer than 0.02mm by weight (O'Donnell, 2009). In this method aggregates are categorized into four groups for frost resistance analysis purposes. The groups, which are shown in Table 6.4, are: Frost Group 1 (FG-l), Frost Group 2 (FG-2), Frost Group 3 (FG-3), and Frost Group 4 (FG-4). The higher the frost group number, the more susceptible to freezing the soil is.

Table 6.4 Frost groups (After O'Donnell, 2009)

Frost Group	**Type of Soil**	**Percentage finer than 0.02mm by Weight**
FG-1	Gravely Soils	3-10
FG-2	Gravely Soils	10-20
	Sands	3-5
FG-3	Gravely Soils	Over 20
	Sand, except very fine silty sands	Over 15
FG-4	very fine silty sands	Over 15

The grading curves in Figure 5.1 show that the percentages passing the 0.063mm sieve for T-1, T-2 and T-3 are 0.96%, 0.46% and 0.45%, respectively. Therefore, all these materials (T-1, T-2 and T-3) have less than 3% passing the 0.02mm sieve so that they fall under category FG-1. For mixes tested in Iran all mixes had less than 10% passing the No. 200 (0.075mm) apart from 80%NA+20%RCA, 100%RAP and 80%RAP+20%RCA. For these three materials, the portions of the grading curves below the No. 200 sieve were determined using hydrometer analysis. This test gave the following results:

Material	% finer than 0.02 mm
80%NA+20%RCA	3.2%
100%RAP	3.8%
80%RAP+20%RCA	4.5%

Therefore all mixes can be classified as FG-1 i.e. the low frost susceptibility. From the results and discussion already made in Section 5.14 of chapter 5, T-1, T-2 and T-3 can be categorized as non-frost susceptible according to MCHWSeries 800, because the mean heave (showe with FH in Table 6.5) of them is less than 15mm. As all mixes were found to fall under category FG-1 (low frost susceptibility), it follows from Table 4.4 that these materials are appropriate for all significance levels (1-4). A summary of the results of frost heave (FH), percentages finer than 0.02 mm and corresponding significance levels is presented in Table 6.5.

Table 6.5 Performance potential based on frost susceptibility

Aggregate	Test Result	Significance Level
T-1	FH=10.7mm Finer than 0.02mm< 3.89%	1, 2,3,4
T-2	FH=9.0mm Finer than 0.02mm< 0.46%	1, 2,3,4
T-3	FH=8.7mm Finer than 0.02mm< 0.45%	1, 2,3,4
100%NA	Finer than 0.02mm< 4.49%	1, 2,3,4
80%NA+20%RCA	Finer than 0.02mm=3.2%	1, 2,3,4
50%NA+50%RCA	Finer than 0.02mm< 9.76%	1, 2,3,4
20%NA+80%RCA	Finer than 0.02mm< 7.88%	1, 2,3,4
100%RCA	Finer than 0.02mm< 9.5%	1, 2,3,4
20%RAP+80%RCA	Finer than 0.02mm< 8.22%	1, 2,3,4
50%RAP+50%RCA	Finer than 0.02mm< 9.20%	1, 2,3,4
80%RAP+20%RCA	Finer than 0.02mm=4.5%	1, 2,3,4
100%RAP	Finer than 0.02mm= 3.8%	1, 2,3,4

Specific gravity and water absorption is not directly related to the quality in itself; however, it can be an indication of potential problems and is required for computations involving volume and mass. Water absorption has been used as a preliminary indicator of aggregate durability as related to freezing and thawing. High absorption is often an indicator of unsound aggregates (Weyers *et al.* 2005). Some state DOT's have limits on the absorption of aggregates to help prevent accumulation of freezing and thawing damage. For instance, the Minnesota DOT limits the absorption of PCC aggregates to 1.7%. According to Weyers *et al.* 2005, water absorption test is not suitable for investigating RAP or RCA because of the adverse effects of adhered binder and mortar on absorption properties.

6.5 Performance Potential Based on Shear Strength

It should be noted that according to NCHRP 598 (also see Table 4.8), it is the static triaxial test that is recommended for shear strength mesurement. Nevertheless, for the present research, tests were carried out using a large direct shear test apparatus in the UK because of the availale triaxial cells were too small and inappropriate for the type of materials to be tested. The suggestion to use a large scale shear box was given by professionals from a collaborating establishment (Day Group Ltd) in the UK. This alternative approach has been shown, after many years of industial experience, to give reasonable results that correlate well with static triaxial test data. The results of ϕ' and c' for T-1, T-2 and T-3 were presented in Table 5.4. As the shear strength tests for the three mixes has been carried out at OMC, in using Table 4.8, it was necessary to determine the deviator stresses (σ_d) coresponding to a confining stress of 5 psi in a static triaxial test. Since the tested materials were granular in nature and hence of high permeability, it is reasonable to assume that the pore water pressure conditions in the direct shear test would be comparable to those which would have been experienced in a static triaxial test. Therefore the results of ϕ' and c' from a shear box test were expected to be close to those that would have been obtained from a triaxial test. With this assumption the σ_d was evaluated from Figure 6.4 which shows Mohr's failure envelop for a triaxial compression test series with only a single test (Bowles, 1996).

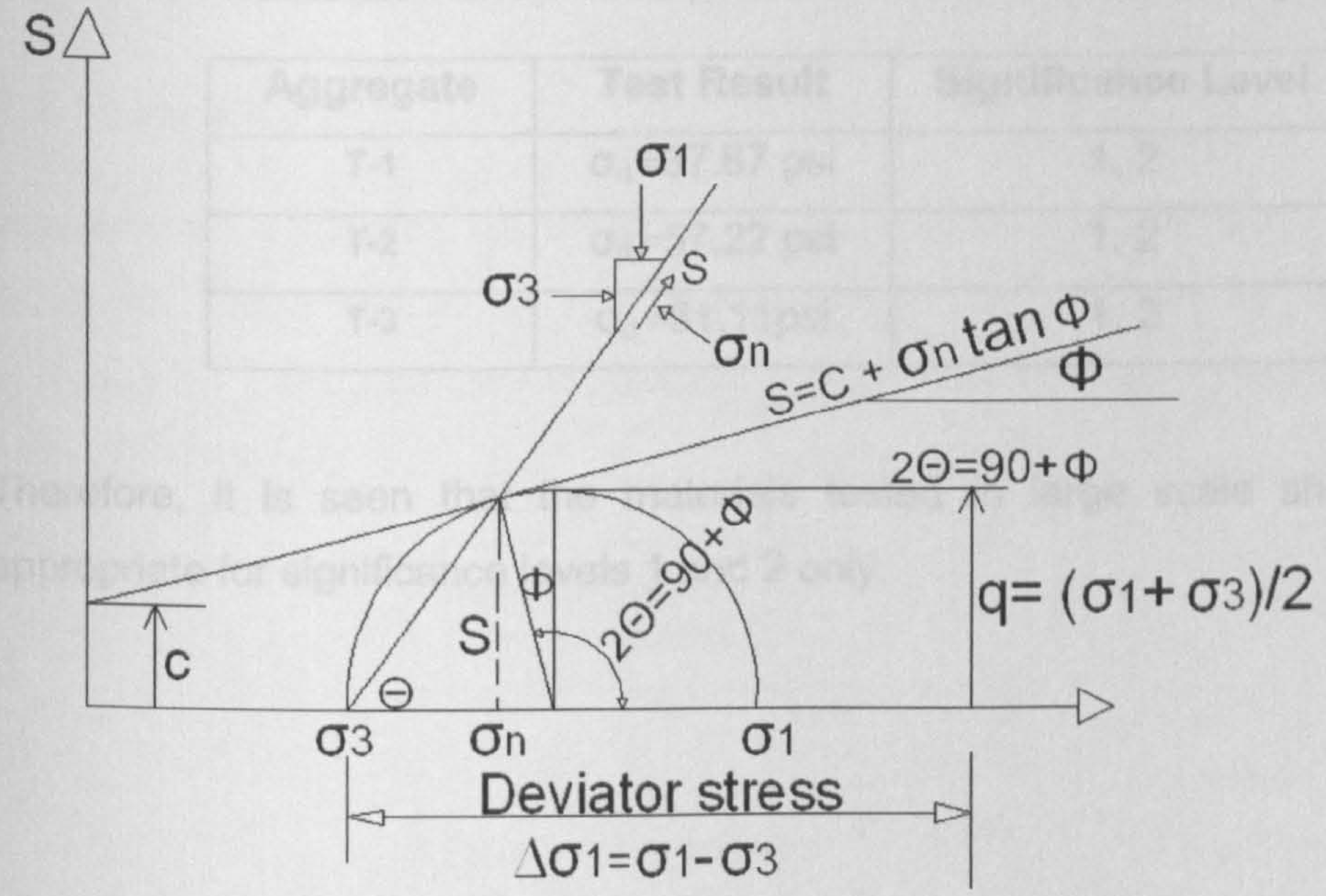


Figure 6.4 Mohr's failure stress circle for a triaxial compression test (After Bowles, 1996)

The major and minor principal stresses (σ_1 and σ_3) on a shear plane for this sample are related through Equation 6.1, which may be solved for the desired confining stress which of 5 psi (34.47 kPa). Solving Equation 6.1 in terms of σ_3 , ϕ' and c' will give σ_1. The principal stress difference σ_1 and σ_3 will be deviator stress (σ_d).

$$\sigma_1 = \sigma_3 \tan^2(45 + \tfrac{\varphi}{2}) + 2C.\tan(45 + \tfrac{\varphi}{2}) \qquad \textbf{(Equation 6.1)}$$

The results are presented in Table 6.6:

Table 6.6 The results of static triaxial test

Mixes	c' (kPa)	ϕ' (degrees)	σ_1 (psi)	σ_d (psi)
T-1	28	41.5	42.67	37.67
T-2	44	46.0	62.23	57.22
T-3	20	40.5	36.11	31.11

As seen in Table 6.6 above, the deviator stresses of all these mixes are less than 60 psi and greater than 25 psi. Now, referring to the criteria already given in Table 4.8, the deviator stress values listed in Table 6.6 indicate the interpreted significance levels shown in Table 6.7.

Table 6.7 Performance potential based on shear strength

Aggregate	Test Result	Significance Level
T-1	σ_d =37.67 psi	1, 2
T-2	σ_d =57.22 psi	1, 2
T-3	σ_d =31.11psi	1, 2

Therefore, it is seen that the materials tested in large scale shear box are appropriate for significance levels 1 and 2 only.

CHAPTER 7 MODELING AND DESIGN

7.0 Preamble

This chapter compares the damage criteria in TRL, Asphalt Institute and Mohr-Coulomb failure model. A typical pavement cross section is modeled with KENLAYER™ program and discussed. The stages of applied model are clarified below (Figure 7.0):

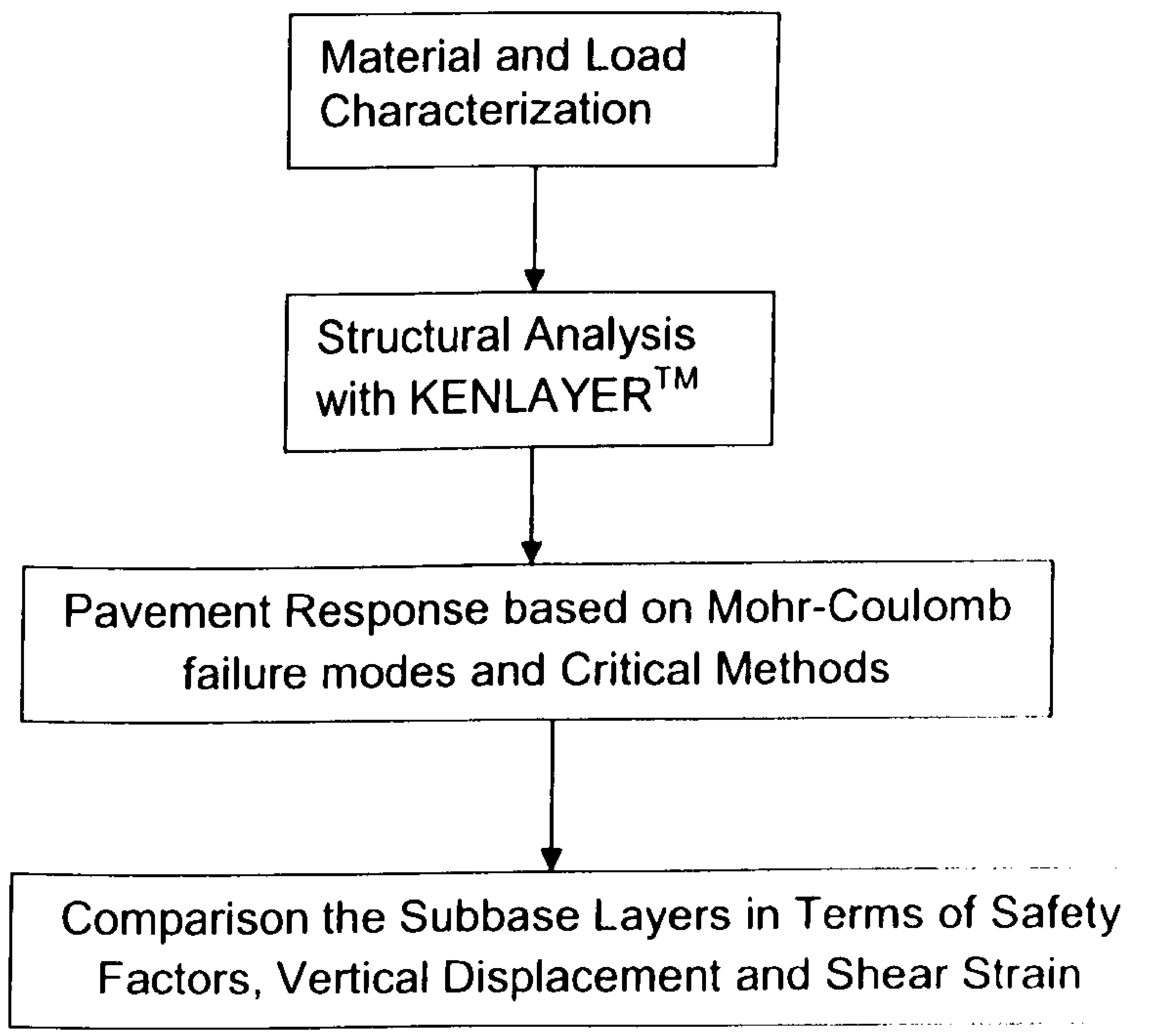


Figure 7.0 Stages of modeled used

7.1 Design Criteria of TRL

According to IAN 73 (2006), the single-stage design criteria were defined with respect to the following variables:

a) The horizontal tensile strain at the bottom of the pavement layer
b) The vertical compressive strain at the top of the subgrade

As a result of further research aided by practical experience and numerical analysis, the old theoretical model shown in Figure 7.1(a) was divided into two parts; namely, the pavement design model [Figure 7.1(b)] and the foundation and design model [Figure 7.1(c)]. The performance related design approaches discussed in Section 4.5 should satisfy the dominant criterion listed in Table 7.1 which are to be used for foundation design in the 'Shortly after construction' and 'Long-term' design stages (Chaddock and Roberts, 2006).

As Table 7.1 shows, all criteria depend not only on subbase specifications but also on other parameters such as foundation surface modulus. This is a composite parameter which is determined by (i) the properties of all underlying layers, and (ii) the CBR and strain of the subgrade. In this research, the focus is on the loading effects on the subbase layer only, rather than the combined multilayered pavement structure.

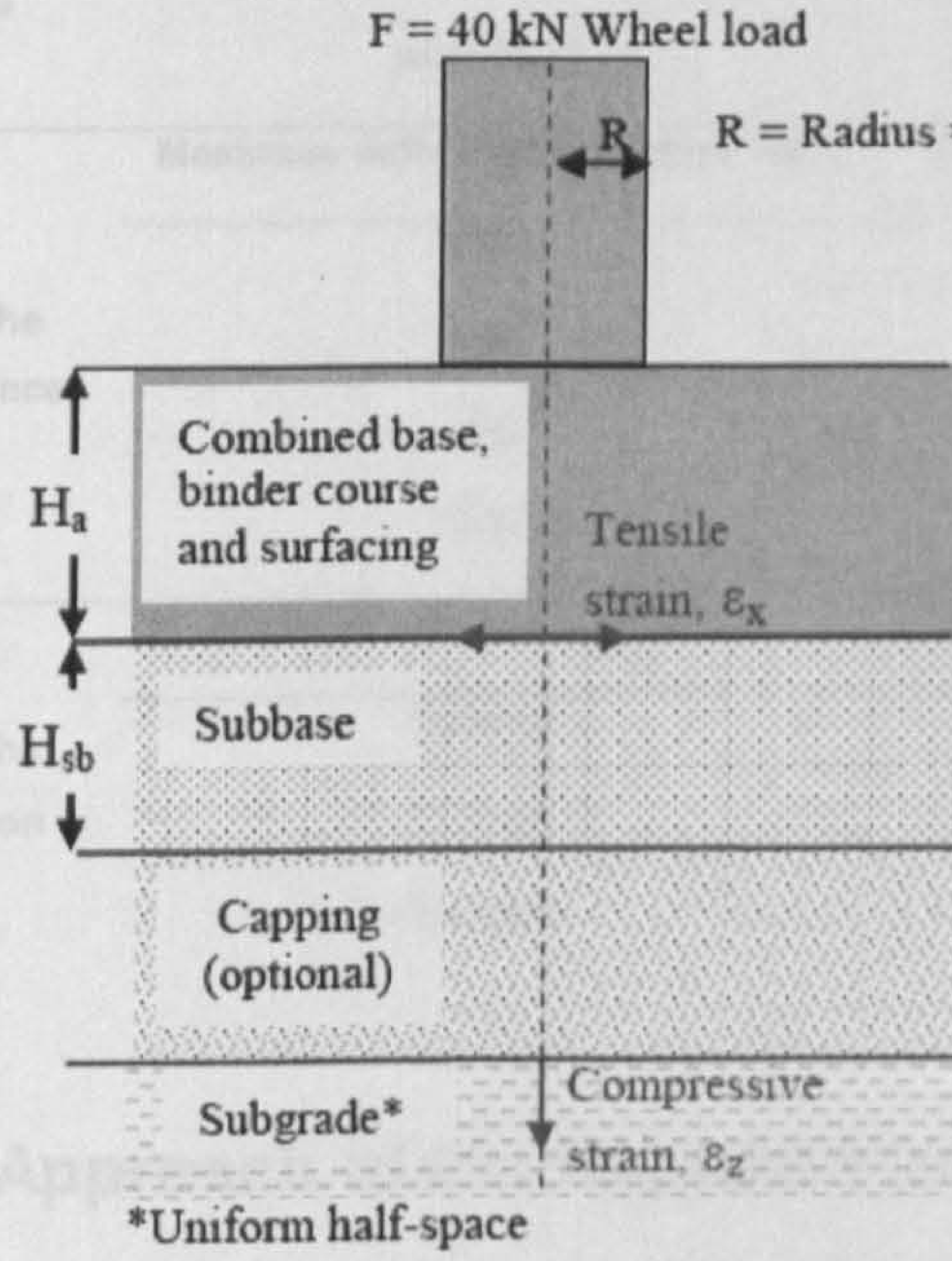


Figure 7.1(a) Theoretical pavement design (referred to as the old model)

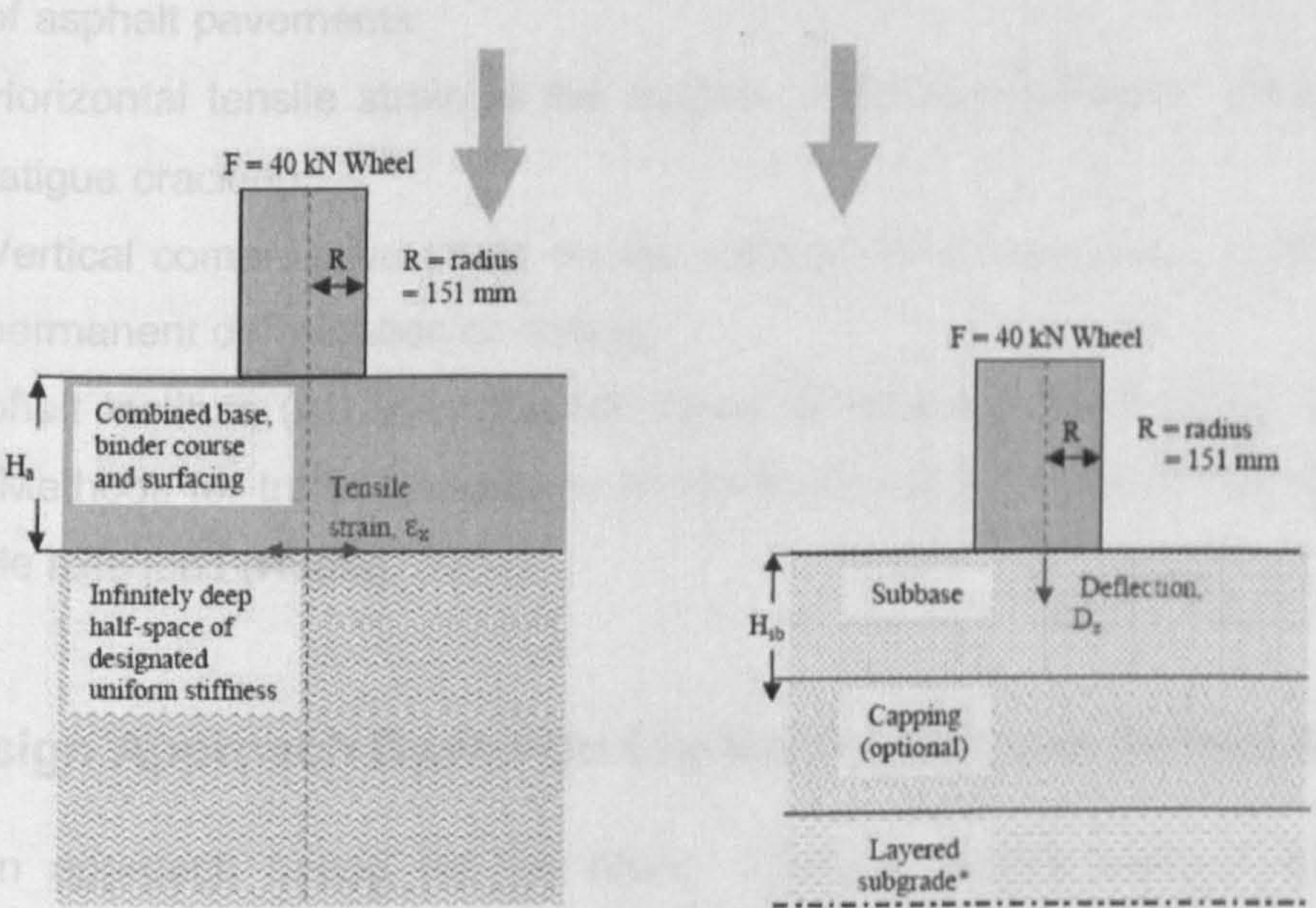


Figure 7.1(b) Theoretical pavement model   Figure 7.1(c) Adopted theoretical foundation model

Figure 7.1 Evolution of pavement and foundation design (After Chaddock and Roberts, 2006)

Table 7.1 Design criteria (After Chaddock and Roberts, 2006)

Label	Criteria	Permissible values		Stage
		Maximum subgrade strain (με) for:	**Soil subgrade CBR range (%)**	
ε_z	**Vertical compressive strain at the top of the subgrade**	ε_z<(403.7*CBR+1024)*k_f	>2.5 to ≤5	**Shortly after construction**
		ε_z<(-31.03*CBR+3210)*k_f	>5 to ≤15	
		ε_z<(-2752*Log_{10}(CBR)+5947)*k_f	> 15 to ≤30	
		,where k_f=1		
		Maximum deflection (microns) for:	**Foundation class**	
D_z	**Deflection of the foundation surface**	2960	FC1	**Long-term**
		1480	FC2	
		740	FC3	
		370	FC4	
		Minimum thickness (mm) for:	**Foundation class**	
H	**Thickness of the upper foundation layer**	150 mm	FC1 and FC2	**Both stage**
		175 mm	FC3	
		200 mm	FC4	

7.2 The Design Approach of the Asphalt Institute

Two types of strains have frequently been considered to be the most critical for the design of asphalt pavements:

a) Horizontal tensile strain at the bottom of the asphalt layer, which causes fatigue cracking.
b) Vertical compressive strain on the surface of the subgrade, which causes permanent deformation or rutting.

The Asphalt Institute (AI) uses the two types of strains defined above as failure criteria. Methods for traffic analysis in AI are expressed in terms of an 18 kip (80 kN) single axle load (Huang, 2004).

7.3 Design Approach Based on the Mohr-Coulomb Failure Model

A design approach based on the Mohr- Coulomb failure criterion has been successfully adopted in some design methods such as the South African Mechanistic Design Method (SAMDA). The criteria and associated design parameters of the method have been published extensively over the last few decades. Very often, the structural analysis of a pavement with a granular base and subbase will result in the mechanistic design method predicting almost no resistance against shear failure in the subbase layer. This is caused by the linear elastic material models used in the static, linear elastic multi-layer solution procedure which allows tensile stresses to develop in an unbound material. The occurrence of tensile stress in a granular layer is again determined by the modular ratio of the stiffness of the granular layer in relation to stiffness of the immediate supporting layer (Theyse and Muthen, 2000). The linear elastic model and the resulting Mohr stress circle for such a case is indicated in Figure 7.2 (Theyse and Muthen, 2000, Theyse *et al.*, 1996a).

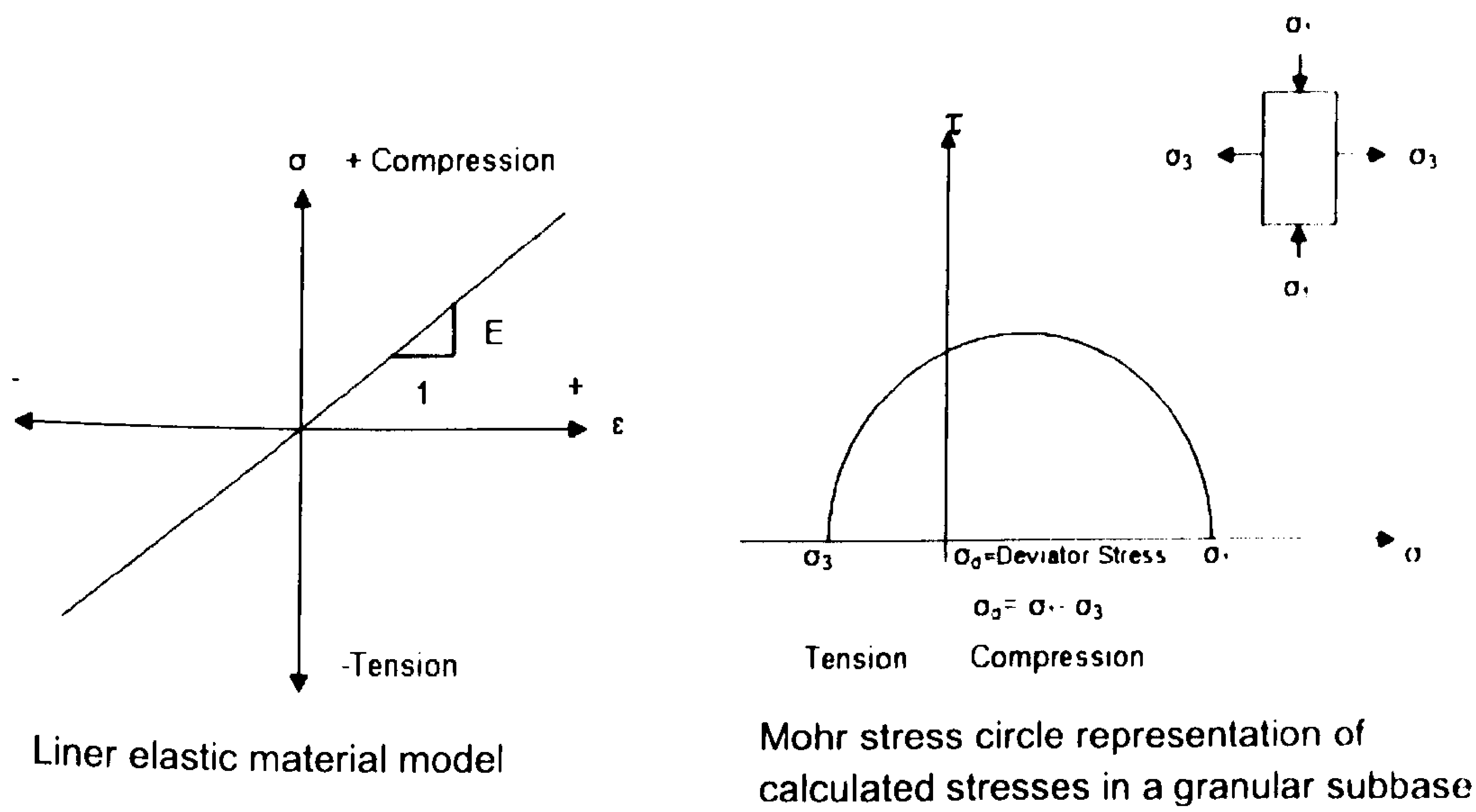
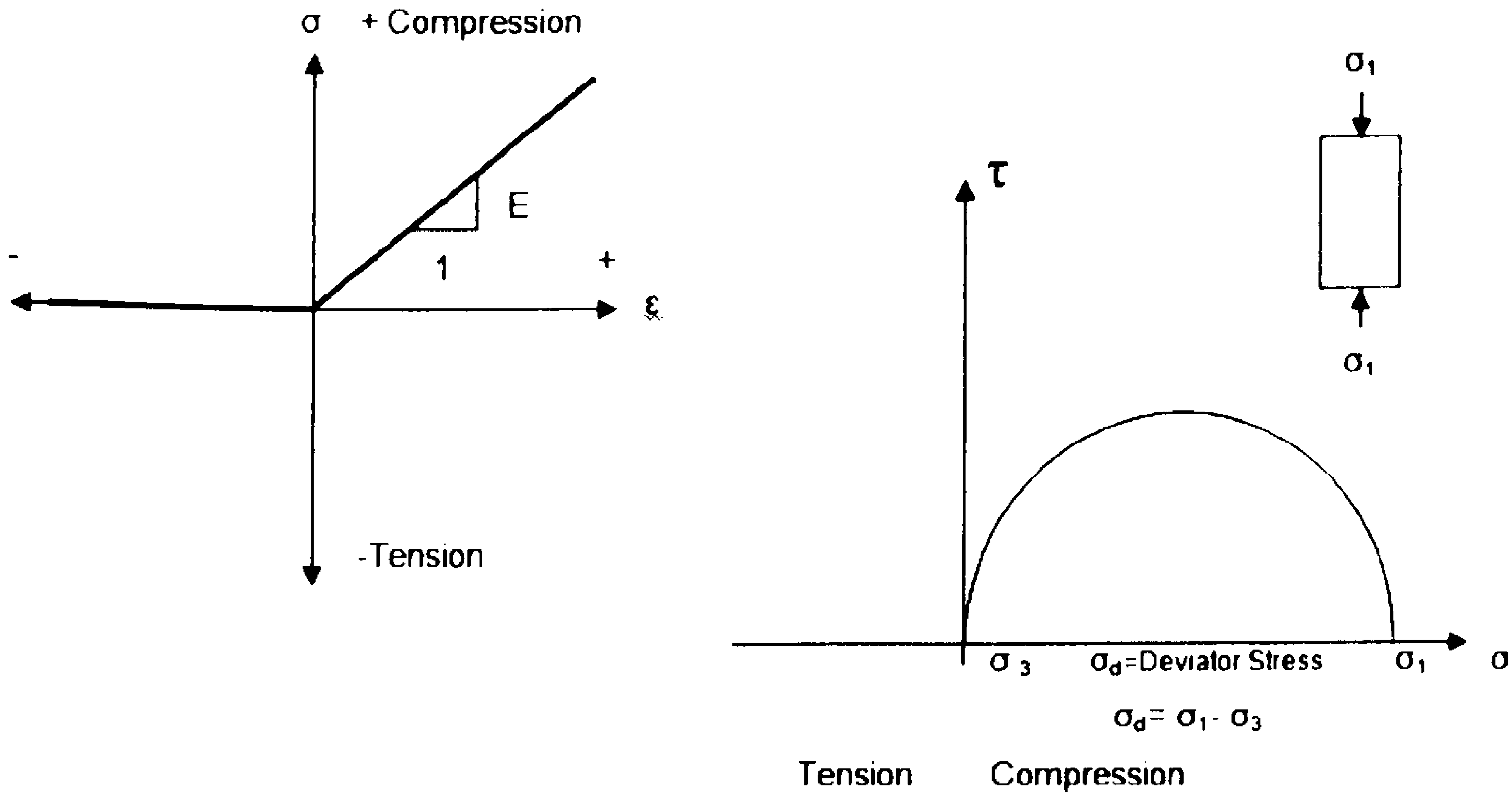


Figure 7.2 Conventional linear elastic material model with the resulting stress state in granular subbases (After Theyse *et al.*, 1996a)

An interim solution is not to allow any tensile stress to develop in granular materials. If a tensile minor principle stress is calculated in a granular material, the value is set equal to zero. What this implies in practice is that the granular layer will only carry loading in compression. If the minor principal stress is set equal to zero, a rearrangement of stresses will take place to transfer the loads by compression. The major principal stress is therefore adjusted under the condition that the deviator stresses remains constant. The Mohr circle is in effect shifted by this procedure as indicated in Figure 7.3. Although this tentative adjustment of stresses has not been proven theoretically, it does provide more meaningful pavement designs compared to proven practice. The ultimate solution would, however, be to use a material model as illustrated in Figure 7.3 rather than the model in Figure 7.2 (Theyse and Muthen, 2000, Theyse *et al.*, 1996a).



No tension elastic material model

Mohr stress circle representation of adjusted stresses in a granular subbase

Figure 7.3 linear elastic material model with the resulting stress state in granular subbases (After Theyse *et al.*, 1996a)

The process starts with the load and material characterization which includes layer thickness and elastic material properties for each layer in the pavement structure. The structural analysis usually involves a linear elastic, static analysis of the multi layer system. Results of the pavement response to the applied loading condition are expressed in terms of stresses and strains at critical point in the pavement structure (Theyse *et al.*, 1996b).

7.3.1 Failure Modes and Critical Parameters for Granular Materials

Researches cited by Liebenberg (2003) showed that granular layers fail in shear when the shear strength of the layer is exceeded. A safety factor against shear failure for granular materials can be derived from the Mohr-Coulomb theory. This safety factor is for static loading and represents the ratio of the material shear strength to the applied stresses responsible for shear. The major and minor principal stresses (σ_1 and σ_3) at the mid-point of the layer are expressed as the critical parameters used in calculating the safety factor. Cohesion, internal friction, moisture regime and stress condition of the material are required to calculate the safety factor using Equation 7.1 (Liebenberg, 2003).

$$F = \frac{\sigma_3 .K.\left(\tan^2(45+\frac{\varphi}{2})-1\right)+2K.C.\tan(45+\frac{\varphi}{2})}{\sigma_1 - \sigma_3}$$ **(Equation 7.1)**

F= Safety factor against shear failure

σ_1 and σ_3 =Calculated major and minor stresses acting in the middle of the layer

C= Cohesion (kPa)

φ = Angle of internal friction (degree)

K = Constant relating to level of saturation

The factor K depends on the level of saturation and the values suggested by Maree (1982). These values were 0.6 for highly saturated conditions (saturation of around 90 percent), and 0.95 for normal to dry moisture conditions (0.45 percent saturation). Theyse and Muthen (2000) redefined this factor for low, moderate and dry moisture conditions as shown below:

a) 0.95 for normal moisture conditions

b) 0.8 for moderate moisture conditions

c) 0.65 for wet (saturated) conditions

A safety factor of smaller than 1 demonstrates that the shear stress is more than the shear strength of the aggregate and rapid shear failure occur under static loading. Under real life dynamic loading conditions, which typically occur on a pavement, the applied shear stress will only exceed the shear strength for a short period of time and shear failure will not happen under one or two load repetitions. The shear failure or deformation will gradually accumulate under a number of load repetitions. The rate of shear deformation of the granular layer is influenced by the magnitude of the safety factor. The safety factor or the major and minor principal stresses are referred to as the critical parameters for granular layers. The major and minor principal stresses and hence the safety factor must be calculated at the mid-depth of granular layers like subbase (Theyse and Muthen, 2000).

7.4 Pavement Modeling Using KENLAYER™ Software

Pavement modeling with KENLAYER™ software is used to predict pavement performance in order to compare the response of different materials based on resilient modulus obtained from laboratory tests as discussed in Chapter 6. Since the objective of this study is to predict the performance of pavement subbase layers, the pavement layer thickness was kept constant as shown in Figure 7.4

with 12 cm for asphalt surface, 15cm for base layer and 15cm as subbase layer. The asphalt surface, base layer and subgrade was assumed have the resilient moduli of 400000 psi, 30000 psi and 7000 psi respectively. The Poisson ratio of all layers was assumed 0.35.

The pavement simulation was performed using the KENLAYER software developed by Huang (2004).

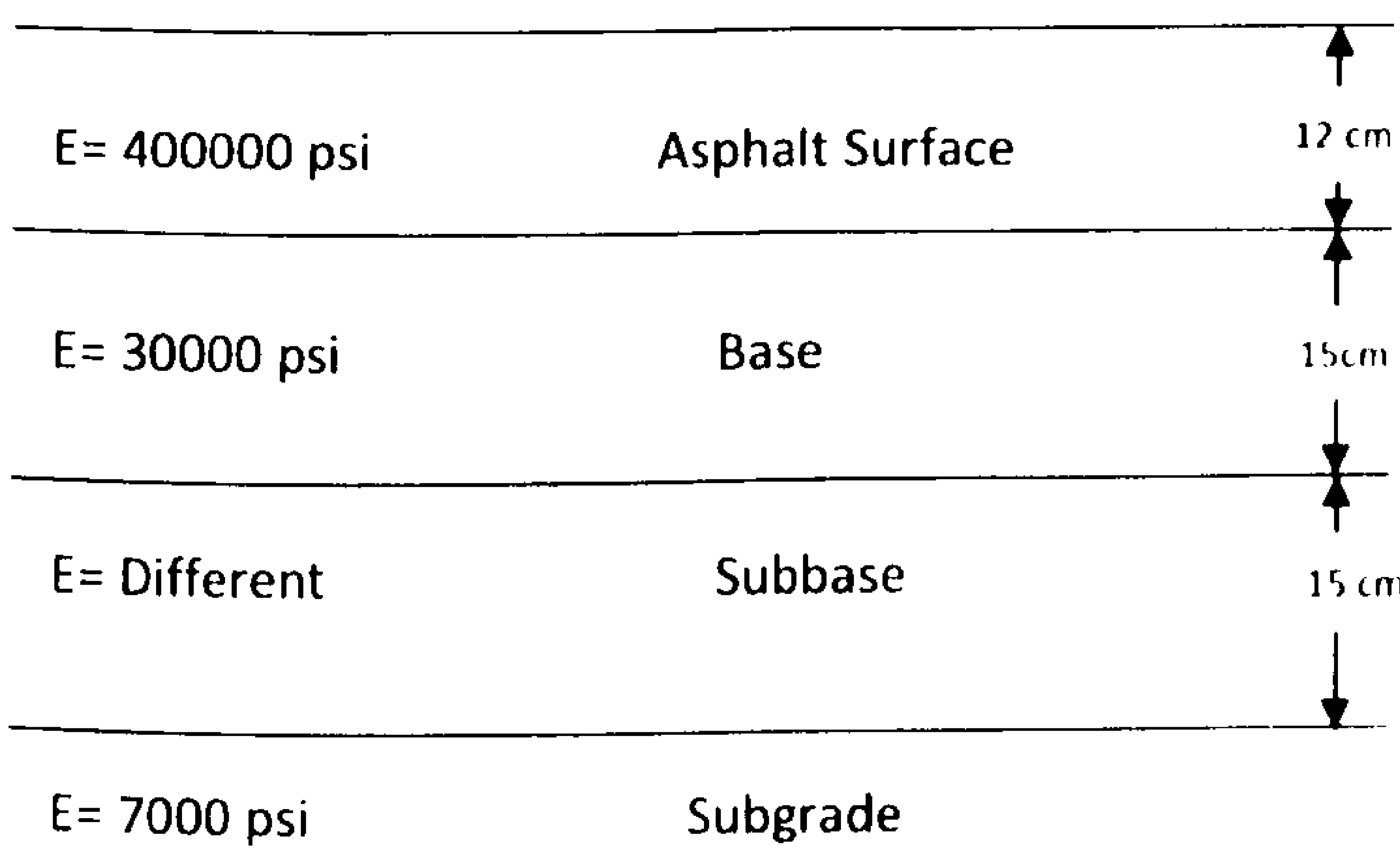


Figure 7.4 Pavement configuration used in KENLAYER^{TM} computer program

The input loading was as illustrated in Figure 7.1 and was chosen a single axle load from a single wheel having a circular contact area defined by a radius of 151mm (5.945 in). The wheel load of 40 kN which will create the contact tyre pressure of 80.991 kpsi. Considering the voluminous nature of the modeling data, and to avoid monotony, the input and output values from calculations are presented in appendix B. In the computer model, the mixes of T-1, T-2 and T-3 were represented as a subbase under a specified wheel load of 40 kN. Outputs of outputs of: (a) vertical displacement, (b) vertical stress, (c) vertical strain, (d) principal stress, (e) principal strain (f) shear stress. Each of these quantities was calculated at the following radial distances (expressed in multiples of the radius, R, of the contact area) away from the circular loaded area of the wheel: (i) 0, (ii) 0.5R, (iii) R and (iv) 1.5R.

As shown in Table 7.2, the material mixes T-1, T-2, T-3 were modeled in both soaked and unsoaked conditions with the corresponding CBR which were correlated to resilient moduli in Chapter 6.

Table 7.2 Properties of mixes for KENLAYER™ modeling

Mix	CBR	Resilient Modulus (psi)
T- 1	Unsoaked CBR=50.15%	17500
	Soaked CBR=31%	14800
T- 2	Unsoaked CBR=71.14%	18750
	Soaked CBR=42%	17250
T- 3	Unsoaked CBR=28.59%	14700
	Soaked CBR=25%	14000

The required input values of ϕ' and c' (see Table 5.4) were obtained from the results of shear box tests and used to calculate the safety factor.

7.5 Results of the Pavement Modeling

7.5.1 Mixes in unsoaked condition

The principal stresses σ_1, σ_3 were calculated from KENLAYER™ software for different mixes and safety factor (F) according to Equation 7.1 are presented in Table 7.3. The calculated CBR in unsoaked condition is the CBR corresponding to the OMC and is considered in the model as moderate moisture with K=0.8.

Table 7.3 Principal stresses and safety factors (unsoaked conditions)

Mix	CBR	K	σ_1(psi)	σ_3(psi)	F
T- 1	50.15	0.8	7.703	1.208	2.805
T- 2	71.14	0.8	7.774	1.217	4.614
T- 3	28.59	0.8	7.525	1.184	2.141

The computed results of vertical displacements at four points (radial distances 0, 0.5R, R, and 1.5R) are shown in Table 7.4 and Figure 7.5:

Table 7.4 Vertical displacements (unsoaked conditions)

Mix	Vertical displacement (in)			
	0R	0.5R	1R	1.5R
T-1	0.03175	0.0313	0.03004	0.02821
T-2	0.03172	0.03127	0.03001	0.02819
T-3	0.03182	0.03137	0.0301	0.02828

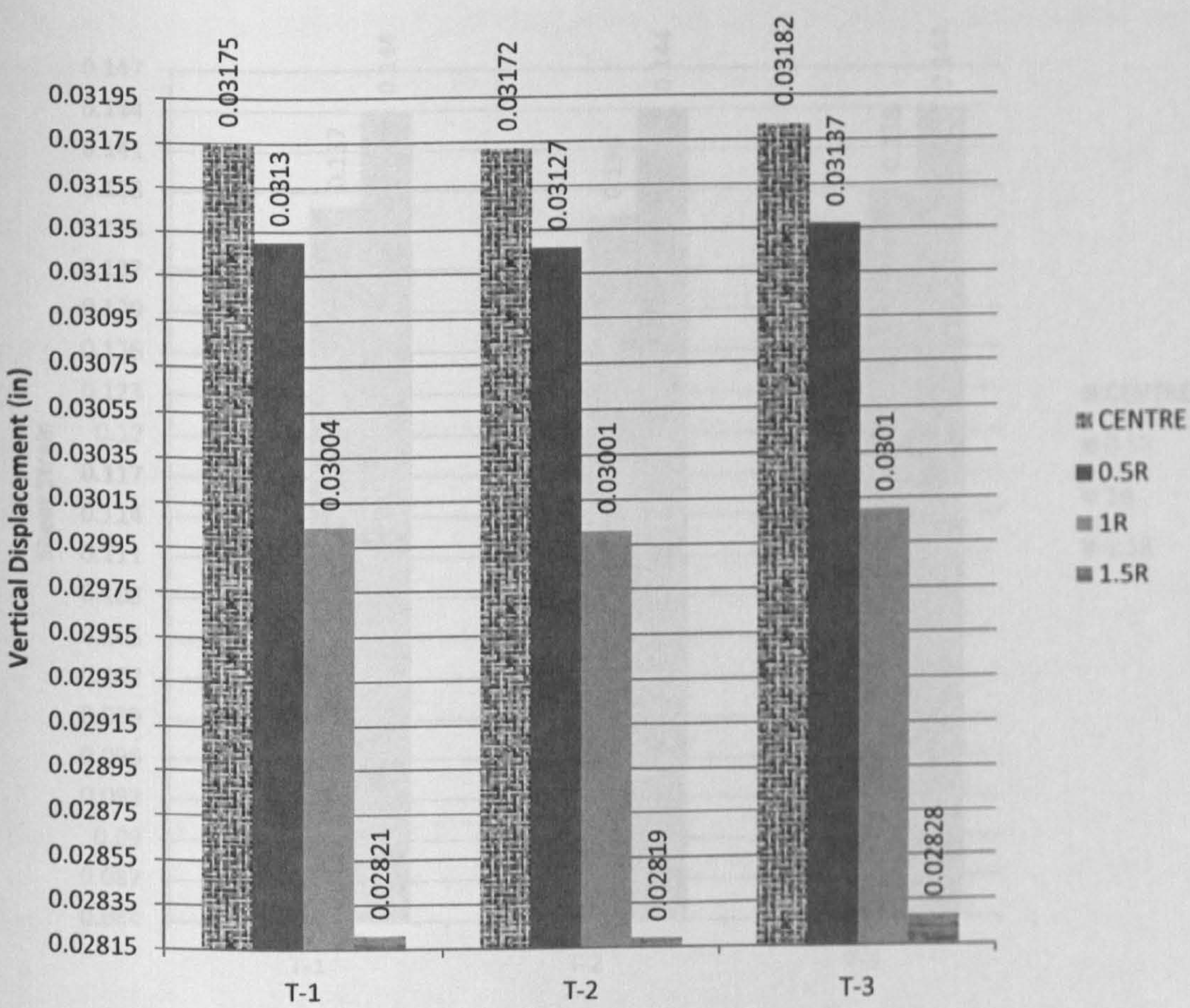


Figure 7.5 Comparison of the vertical displacements for unsoaked mixes

The calculated shear strains at various points are shown in Table 7.5 and Figure 7.6. When the load is applied over a single circular area, the most critical stress, strain, and deflection occur under the center of the area on the axis of symmetry, where shear stress τ_{rz}=0 and radial stress σ_r is equal to tangential stress σ_t ,so σ_r and σ_t are the principal stresses. But at radial distances of 0.5R, R, and 1.5R the strain state, being different from that along the axis of symmetry, is illustrated in Table 7.5 and Appendix B.

Table 7.5 Shear strains (unsoaked conditions)

Mix	Shear Strain (in)			
	0R	0.5R	1R	1.5R
T-1	0	0.0854	0.137	0.144
T-2	0	0.0851	0.136	0.144
T-3	0	0.086	0.138	0.144

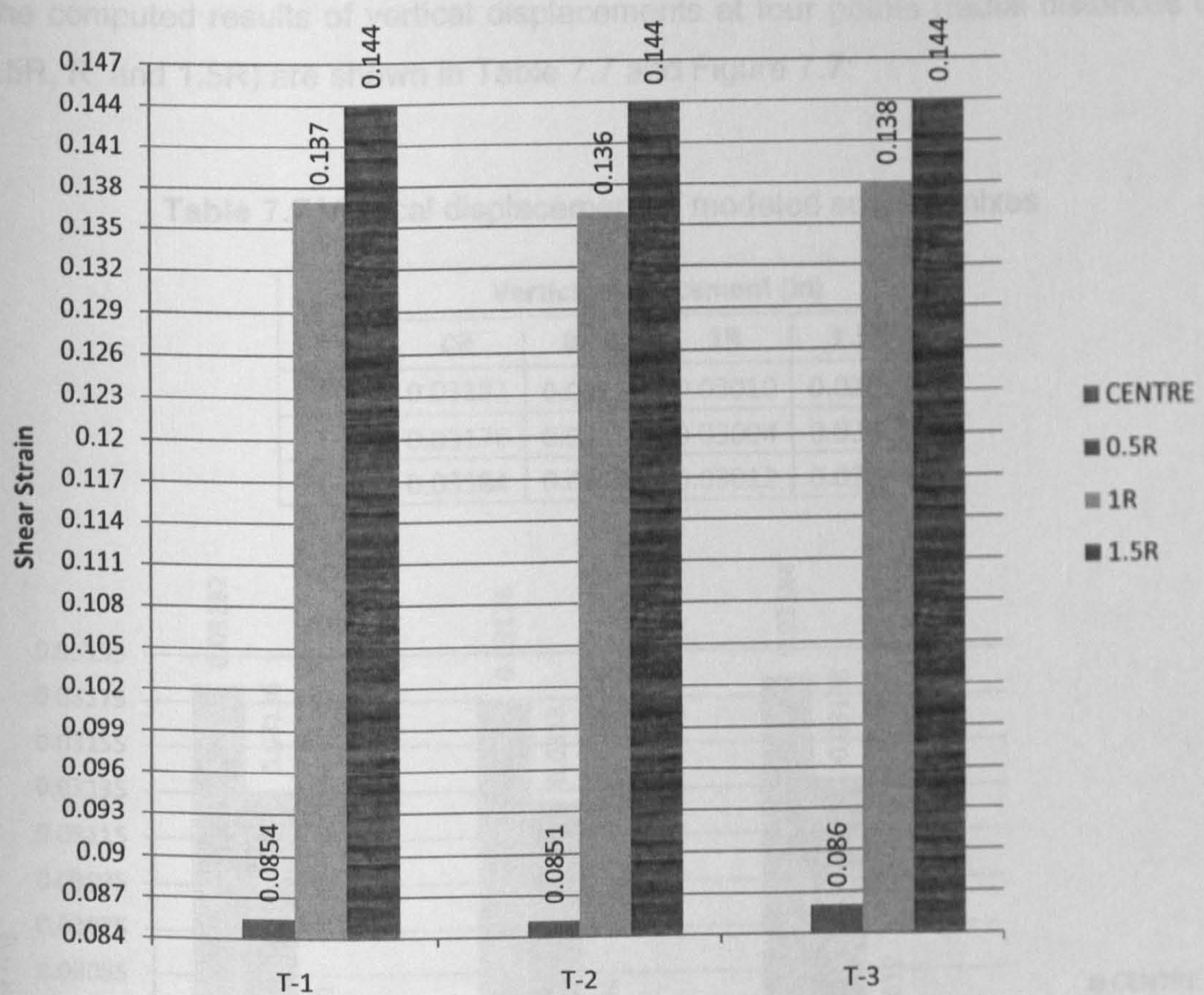


Figure 7.6 Comparison of shear strains in unsoaked conditions

As can be seen from Table 7.3 and Figure 7.5, T-2 had the highest safety factor and lowest vertical displacement and shear strain, followed by T-1 and T-3 in unsoaked and dry conditions. This behavior can be confirmed by performance potential based on stiffness in Table 6.2. The CBR and resilient modulus of T-2 is the highest in dry condition which followed by T-1 and T-3.

7.5.2 Mixes in soaked condition

The principal stresses σ_1, σ_3 were calculated from KENLAYER™ software for different mixes and safety factor (F) according to Equation 7.1 are presented in Table 7.6. The calculated CBR in soaked condition is the CBR corresponding to the OMC and is considered in the model as saturated conditions with K=0.65.

Table 7.6 Principal stresses and safety factors (soaked conditions)

Mix	CBR	K	σ_1(psi)	σ_3(psi)	F
T- 1	31	0.65	7.533	1.185	2.323
T- 2	42	0.65	7.688	1.206	3.787
T- 3	25	0.65	7.475	1.177	1.749

The computed results of vertical displacements at four points (radial distances 0, 0.5R, R, and 1.5R) are shown in Table 7.7 and Figure 7.7:

Table 7.7 Vertical displacement of modeled soaked mixes

Mix	Vertical displacement (in)			
	0R	0.5R	1R	1.5R
T-1	0.03182	0.03136	0.03010	0.02827
T-2	0.03176	0.03130	0.03004	0.02822
T-3	0.03184	0.03138	0.03012	0.02829

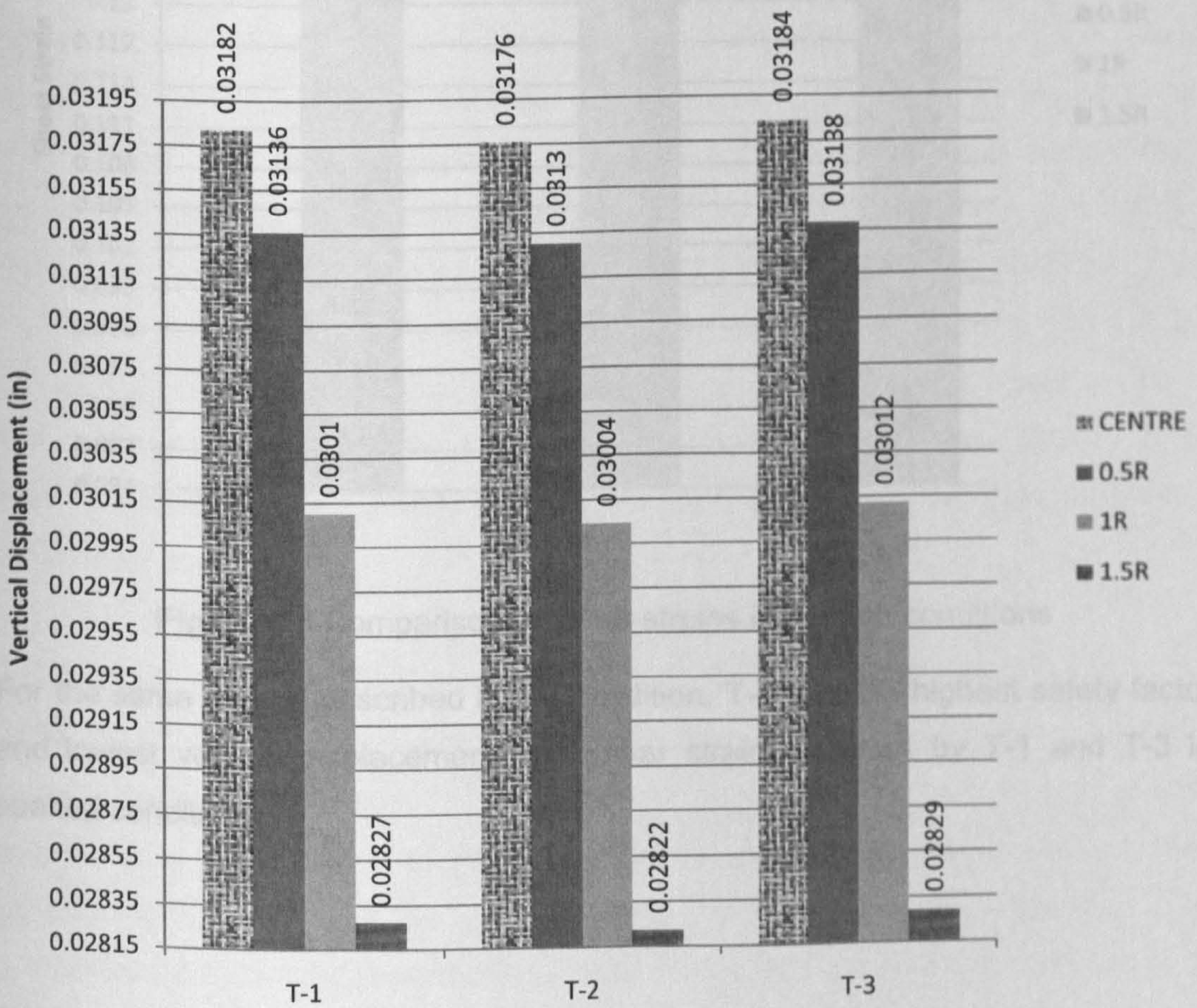


Figure 7.7 Comparison of the vertical displacements for soaked mixes

The calculated shear strains at various points are shown in Table 7.8 and Figure 7.6.

Table 7.8 Shear strain of modeled unsoaked mixes

Mix	Shear Strain (in)			
	0R	0.5R	1R	1.5R
T-1	0	0.086	0.138	0.144
T-2	0	0.0855	0.137	0.144
T-3	0	0.0861	0.138	0.144

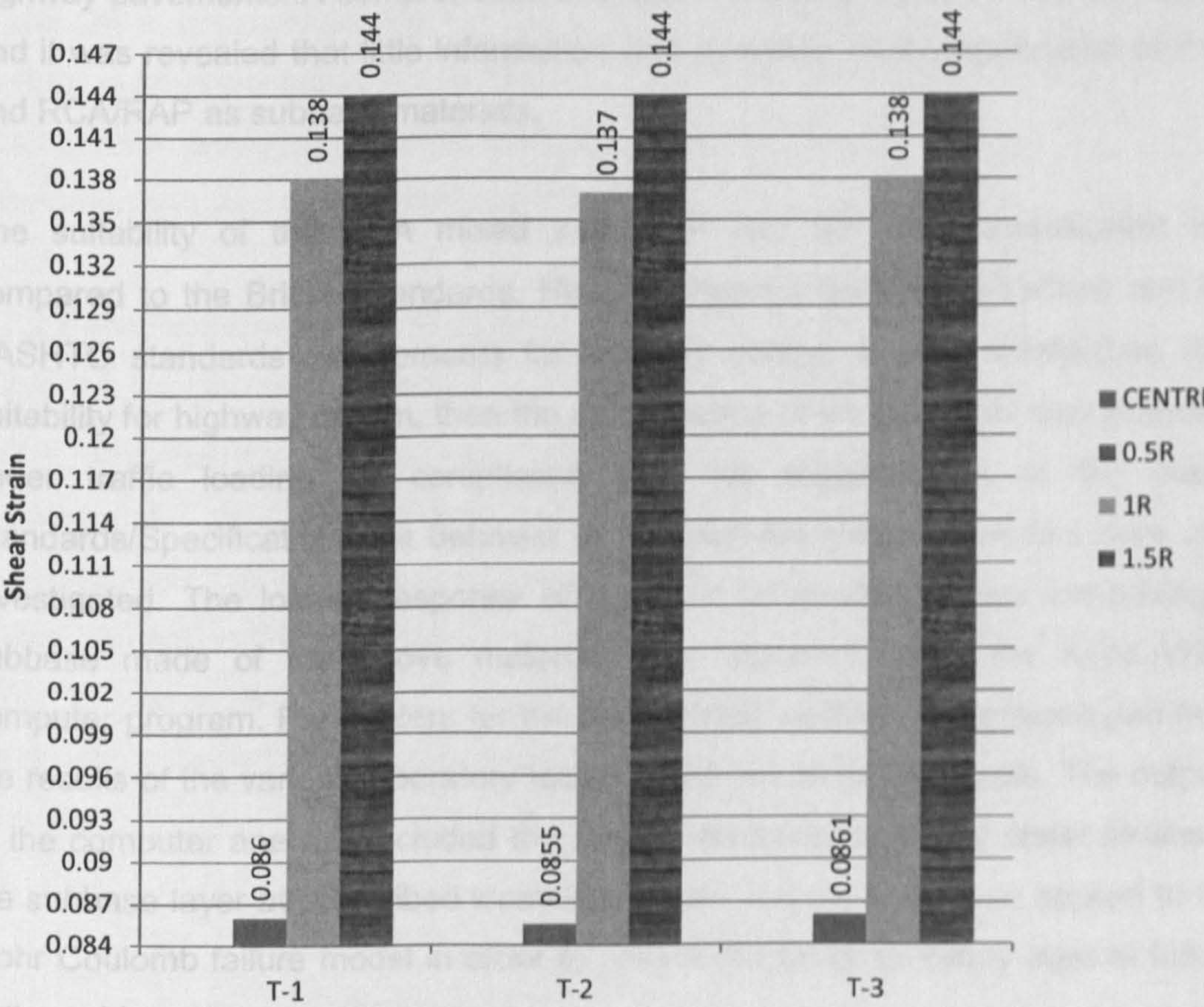


Figure 7.8 Comparison of shear strains in soaked conditions

For the same reason described in dry condition, T-2 had the highest safety factor and lowest vertical displacement and shear strain, followed by T-1 and T-3 in soaked conditions.

CHAPTER 8 FINDINGS AND IMPLICATIONS

8.0 Preamble

At the outset, the research program was developed with key aim of investigating the performance suitability of aggregates formed with different combinations of Recycled Concrete Aggregate (RCA), Reclaimed Asphalt Pavement (RAP) and Natural Aggregates (NA) when used in construction of unbound subbases of highway pavements. A comprehensive review of existing literature was carried out and it was revealed that little information was available on the application of RCA and RCA/RAP as subbase materials.

The suitability of the RCA mixed with RAP and NA were investigated and compared to the British Standards, Highway Agency (HA) specifications and the AASHTO standards requirements for highway design. Having established their suitability for highway design, then the performance of the materials was assessed under traffic loading. In compliance with the requirements of the above Standards/Specifications the behavior of the aforementioned materials were also investigated. The loaded response of a typical pavement system comprising a subbase made of the above materials was modeled using the KENLAYER computer program. Parameters for the KENLAYER analysis were developed from the results of the various laboratory tests carried out on the materials. The outputs of the computer analysis included the vertical displacements and shear strains in the subbase layer at prescribed locations. These outputs were then applied to the Mohr Coulomb failure model in order to predict the factor of safety against failure of the subbase layer.

8.1 Compliance of the tested mixtures with design standards

Both the United Kingdom and Iranian Standards stipulate specific mix design requirements which have to be complied with. The initial aspect of this experimental work was to check the suitability and compliance of the proposed mixes and to benchmark the results against the relevant design standards. The experimental work was reported in Chapter 5 and the evidence of compliance of the proposed mixes with the stated Standards is given in Tables 8.1 and 8.2. In Chapter 5, phase 1 of the experimental programme refers to work carried out in the UK while phase 2 refers work done in Iran.

8.1.1 Comparison of the requirements of the UK and Iranian design standards

There are both similarities and differences in these standards regarding the recommended tests for given materials and situations. For example: (i) the Iranian standards do not prescribe any requirements for water absorption and micro-Deval tests whereas the UK standards do, (ii) the UK standards do not use the sand equivalence test whereas the Iranian standards do, (iii) the UK and Iranian standards specify different minimum subbase CBR values, (iv) the soundness test in the UK is normally carried out using magnesium sulphate whereas in Iran the sodium sulphate solution is used for the same test. However, for the purpose of this research work, both sodium and magnesium sulphate solutions were used in experiments in Iran and as reported in Chapter 5.

8.1.2 Requirements of Standards for the Phase 1 Tests

Table 8.1 lists the various tests performed on the 3 aggregate types and indicates (✓ or ✗) whether or not the materials met the UK design standards. The results showed that, for the listed tests, materials T-1, T-2 and T-3 mixes met the requirements of MCHW Series 800 and the British Standard. The CBR test is also a requirement in the UK design standards, which stipulates a minimum CBR= 30%. This target was not met by material T-3, which had CBR=29% however the compaction performance, as regards the criterion for MDD at the same compaction effort, was higher than that of T-1.

Table 8.1 Requirements vs. Mixes for the Phase 1

Required Tests	T-1	T-2	T-3
Mixture requirement category	✓	✓	✓
Crushed, or broken and totally rounded particles	✓	✓	✓
Resistance to fragmentation - Los Angeles test	✓	✓	✓
Resistance to wear - micro-Deval test	✓	✓	✓
Resistance to freezing and thawing magnesium sulfate soundness	✓	✓	✓
Water absorption	✓	✓	✓
All other BS EN 13242 aggregate requirements	✓	✓	✓
Plasticity Index (Ip)	✓	✓	✓

8.1.3 Requirements of Standards for the Phase 2 tests

Table 8.2 shows the outcome of the phase 2 tests on the nine materials as compared against the requirements of the Iran Highway Asphalt Paving Code. The results show that some of the proposed mixes did not meet the requirements of Iran Highway Asphalt Paving Code.

Table 8.2 Requirements vs. Mixes for the Phase 2

Proposed Mixes	100%NA	80%NA+20%RCA	50%NA+50%RCA	20%NA+80%RCA	100%RCA	20%RAP+80%RCA	50%RAP+50%RCA	80%RAP+20%RCA	100%RAP
Plasticity Index	✓	✓	✓	✓	✓	✓	✓	✓	✓
Liquid Limit	✓	✓	✓	✓	✓	✓	✓	✓	✓
Sand Equivalent value	✓	✓	✓	✓	✓	✓	✓	✓	✓
Los Angeles Abrasion	✓	✓	✓	✓	✓	✓	✓	✓	✓
CBR	✓	✓	✓	✓	✓	✓	✓	✗	✗
Sodium Sulphate Soundness	✗	✗	✗	✗	✗	✗	✗	✗	✓
Magnesium Sulfhate Soundness	✓	✓	✓	✓	✓	✓	✓	✓	✓

More specifically all proposed mixes meet the requirements for Plasticity Index (PI), Liquid Limit (LL), Sand Equivalent value (SE) and Los Angeles Abrasion (LAA) tests. The 100%RAP and 80%RAP+20%RCA mixes had a CBR less than 25% and all mixes except 100%RAP had sodium sulphate soundness values exceeding 12%. It should be borne in mind that the specification of the Iran Highway Asphalt Paving Code was meant for natural aggregates assessment. It is remarkable to reveal that out of the three mixes containing RCA and RAP, the 50%RAP+50%RCA mix had a better performance in the compaction test. A similar compaction test result was obtained for the 50%RAP+50%RCA mix, for the materials used in the UK.

8.2 Performance Assessments

Table 8.3 summarizes the performance assessment findings reported in Chapter 6. Toughness related tests indicated that recycled aggregates, natural aggregates and their reported mix variations are appropriate for use in conditions representing significance levels 1and 2 for use in low and medium traffic conditions in non freezing climates with low and high moisture condition. These subbase materials could be used in freezing climates with low traffic and moisture levels. The results show that blending RCA with virgin and RAP aggregates improves performance (based on the toughness test) but not sufficiently to take the performance to the next higher significance level.

CBR test results and their correlation with resilient modulus were applied for assessing the stiffness performance. The results showed that the tested mixes are appropriate only for level 1(low traffic, non-freezing and low moisture).

From the durability and susceptibility points of view, all aggregates were shown to be appropriate for use in conditions representing significance level 4 for use in extreme traffic and climatic conditions. Therefore these subbase materials would satisfy all significance levels (i.e., high traffic, high moisture and freezing conditions). Using 100%RCA is appropriate for level 4, but adding NA and RAP to RCA could improve their performance based on durability and susceptibility properties.

The correlation between large shear box test and static triaxial test for materials tested in the UK showed that T-1, T-2 and T-3 are appropriate for significance levels of 1 (low traffic, low moisture and no freezing) and 2(medium traffic and no freezing).

Table 8.3 Performance of the mixes

Proposed Mixes	Toughness	Stiffness	Durability	Frost susceptibility	Shear strength
T-1	1, 2	1	1,2,3,4	1,2,3,4	1, 2
T-2	1, 2	1	1,2,3,4	1,2,3,4	1, 2
T-3	1, 2	1	1,2,3,4	1,2,3,4	1, 2
100%NA	1, 2	1	1,2,3,4	1,2,3,4	
80%NA+20%RCA	1, 2	1	1,2,3,4	1,2,3,4	
50%NA+50%RCA	1, 2	1	1,2,3,4	1,2,3,4	
20%NA+80%RCA	1, 2	1	1,2,3,4	1,2,3, 4	
100%RCA	1, 2	1	1,2,3,4	1,2,3,4	
20%RAP+80%RCA	1, 2	1	1,2,3,4	1,2,3,4	
50%RAP+50%RCA	1, 2	1	1,2,3,4	1,2,3,4	
80%RAP+20%RCA	1, 2	1	1,2,3,4	1,2,3,4	
100%RAP	1, 2	1	1,2,3,4	1,2,3,4	

8.3 Findings from Computer Modeling

Modeling results with KENLAYER™ for samples T-1, T-2 and T-3 showed that application of T-2 had the highest safety factor against subbase failure and also the least vertical displacement shear strain under traffic loading. The next best material was T-1 whereas T-3 had the lowest safety factor and largest vertical displacement and shear strain. Thus it can be concluded that adding NA to RCA will increase the safety factor and decrease strains under traffic loading but adding RAP to RCA will have converse effects.

8.4 Overall Conclusions

- T-1(100%RCA), T-2(100%NA) and T-3(50%RAP+50%RCA) were found to satisfy the requirements of MCHW.
- All mixes tested in Iran met the requirements for Atterberg limits, sand equivalence, Los Angeles abrasion and magnesium sulfate soundness. None of the RCA mixes complied with sodium sulfate soundness test.
- The presence of RAP in RCA mixes reduced the CBR values but increased the MDD, an important parameter in field construction. In particular, the mix type: 50%RAP+50%RCA gave the highest MDD. The comparison of CBR test results for the materials tested in Iran and the UK indicated that the source of the material had a great effect on the CBR value. In terms of compaction and CBR requirements, the 50%RAP+50%RCA mix was demonstrated as suitable for unbound subbase application.

- The mixes of RCA with NA have greater CBR values and MDD than the same percentage in the mixes of RCA with RAP with lower OMC.
- From the viewpoint of durability and frost susceptibility, all the materials investigated were applicable to the highest significance level. However, from the point of view of stiffness, the materials were applicable to the lowest significance level. Turning to the toughness and shear strength viewpoints, the same materials fell in the middle significance levels.
- Modeling results indicate that increased replacement of RCA with RAP in a subbase layer decreases the factor of safety against failure. Consequently the number of allowable wheel load repetitions is decreased. Results also showed that, under the same load, vertical displacements and shear strains increase with increasing percentage of RAP in a RAP/RCA mixture.

8.5 Contribution to Knowledge and Practice

The value of this research and its originality in the area of pavement and sustainability can be demonstrated in a number of ways: (1) Assessment of different sources of recycled aggregates and comparative analysis of their behavior, (2) Evaluation of the technical feasibility of recycled aggregates for application in highway construction in developing countries: a case study of Iran, (3) Critical discussion and analysis of material behavior relating to practice. (4) Extension of the use of RAP and RCA in blended form rather than in separately as has been used to date, (5) Application of Mohr- Coulomb failure model for recycled aggregates.

8.6 Recommendations for Future Work

a) To extend further the verification of the performance related results, more controlled tests should be conducted both in the laboratory and in the field, under carefully varied conditions. This should take into account layer thickness, water content, density and stresses resulting from traffic loading.

b) It is also recommended that different subbase mixes containing RCA with NA and RCA with RAP be investigated. In fact some initial exploratory work towards this objective was carried in collaboration with TRL on performance-related tests. Further work should involve discussion with experts from TRL to extend the testing to include construction of a trial pavement section to allow a series of performance related tests to be undertaken. Based on the initial discussions, Appendix B includes the details of the package suggested by TRL but not accomplished owing to the financial constraints at the time of undertaking this research. The consensus, however, was the additional work was merited and is therefore recommended to be carried at a more opportune time in the future.

c) More comprehensive performance testing should be carried out using tube suction tests for frost susceptibility, static triaxial and repeated load tests for shear strength and resilient modulus for stiffness evaluation. These tests could not be done in the present project due to a lack of apparatus and time. In the absence of these tests, the author devised a method of using the current experimental results to correlate required parameters as described in Chapter 7.

d) It is recommended further research geared towards improving mix strength be carried out in order to definitively explain the observation that adding RAP to RCA decreased the water absorption, improved compaction behavior and decreased the strength.

e) The mechanism of the shrinkage of the mixes including RAP and RCA after soaking in the water needs a more thorough study. It is recommended that by installing appropriate instrumentation and using data loggers, the behavior of this type mixes should be studied very carefully. The use of Labview ® instrument and software is suggested.

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APPENDIX A

Appendix A includes large shear box test results for T-1, T-2 and T-3.

A.1. Test results of T-1

In specimen No.1 of T-1 the normal stress was 50kPa. The corresponding maximum shear stress, horizontal displacement and height change were measured to be 75kPa, 11.7mm and 1.2mm, respectively (Figures A.1 and A.2).

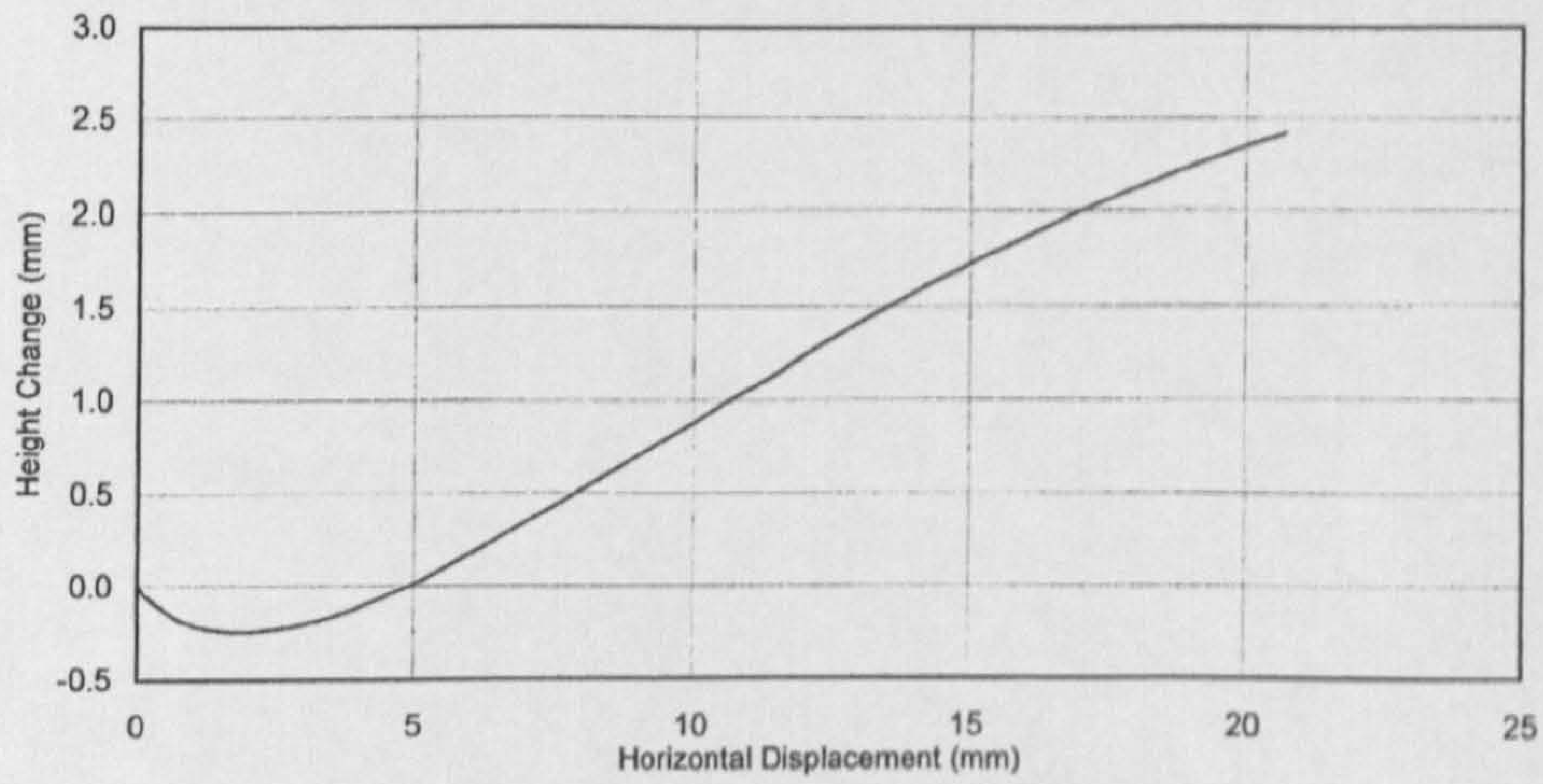

Figure A.1 Height change versus horizontal displacement in specimen No.1 of T-1

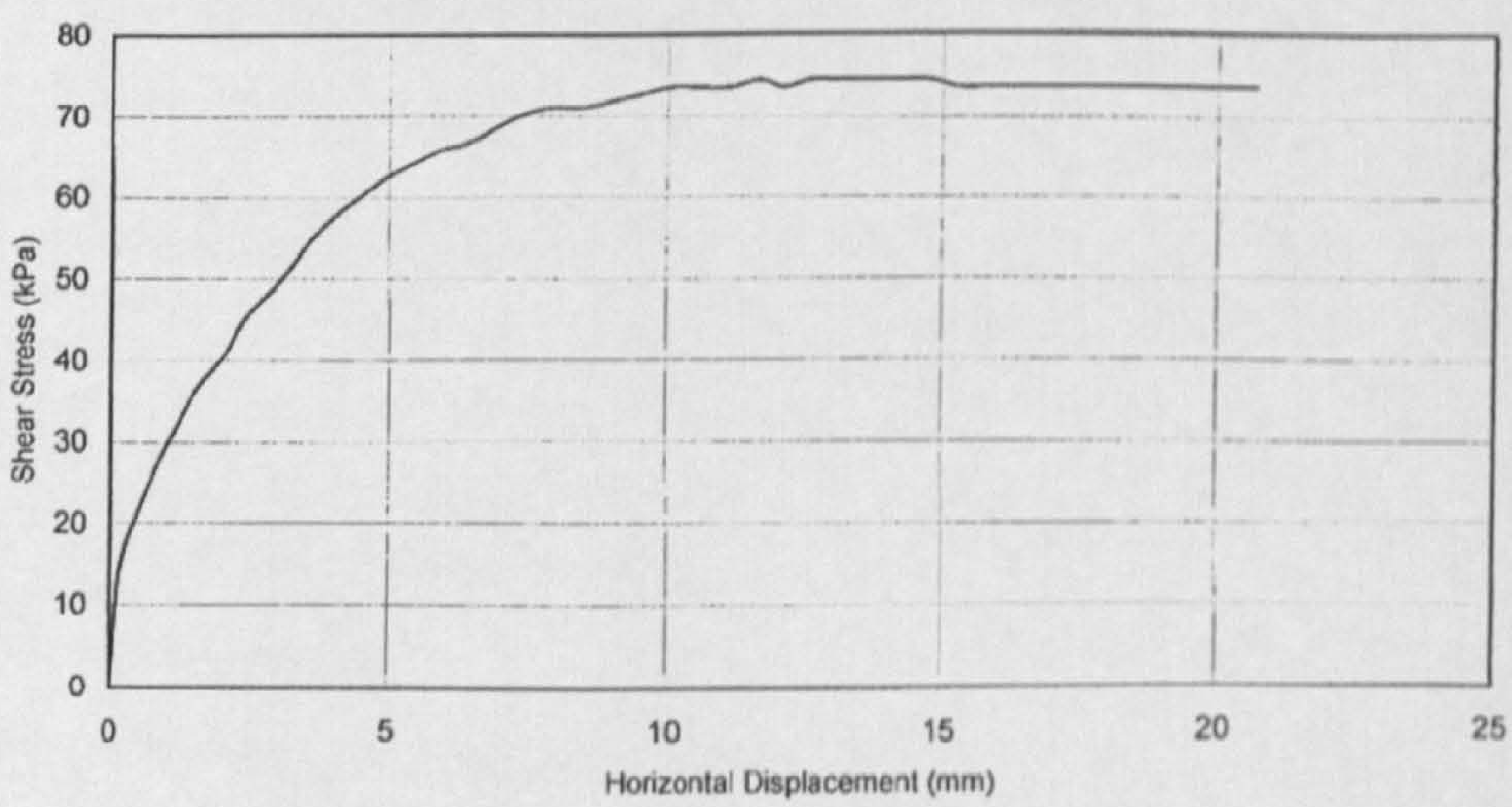

Figure A.2 Shear stress versus horizontal displacement in specimen No.1 of T-1

In specimen No.2 of T-1 the normal pressure was 100kPa. The corresponding maximum shear stress, horizontal displacement and height change were measured to be 115kPa, 20.8mm and 0.8mm, respectively (Figures A.3 and A.4).

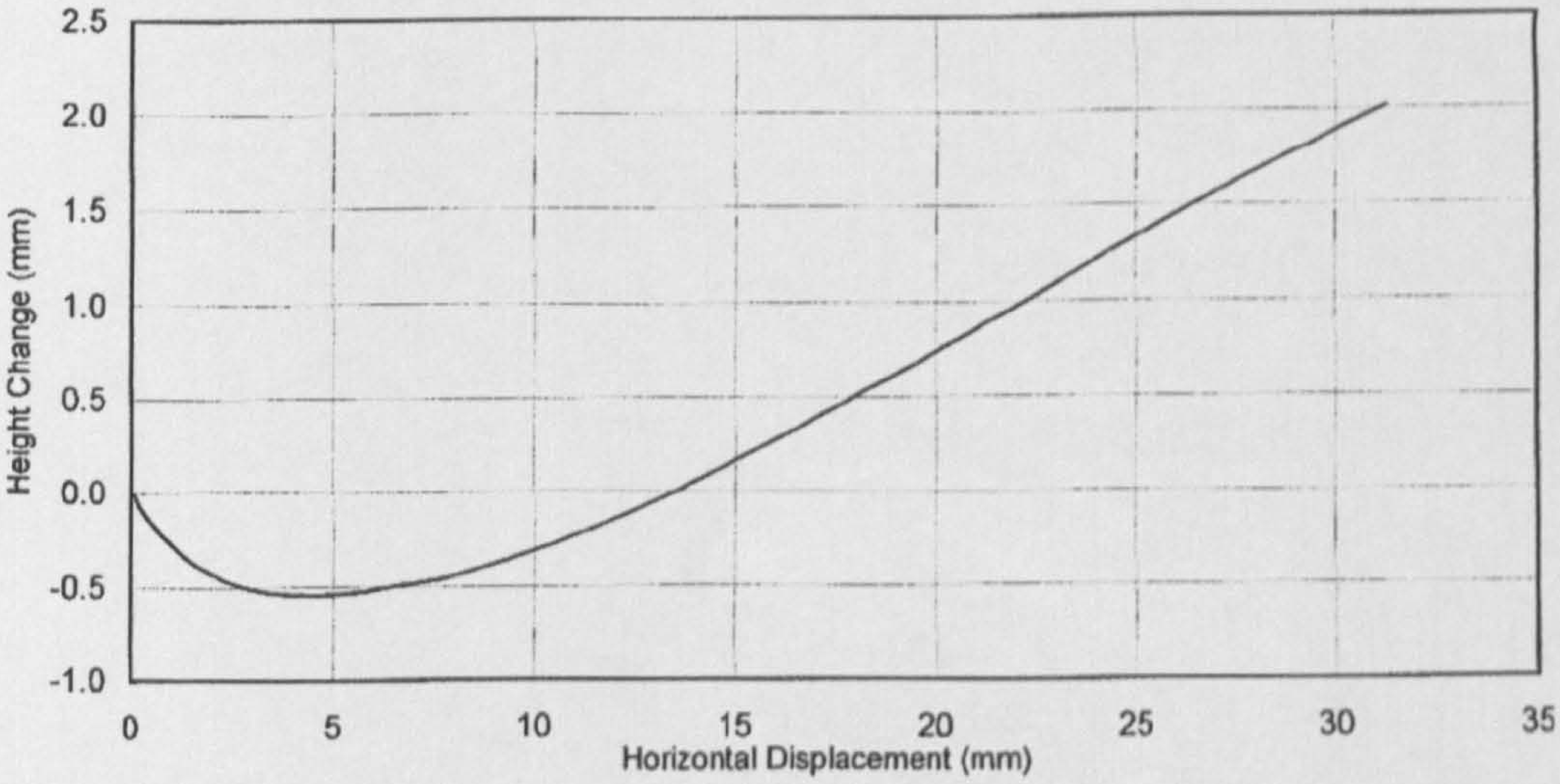

Figure A.3 Height change versus horizontal displacement in specimen No.2 of T-1

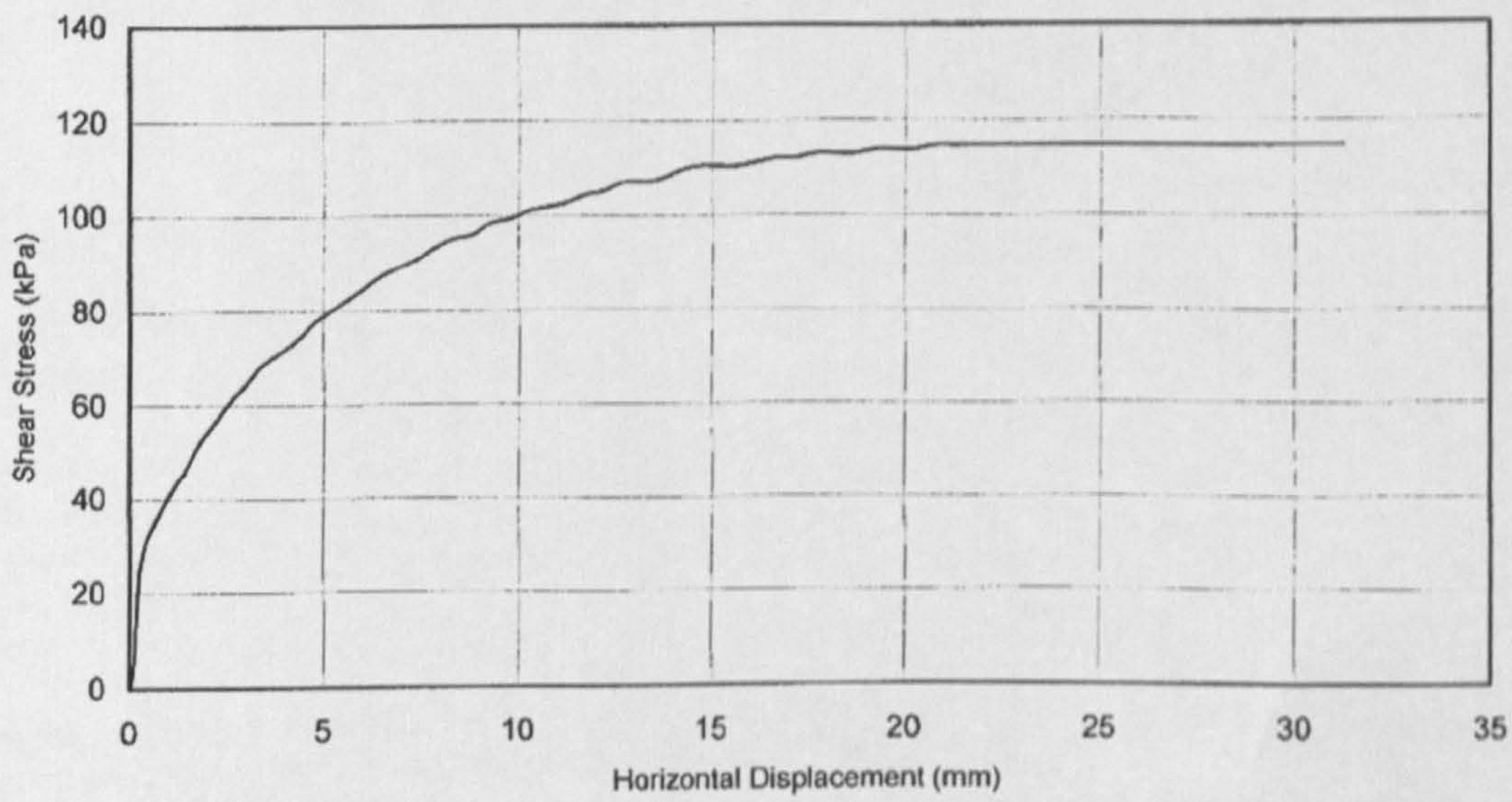

Figure A.4 Shear stress versus horizontal displacement in specimen No.2 of T-1

In specimen No.3 of T-1 the normal pressure was 200kPa. The corresponding maximum shear stress, horizontal displacement and height change were measured to be 207kPa, 17.8mm and 0.8mm, respectively (Figures A.5 and A.6).

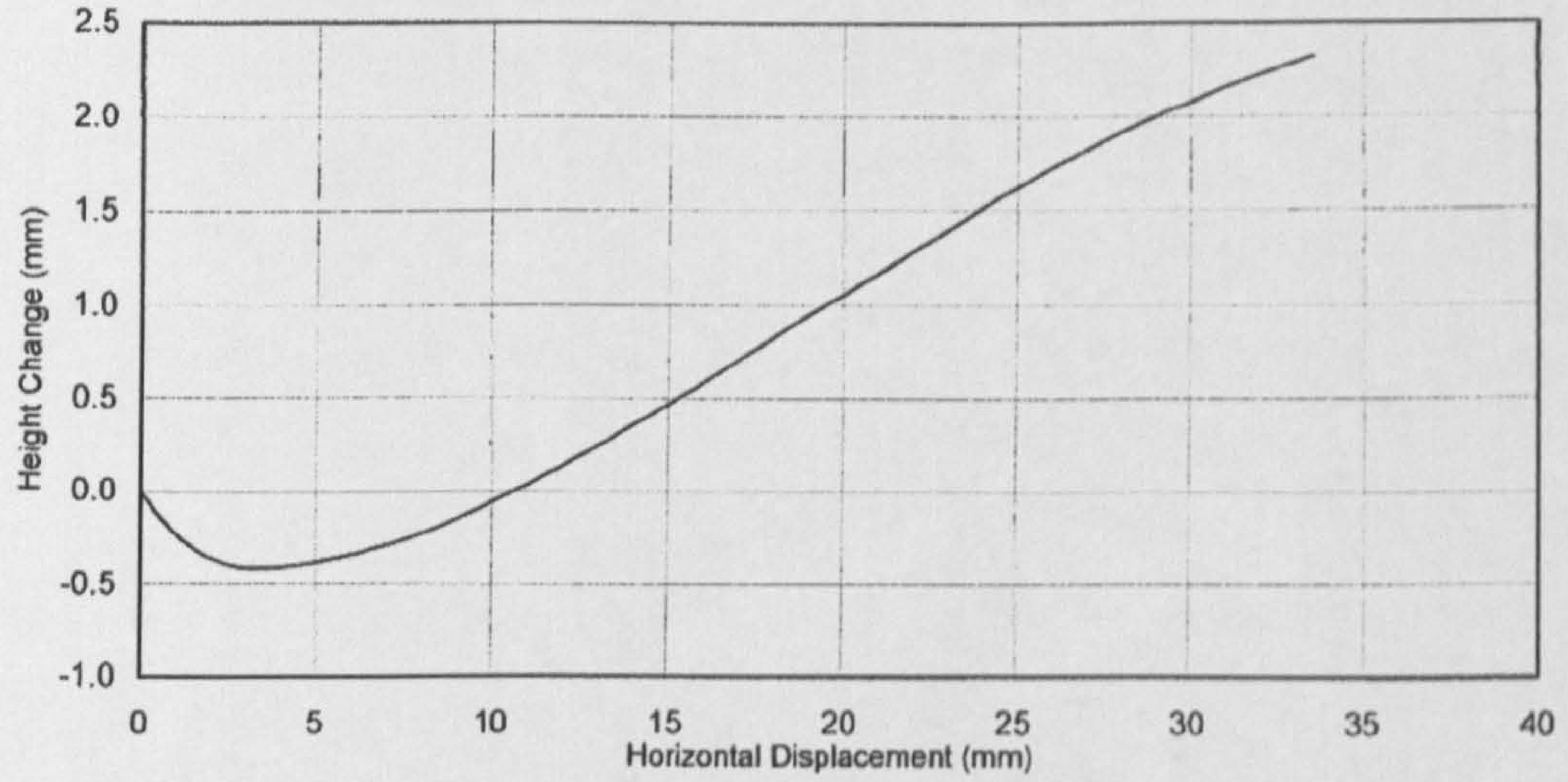

Figure A.5 Height change versus horizontal displacement in specimen No.3 of T-1

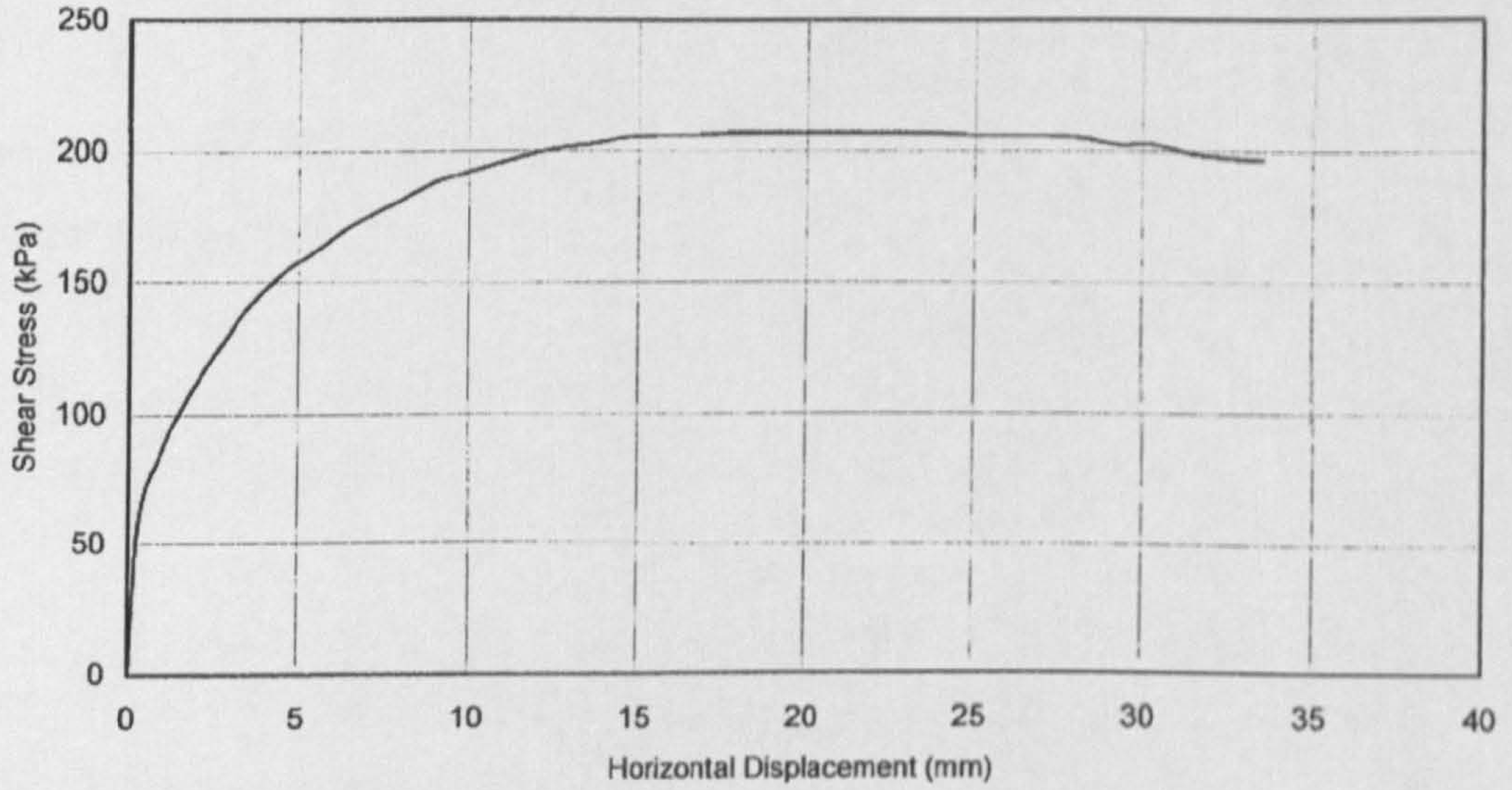

Figure A.6 Shear stress versus horizontal displacement in specimen No.3 of T-1

A plot of peak shear stress versus normal stress from three specimens gives the effective cohesion c' and effective angle of shearing resistance, ϕ'. For T-1, the results were found to be {c'=28kPa, ϕ'=41.5^{o}} as seen in Figure A.7.

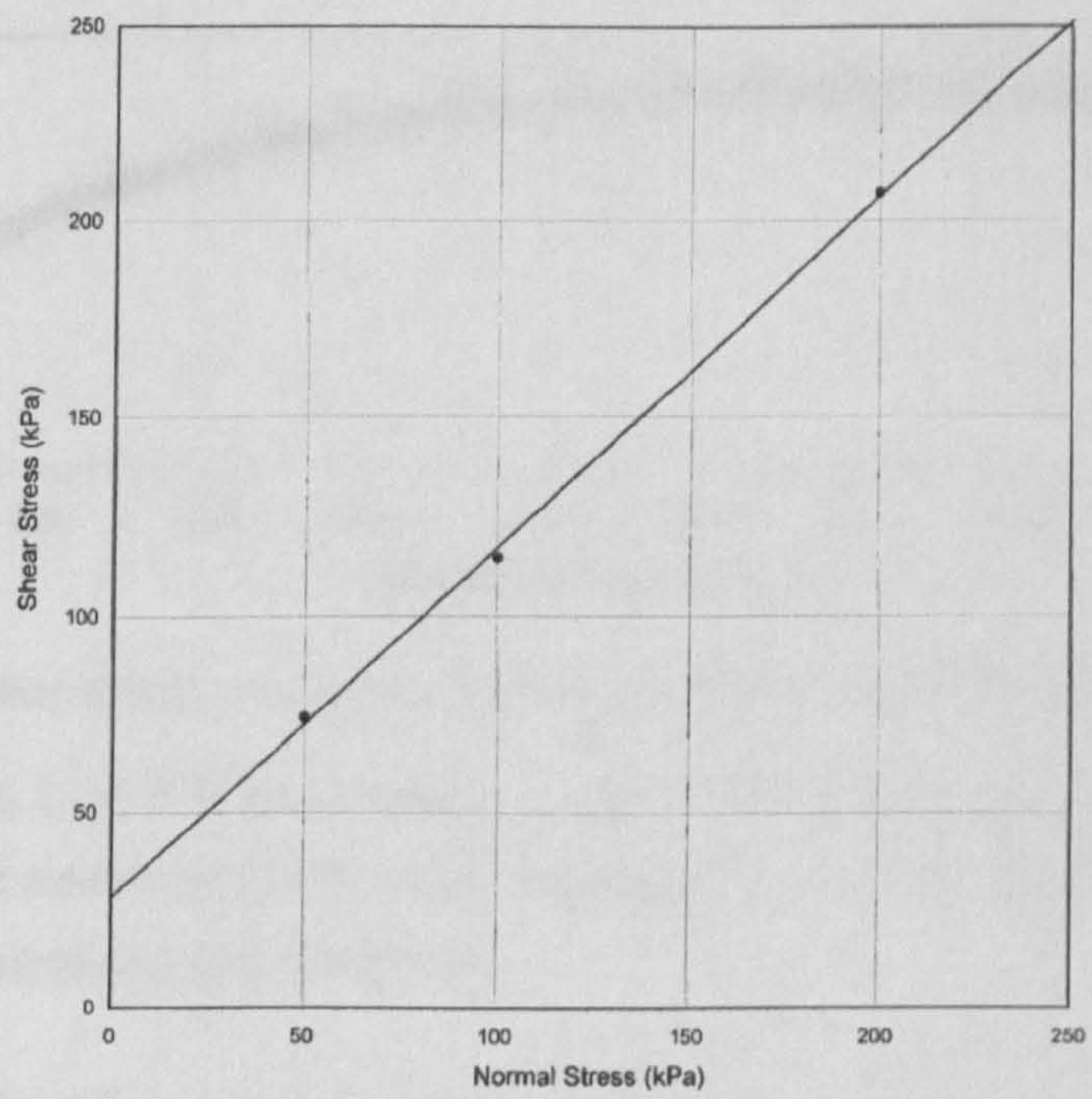

Figure A.7 Shear Stress versus Normal Stress in T-1

A.2 Test results of T-2

In specimen No.1 of T-2 the normal pressure was 50kPa. The corresponding maximum shear stress and horizontal displacement were measured to be 90kPa and 25mm, respectively (Figure A.8).

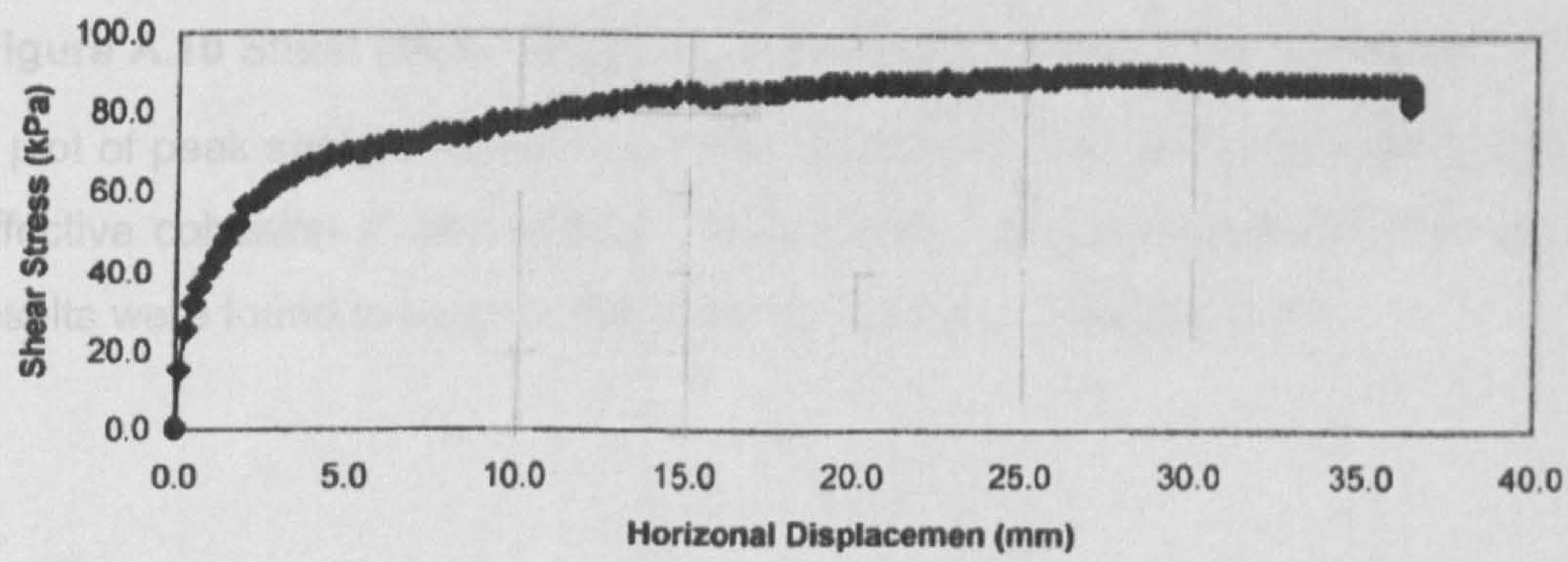

Figure A.8 Shear stress versus horizontal displacement in specimen No.1 of T-2

In specimen No.2 of T-2 the normal pressure was 100kPa. The corresponding maximum shear stress and horizontal displacement were measured to be 157kPa and 26mm, respectively (Figure A.9).

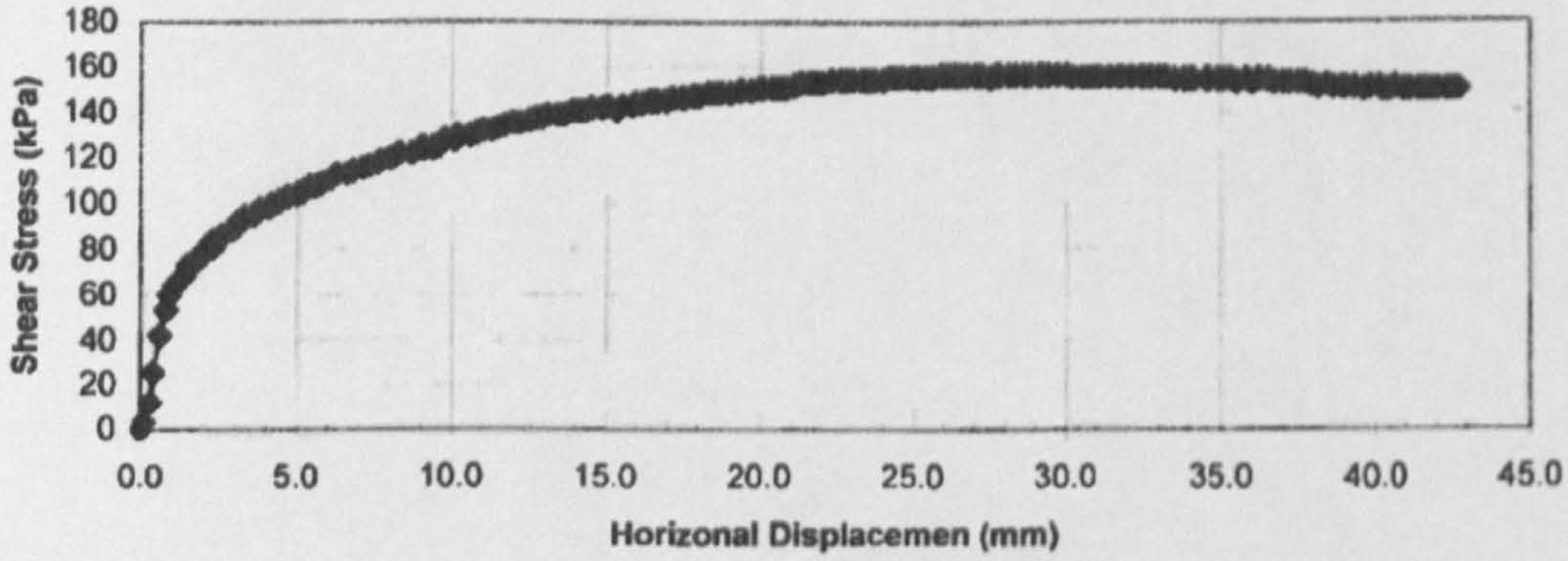

Figure A.9 Shear stress versus horizontal displacement in specimen No.2 of T-2

In specimen No.3 of T-2 the normal pressure was 200kPa. The corresponding maximum shear stress and horizontal displacement were measured to be 249kPa and 26mm, respectively (Figure A.10).

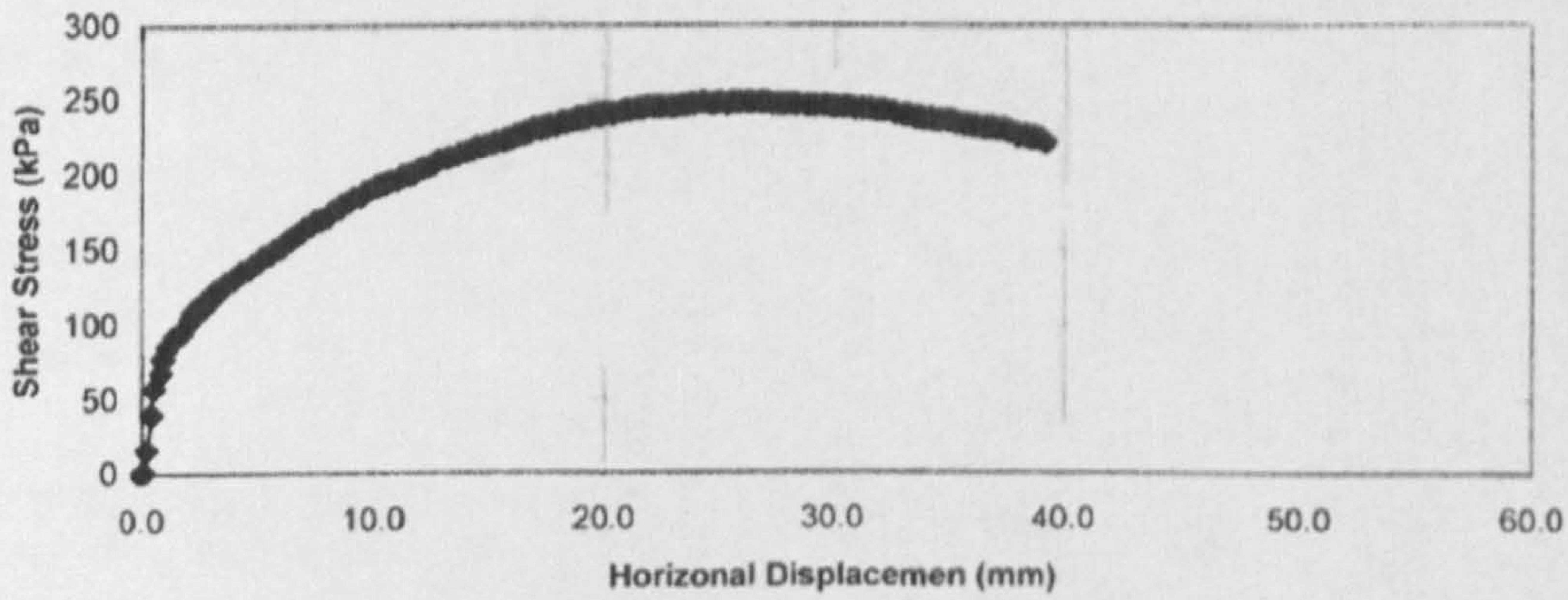

Figure A.10 Shear stress versus horizontal displacement in specimen No.3 of T-2

A plot of peak shear stress and normal stress from three specimens can gives the effective cohesion c' and effective angle of shearing resistance, ϕ'. For T-2, the results were found to be {c'=44kPa, ϕ'=46^o} as seen in Figure A.11.

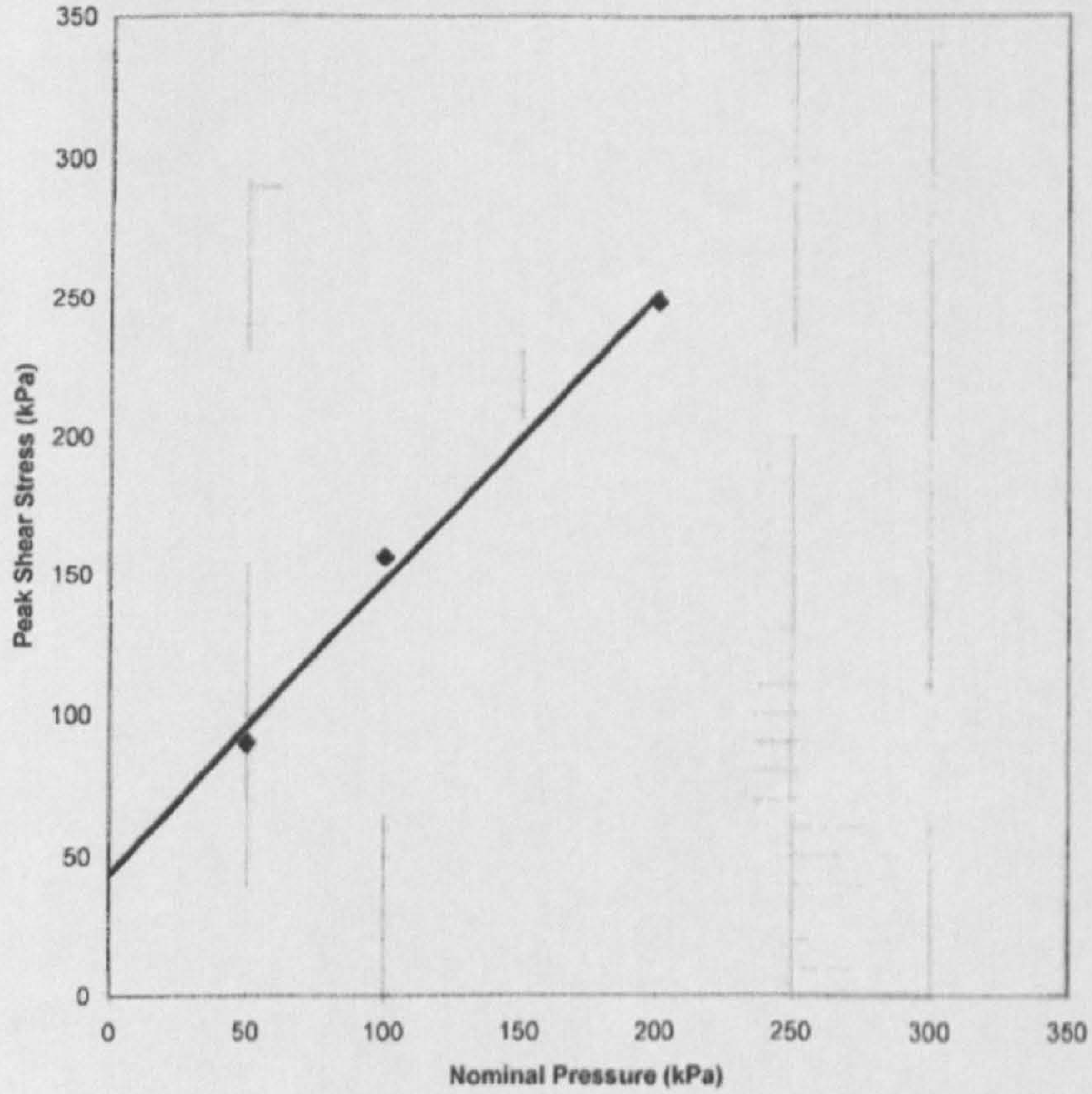

Figure A.11 Shear stress versus normal stress in T-2

A.3 Test results of T-3

In specimen No.1 of T-3 the normal pressure was 50kPa. The corresponding maximum shear stress, horizontal displacement and height change were measured to be 63kPa, 44.5mm and 1.6mm, respectively (Figures A.12 and A.13).

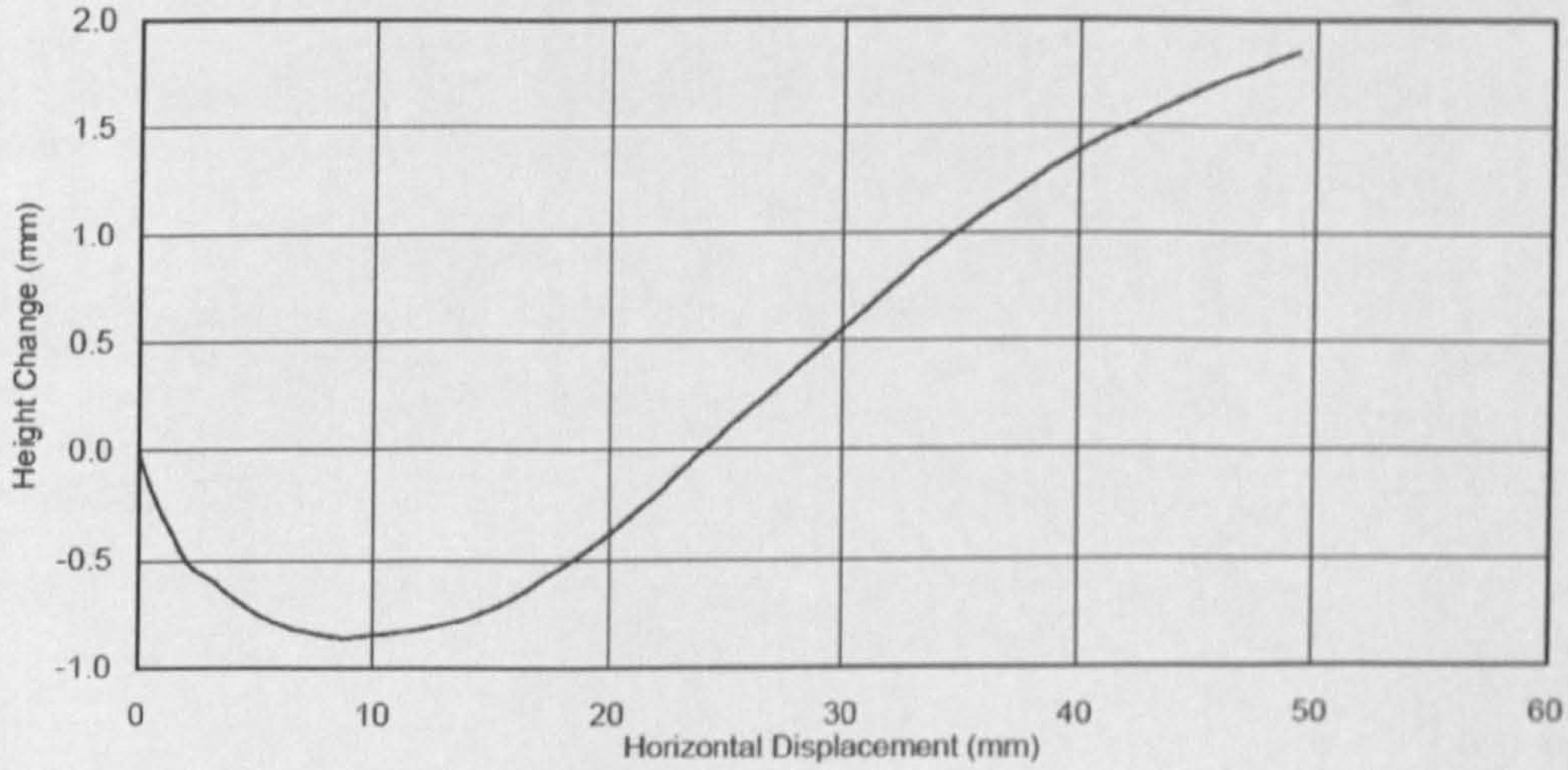

Figure A.12 Height change versus horizontal displacement in specimen No.1of T-3

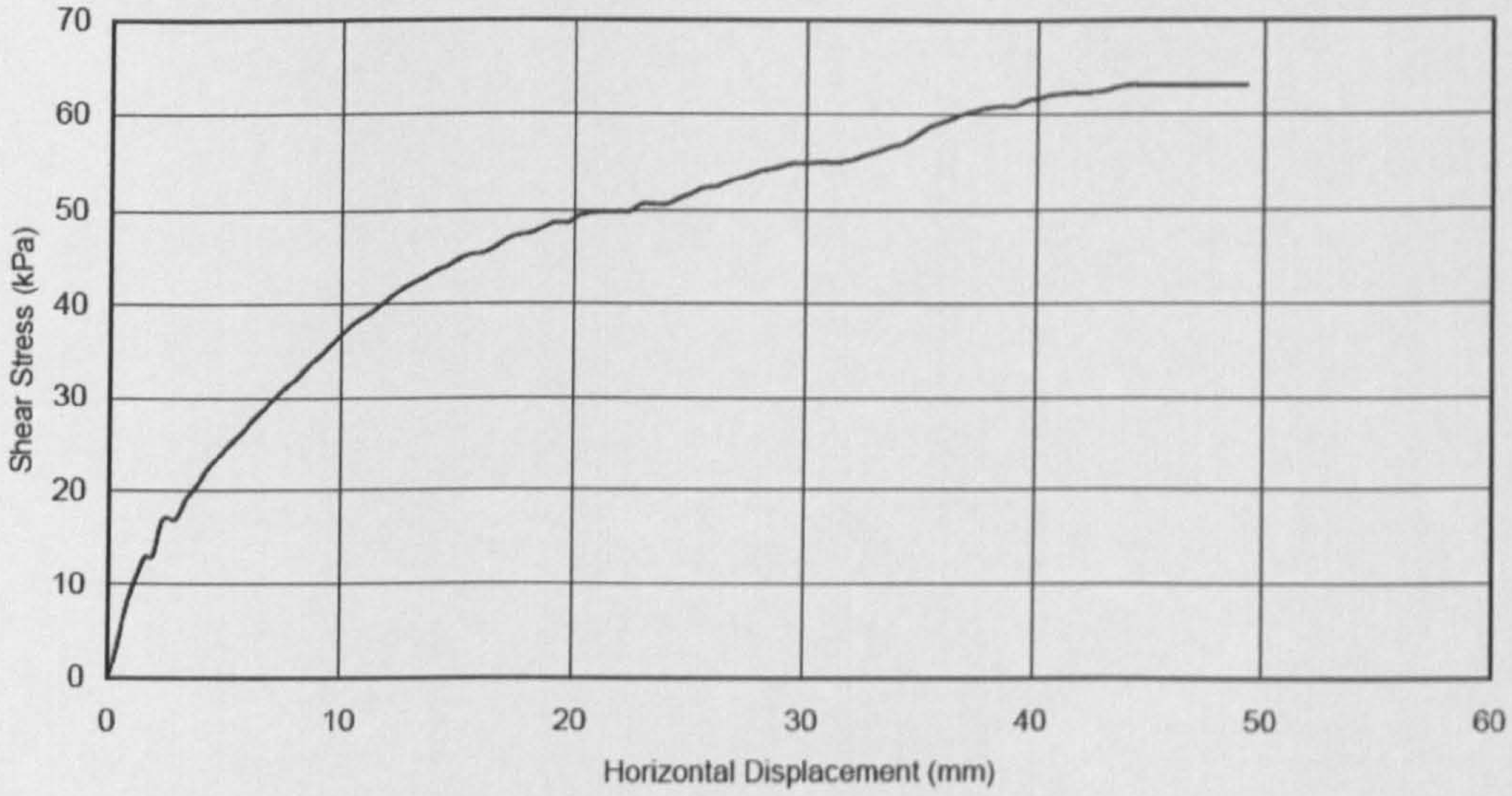

Figure A.13 Shear stress versus horizontal displacement in specimen No.1 of T-3

In specimen No.2 of T-3 the normal pressure was 100kPa. The corresponding maximum shear stress, horizontal displacement and height change were measured to be 104kPa, 44.4mm and -2.0mm, respectively (Figures A.14 and A.15).

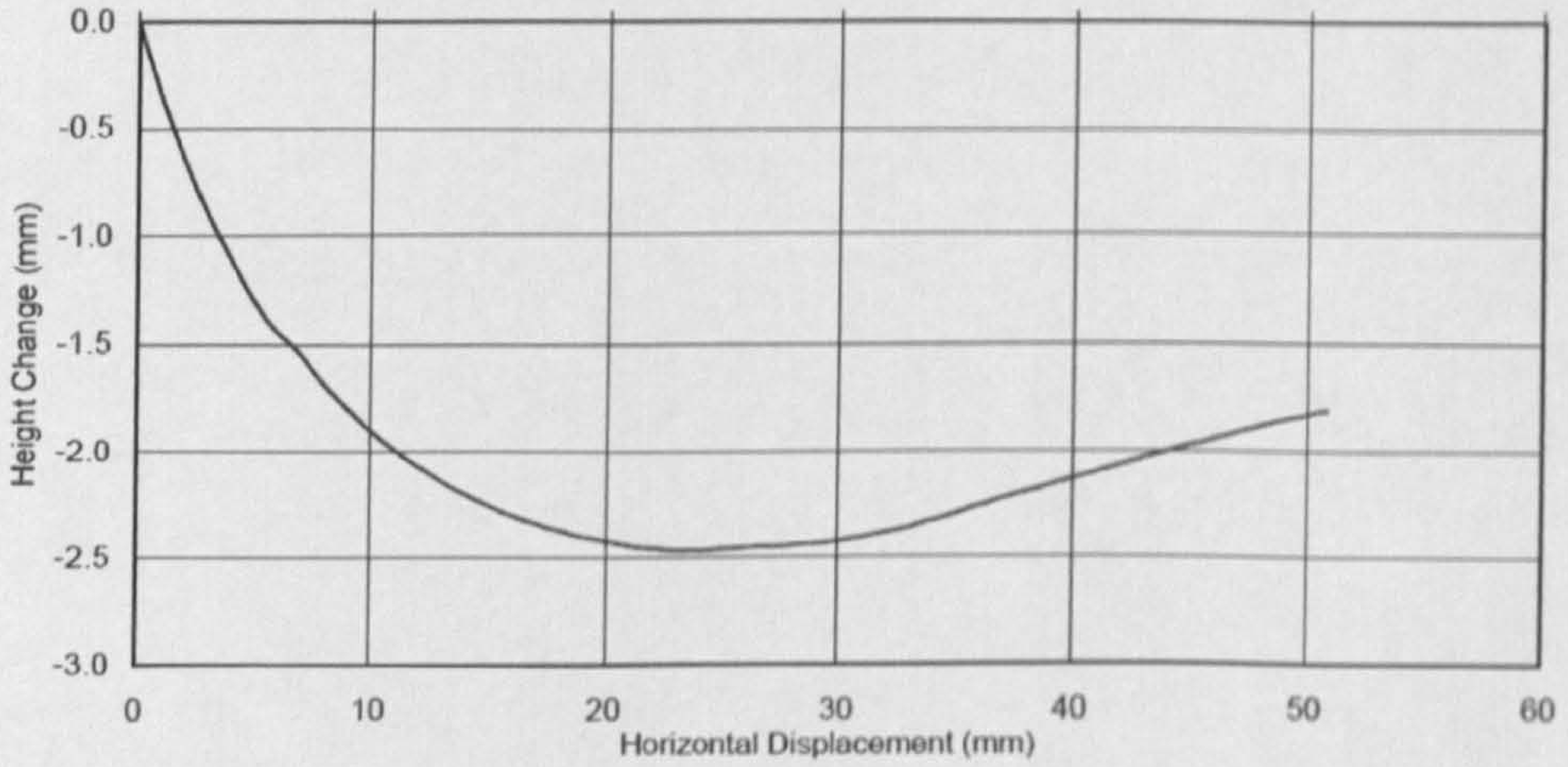

Figure A.14 Height change versus horizontal displacement in specimen No.2 of T-3

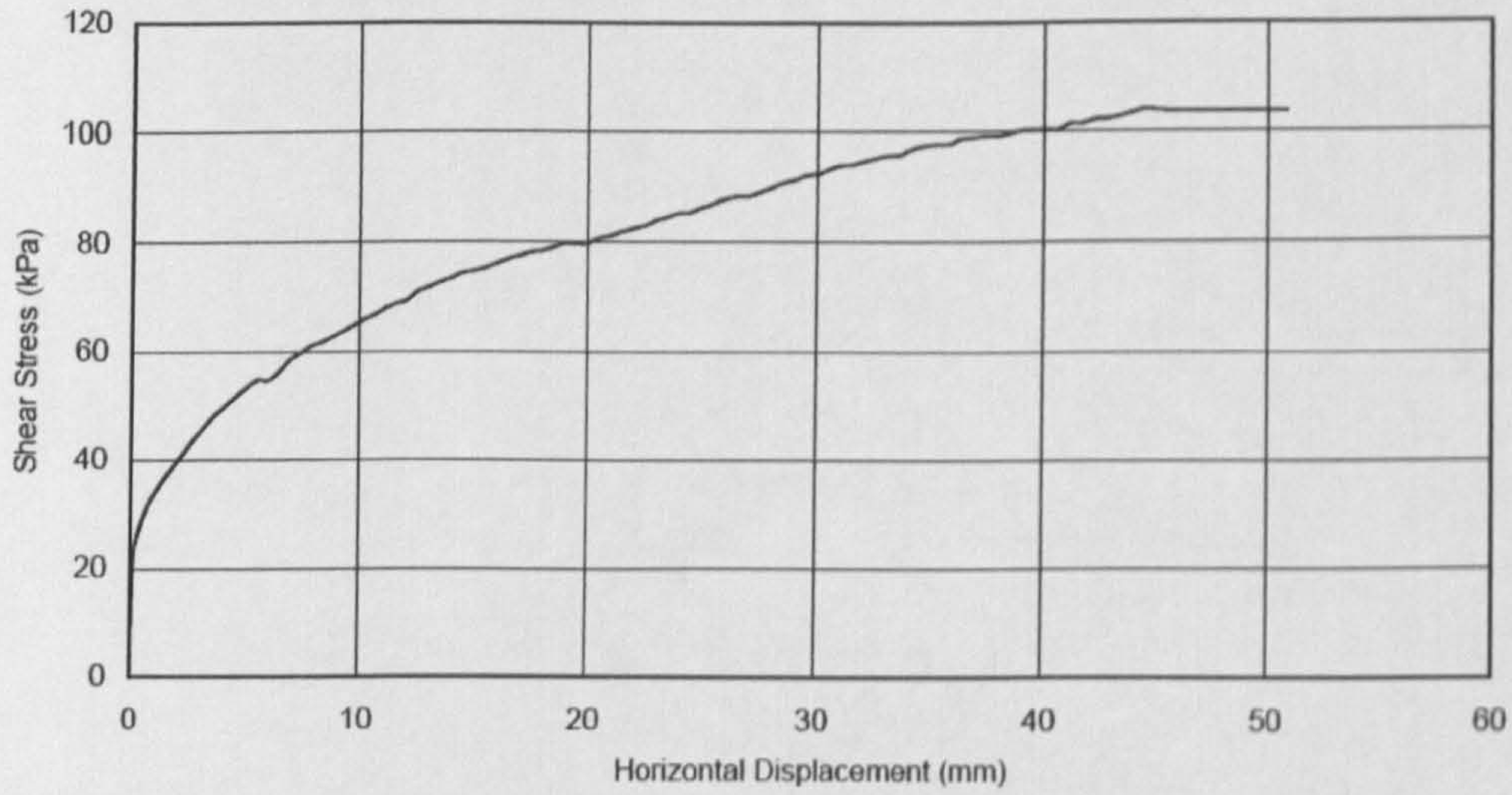

Figure A.15 Shear stress versus horizontal displacement in specimen No.2 of T-3

In specimen No.3 of T-3 the normal pressure was 200kPa. The corresponding maximum shear stress, horizontal displacement and height change were measured to be 191kPa, 45.0mm and -2.7mm, respectively (Figures A.16 and A.17).

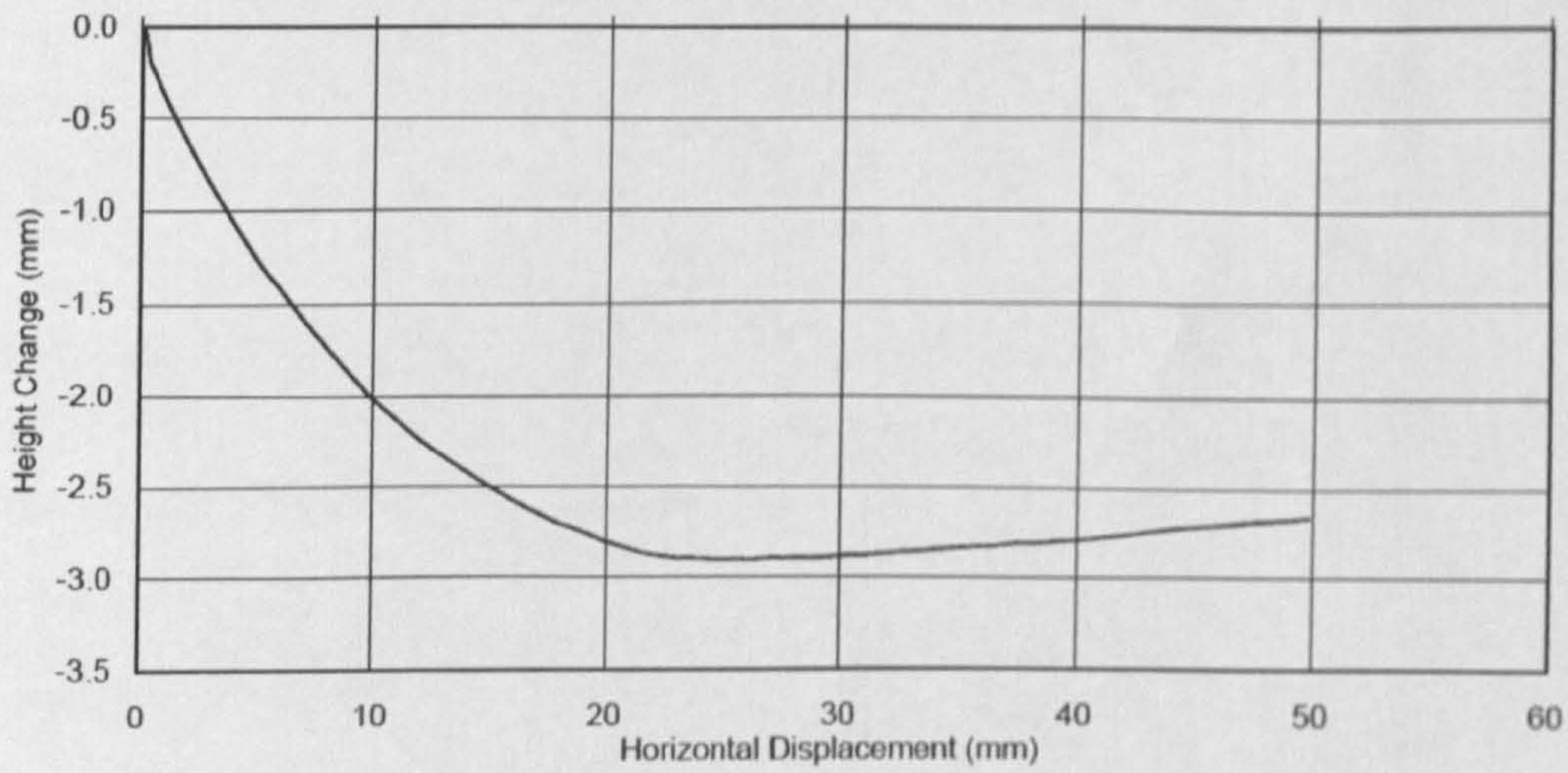

Figure A.16 Height change versus horizontal displacement in specimen No.3of T-3

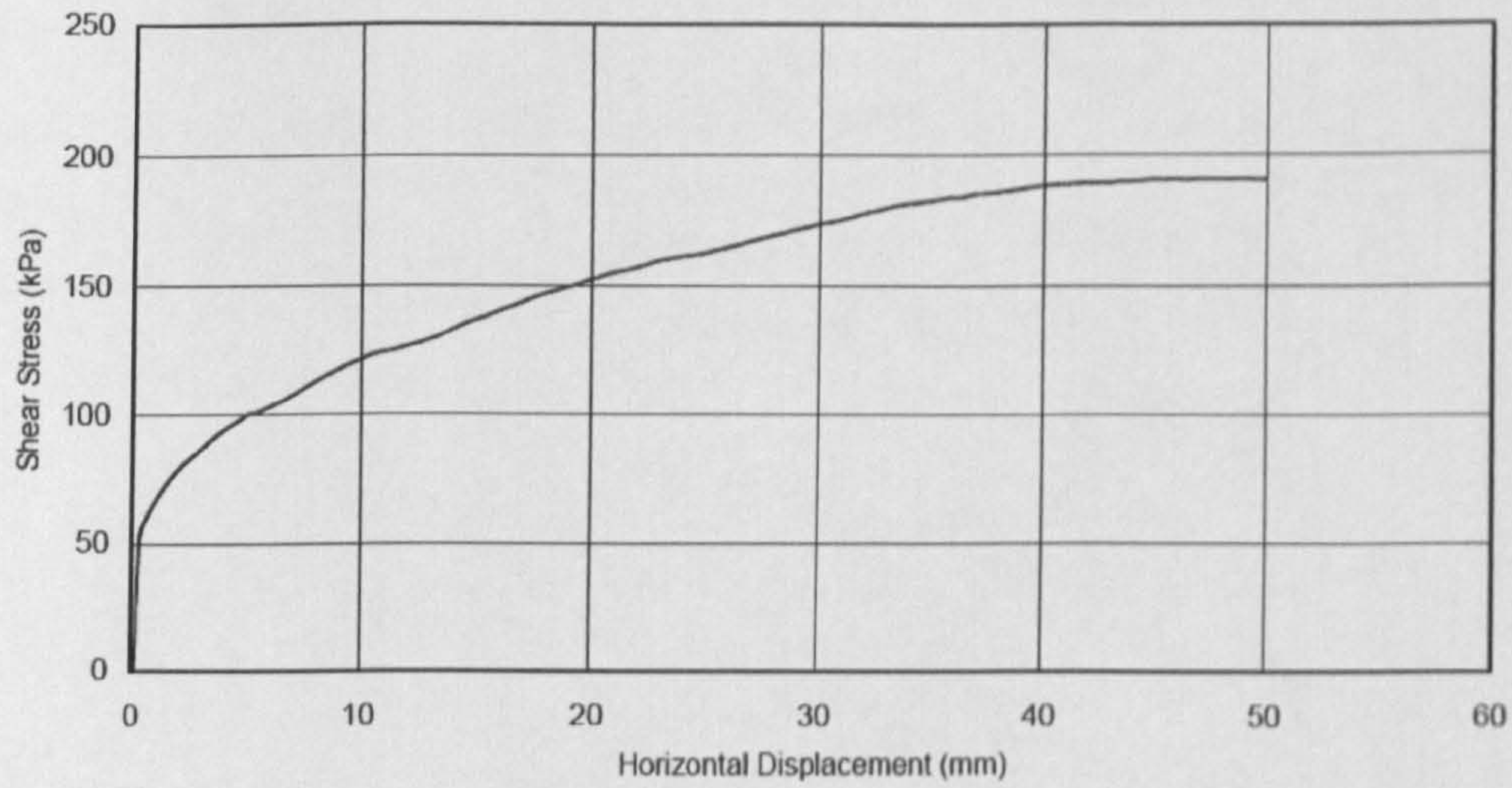

Figure A.17 Shear stress versus horizontal displacement in specimen No.3 of T-3

A plot of peak shear stress versus normal stress from three specimens gives the effective cohesion c' and effective angle of shearing resistance, ϕ'. For T-3, the results were found to be {c'=20kPa, ϕ'=40.5^o} as seen in Figure A.18.

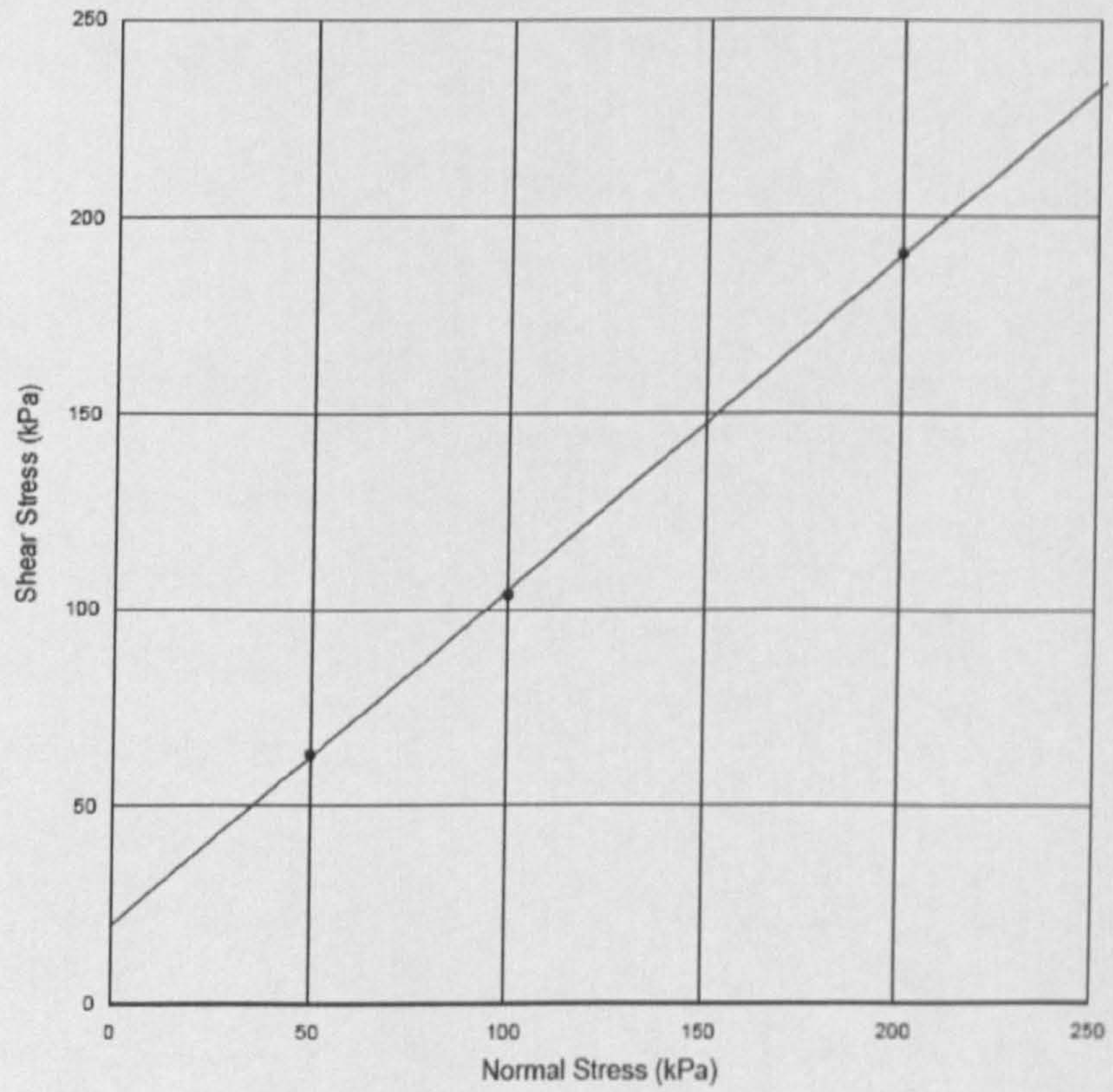

Figure A.18 Shear stress versus normal stress in T-3

APPENDIX B

Appendix B indicates the details of modeling of a typical pavement including a subbase layer made of T-1, T-2 and T-3 mixes with KENLAYERTM . The pavement has been modeled in both unsoaked and soaked conditions.

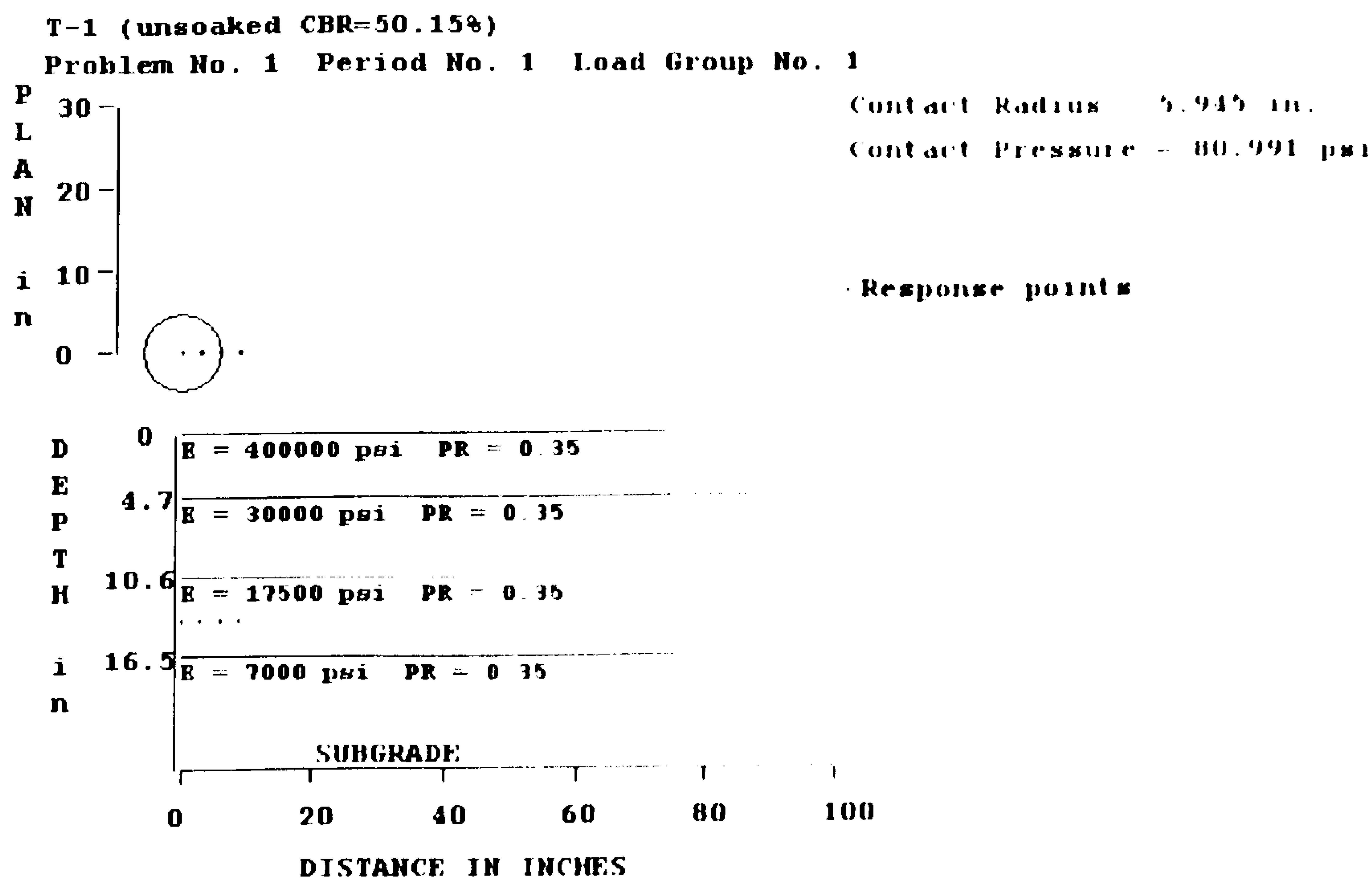

Figure B.1 Details of a typical pavement with subbase layer of T-1(unsoaked condition)

```
INPUT FILE NAME  -h:\kenlayer\T-1 (CBR=50.15% unbounded 2- 4point).DAT

NUMBER OF PROBLEMS TO BE SOLVED =  1

TITLE -T-1 (unsoaked CBR=50.15%)

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA = 0, SO DAMAGE ANALYSIS WILL NOT BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) =  1
NUMBER OF LOAD GROUPS (NLG) =  1
TOLERANCE FOR INTEGRATION (DEL) -- =  0.001
NUMBER OF LAYERS (NL)------------- =  4
NUMBER OF Z COORDINATES (NZ)------ =  1
LIMIT OF INTEGRATION CYCLES (ICL)- =  80
COMPUTING CODE (NSTD)------------- =  9
SYSTEM OF UNITS (NUNIT)------------=  0

Length and displacement in in., stress and modulus in psi
unit weight in pcf, and temperature in F

THICKNESSES OF LAYERS (TH) ARE : 4.7  5.9  5.9
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35  0.35  0.35  0.35
VERTICAL COORDINATES OF POINTS (ZC) ARE:  13.55
CONDITIONS OF INTERFACES (INT) ARE :  1  0  0

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE :    1  4.000E+05   2  3.000E+04
  3  1.750E+04   4  7.000E+03

LOAD GROUP NO. 1 HAS 1 CONTACT AREA
CONTACT RADIUS (CR)--------------- =  5.945
CONTACT PRESSURE (CP)------------- =  80.991
RADIAL COORDINATES OF  4  POINT(S) (RC) ARE :  0  2.9725  5.945  8.9175

PERIOD NO.  1   LOAD GROUP NO.  1
```

RADIAL COORDINATE	VERTICAL COORDINATE	VERTICAL DISPLACEMENT	VERTICAL STRESS (STRAIN)	RADIAL STRESS (STRAIN)	TANGENTIAL STRESS (STRAIN)	SHEAR STRESS (STRAIN)
0.00000	13.55000	0.03175	7.703	1.208	1.208	0.000
(STRAIN)			3.919E-04	-1.092E-04	-1.092E-04	.000E+00
2.97250	13.55000	0.03130	7.359	1.201	1.132	0.553
(STRAIN)			3.738E-04	-1.012E-04	-1.065E-04	.854E-04
5.94500	13.55000	0.03004	6.469	1.172	0.939	0.887
(STRAIN)			3.275E-04	-8.121E-05	-9.918E-05	.137E-03
8.91750	13.55000	0.02821	5.363	1.107	0.702	0.933
(STRAIN)			2.703E-04	-5.805E-05	-8.930E-05	.144E-03

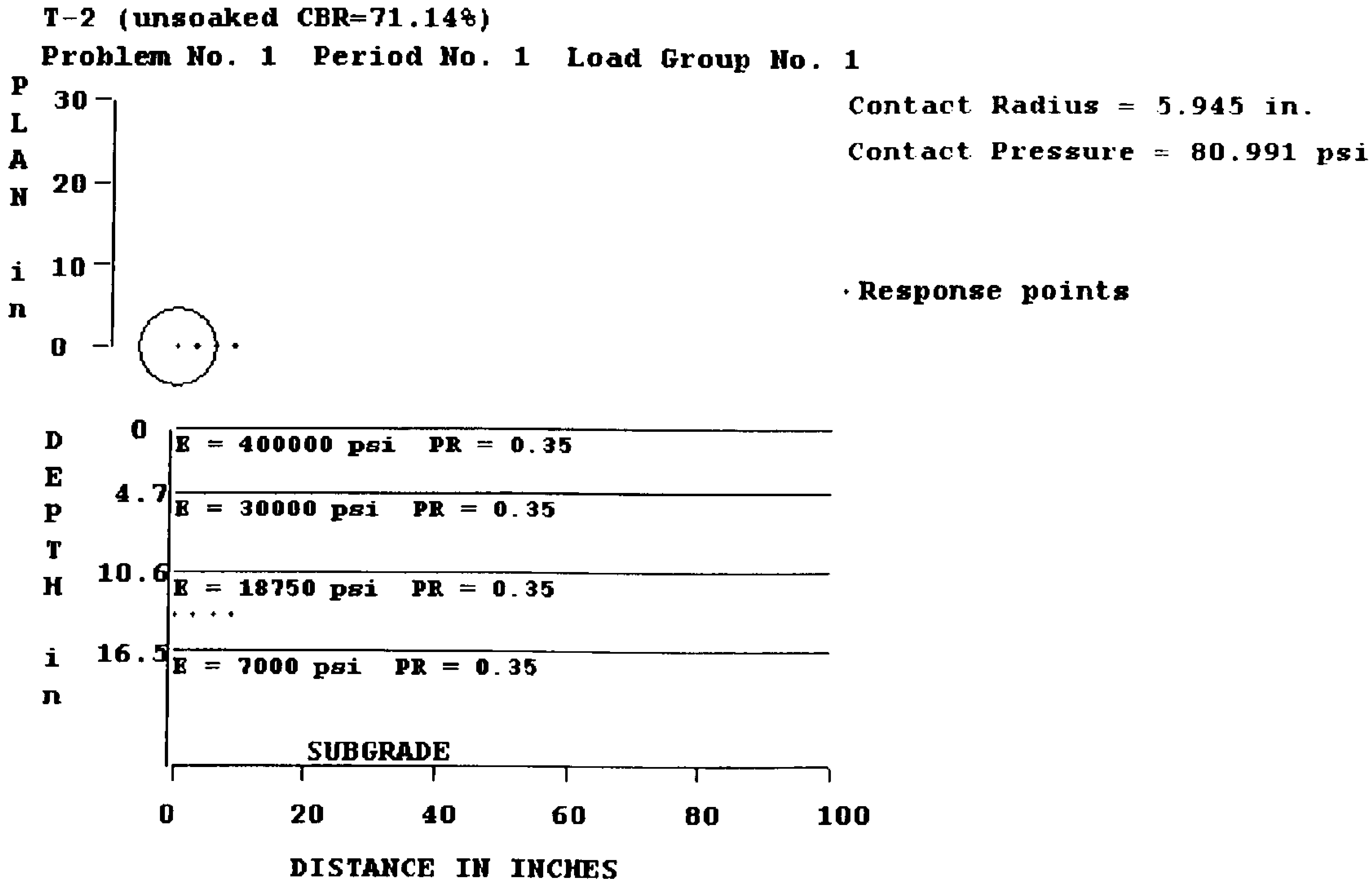

Figure B.2 Details of a typical pavement with subbase layer of T-2(unsoaked condition)

```
INPUT FILE NAME  -h:\kenlayer\T-2 (un soaked CBR=71.14% unbounded 2- 4poits) .DAT

NUMBER OF PROBLEMS TO BE SOLVED =  1

TITLE -T-2 (unsoaked CBR=71.14%)

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA = 0, SO DAMAGE ANALYSIS WILL NOT BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) =  1
NUMBER OF LOAD GROUPS (NLG) =  1
TOLERANCE FOR INTEGRATION (DEL) -- =  0.001
NUMBER OF LAYERS (NL)------------- =  4
NUMBER OF Z COORDINATES (NZ)------ =  1
LIMIT OF INTEGRATION CYCLES (ICL)- =  80
COMPUTING CODE (NSTD)------------- =  9
SYSTEM OF UNITS (NUNIT)------------=  0

Length and displacement in in., stress and modulus in psi
unit weight in pcf, and temperature in F

THICKNESSES OF LAYERS (TH) ARE : 4.7  5.9  5.9
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35  0.35  0.35  0.35
VERTICAL COORDINATES OF POINTS (ZC) ARE:  13.55
CONDITIONS OF INTERFACES (INT) ARE :  1  0  0

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE :    1  4.000E+05   2  3.000E+04
  3  1.875E+04   4  7.000E+03

LOAD GROUP NO. 1 HAS 1 CONTACT AREA
CONTACT RADIUS (CR)--------------- =  5.945
CONTACT PRESSURE (CP)------------- =  80.991
RADIAL COORDINATES OF  4  POINT(S) (RC) ARE :  0  2.9725  5.945  8.9175

PERIOD NO.  1   LOAD GROUP NO.  1
```

RADIAL COORDINATE	VERTICAL COORDINATE	VERTICAL DISPLACEMENT	VERTICAL STRESS (STRAIN)	RADIAL STRESS (STRAIN)	TANGENTIAL STRESS (STRAIN)	SHEAR STRESS (STRAIN)
0.00000	13.55000	0.03172	7.774	1.217	1.217	0.000
(STRAIN)			3.692E-04	-1.029E-04	-1.029E-04	.000E+00
2.97250	13.55000	0.03127	7.422	1.210	1.140	0.591
(STRAIN)			3.520E-04	-9.529E-05	-1.003E-04	.851E-04
5.94500	13.55000	0.03001	6.513	1.181	0.942	0.948
(STRAIN)			3.077E-04	-7.620E-05	-9.335E-05	.136E-03
8.91750	13.55000	0.02819	5.386	1.115	0.702	0.998
(STRAIN)			2.533E-04	-5.417E-05	-8.393E-05	.144E-03

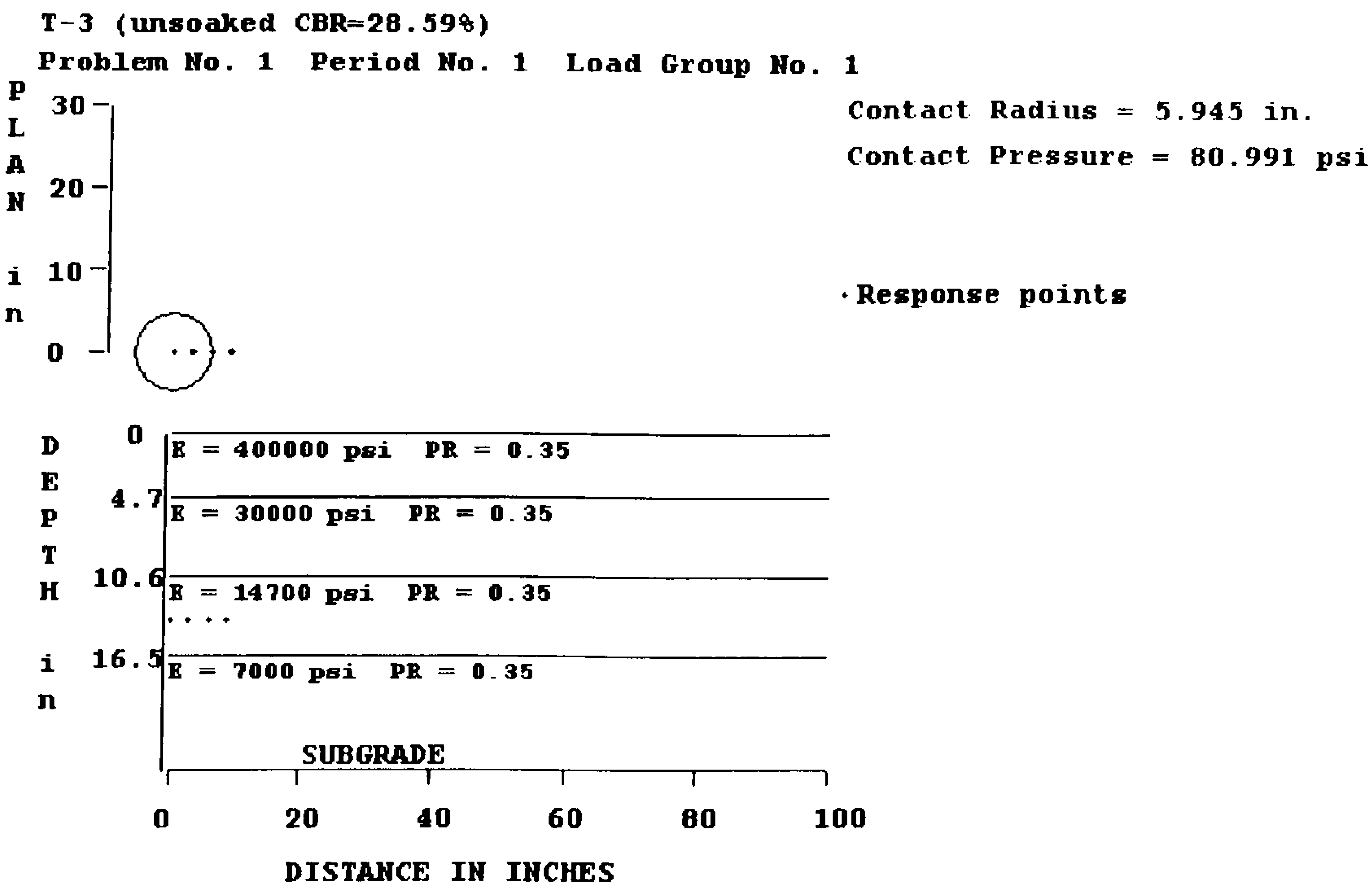

Figure B.3 Details of a typical pavement with subbase layer of T-3(unsoaked condition)

```
INPUT FILE NAME  -h:\kenlayer\T-3 (unsoaked CBR=28.59% unbounded 2- 4poits).DAT

NUMBER OF PROBLEMS TO BE SOLVED =  1

TITLE -T-3 (unsoaked CBR=28.59%)

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA = 0, SO DAMAGE ANALYSIS WILL NOT BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) =  1
NUMBER OF LOAD GROUPS (NLG) =  1
TOLERANCE FOR INTEGRATION (DEL) -- =  0.001
NUMBER OF LAYERS (NL)------------- =  4
NUMBER OF Z COORDINATES (NZ)------ =  1
LIMIT OF INTEGRATION CYCLES (ICL)- =  80
COMPUTING CODE (NSTD)------------- =  9
SYSTEM OF UNITS (NUNIT)------------=  0

Length and displacement in in., stress and modulus in psi
unit weight in pcf, and temperature in F

THICKNESSES OF LAYERS (TH) ARE : 4.7  5.9  5.9
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35  0.35  0.35  0.35
VERTICAL COORDINATES OF POINTS (ZC) ARE:  13.55
CONDITIONS OF INTERFACES (INT) ARE :  1  0  0

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE :    1  4.000E+05    2  3.000E+04
  3  1.470E+04    4  7.000E+03

LOAD GROUP NO. 1 HAS 1 CONTACT AREA
CONTACT RADIUS (CR)--------------- =  5.945
CONTACT PRESSURE (CP)------------- =  80.991
RADIAL COORDINATES OF  4  POINT(S) (RC) ARE :  0  2.9725  5.945  8.9175

PERIOD NO.  1   LOAD GROUP NO.  1
```

RADIAL COORDINATE	VERTICAL COORDINATE	VERTICAL DISPLACEMENT	VERTICAL STRESS (STRAIN)	RADIAL STRESS (STRAIN)	TANGENTIAL STRESS (STRAIN)	SHEAR STRESS (STRAIN)
0.00000	13.55000	0.03182	7.525	1.184	1.184	0.000
(STRAIN)			4.556E-04	-1.268E-04	-1.268E-04	.000E+00
2.97250	13.55000	0.03137	7.198	1.177	1.112	0.468
(STRAIN)			4.352E-04	-1.178E-04	-1.237E-04	.860E-04
5.94500	13.55000	0.03010	6.357	1.149	0.929	0.749
(STRAIN)			3.830E-04	-9.534E-05	-1.155E-04	.138E-03
8.91750	13.55000	0.02828	5.303	1.086	0.702	0.785
(STRAIN)			3.182E-04	-6.906E-05	-1.044E-04	.144E-03

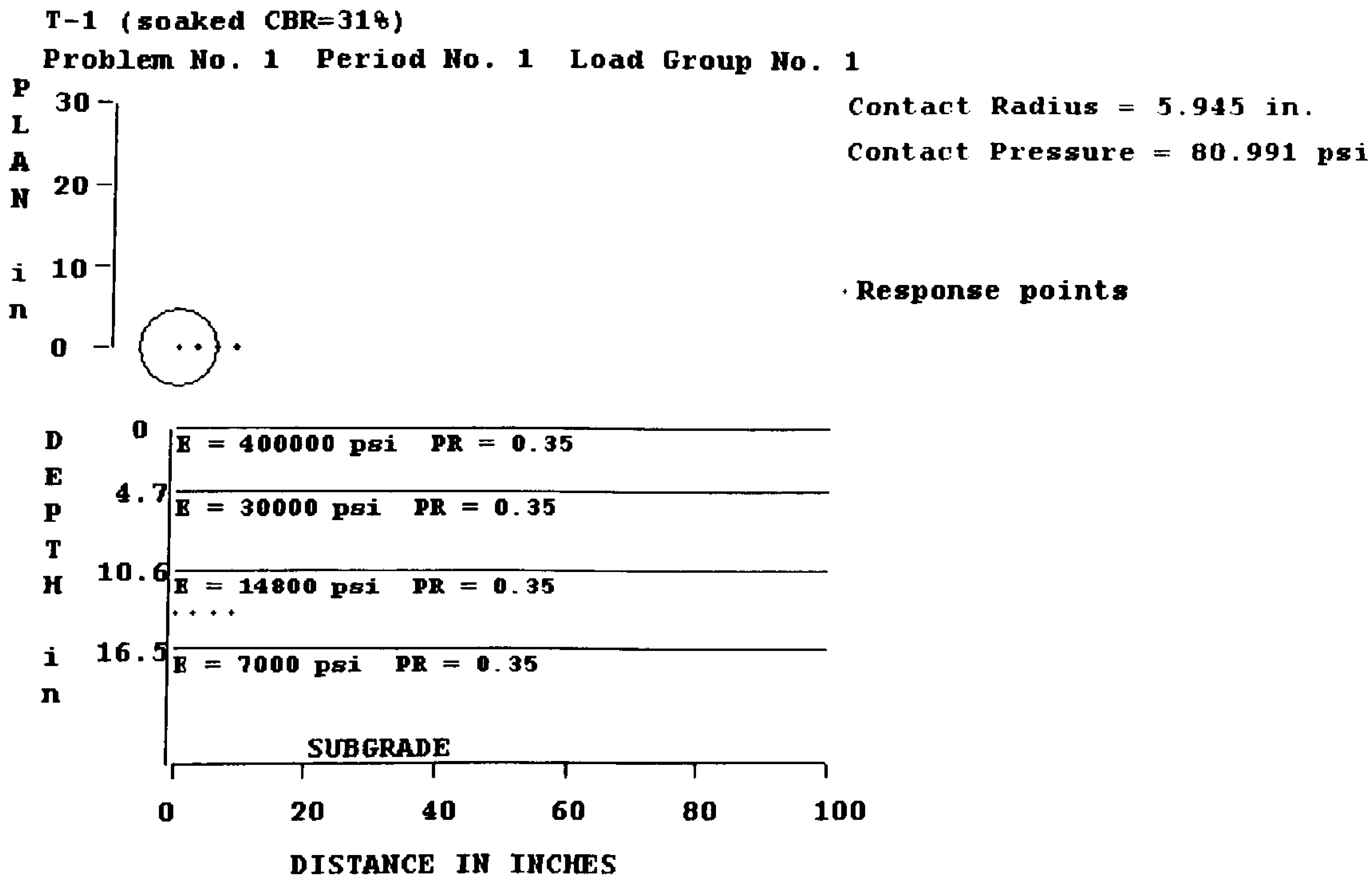

Figure B.4 Details of a typical pavement with subbase layer of T-1(soaked condition)

```
INPUT FILE NAME  -h:\kenlayer\T-1 (soaked CBR=31% unbounded 2- 4 point).DAT

NUMBER OF PROBLEMS TO BE SOLVED =  1

TITLE -T-1 (soaked CBR=31%)

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA = 0, SO DAMAGE ANALYSIS WILL NOT BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) =  1
NUMBER OF LOAD GROUPS (NLG) =  1
TOLERANCE FOR INTEGRATION (DEL) -- =  0.001
NUMBER OF LAYERS (NL)------------- =  4
NUMBER OF Z COORDINATES (NZ)------ =  1
LIMIT OF INTEGRATION CYCLES (ICL)- =  80
COMPUTING CODE (NSTD)------------- =  9
SYSTEM OF UNITS (NUNIT)------------=  0

Length and displacement in in., stress and modulus in psi
unit weight in pcf, and temperature in F|

THICKNESSES OF LAYERS (TH) ARE : 4.7  5.9  5.9
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35  0.35  0.35  0.35
VERTICAL COORDINATES OF POINTS (ZC) ARE:  13.55
CONDITIONS OF INTERFACES (INT) ARE :  1  0  0

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE :    1  4.000E+05   2  3.000E+04
  3  1.480E+04   4  7.000E+03

LOAD GROUP NO. 1 HAS 1 CONTACT AREA
CONTACT RADIUS (CR)--------------- =  5.945
CONTACT PRESSURE (CP)------------- =  80.991
RADIAL COORDINATES OF  4  POINT(S) (RC) ARE :  0  2.9725  5.945  8.9175

PERIOD NO.  1   LOAD GROUP NO.  1
```

RADIAL COORDINATE	VERTICAL COORDINATE	VERTICAL DISPLACEMENT	VERTICAL STRESS (STRAIN)	RADIAL STRESS (STRAIN)	TANGENTIAL STRESS (STRAIN)	SHEAR STRESS (STRAIN)
0.00000	13.55000	0.03182	7.533	1.185	1.185	0.000
(STRAIN)			4.529E-04	-1.261E-04	-1.261E-04	.000E+00
2.97250	13.55000	0.03136	7.205	1.178	1.113	0.471
(STRAIN)			4.326E-04	-1.171E-04	-1.230E-04	.860E-04
5.94500	13.55000	0.03010	6.362	1.149	0.929	0.754
(STRAIN)			3.807E-04	-9.475E-05	-1.148E-04	.138E-03
8.91750	13.55000	0.02827	5.305	1.087	0.702	0.791
(STRAIN)			3.162E-04	-6.859E-05	-1.038E-04	.144E-03

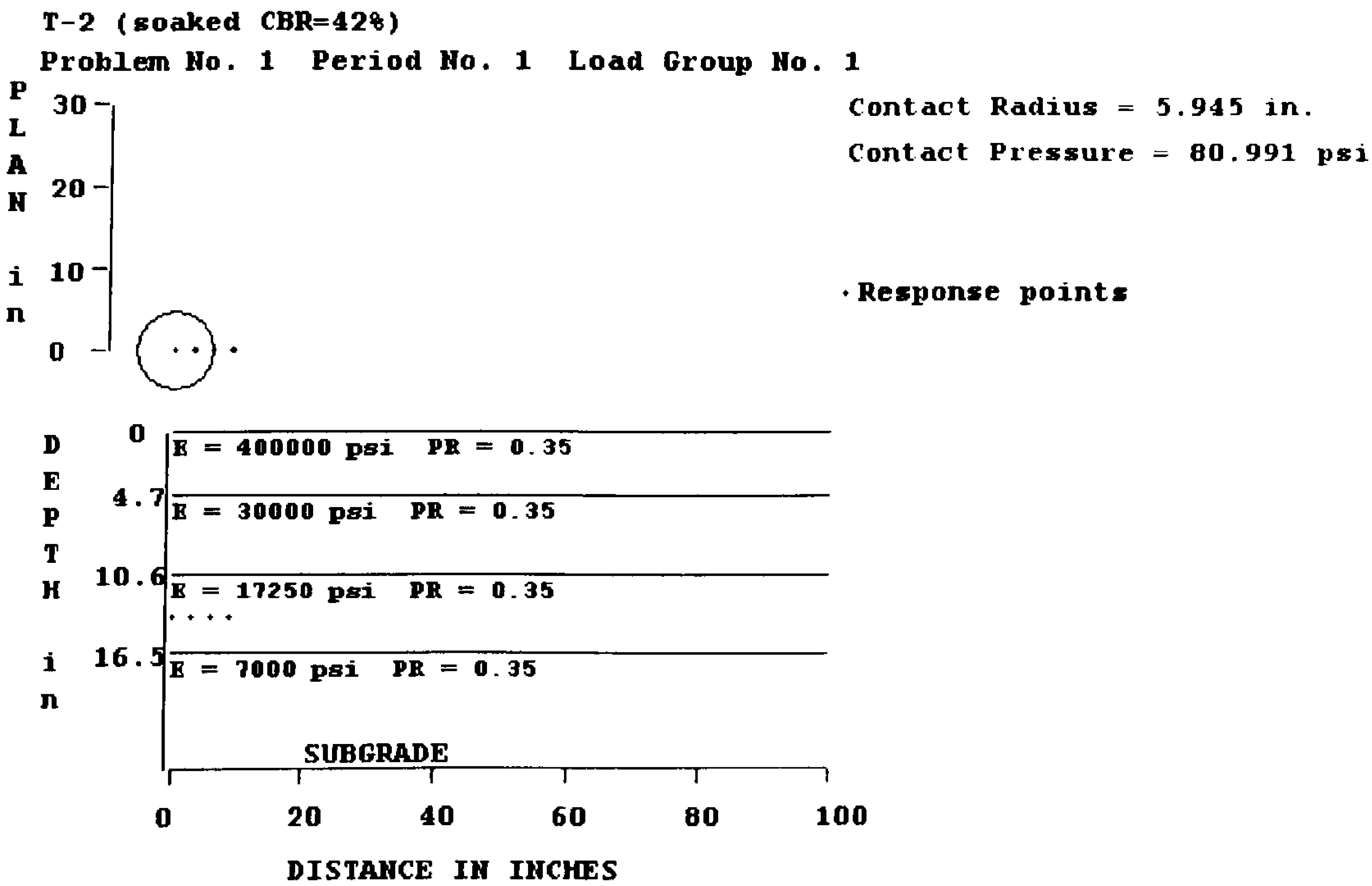

Figure B.5 Details of a typical pavement with subbase layer of T-2(soaked condition)

```
INPUT FILE NAME  -h:\kenlayer\T-2 (soaked CBR=42% unbounded 2- 4poits) .DAT

NUMBER OF PROBLEMS TO BE SOLVED =  1

TITLE -T-2 (soaked CBR=42%)

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA = 0, SO DAMAGE ANALYSIS WILL NOT BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) =  1
NUMBER OF LOAD GROUPS (NLG) =  1
TOLERANCE FOR INTEGRATION (DEL) -- =  0.001
NUMBER OF LAYERS (NL)------------- =  4
NUMBER OF Z COORDINATES (NZ)------ =  1
LIMIT OF INTEGRATION CYCLES (ICL)- =  80
COMPUTING CODE (NSTD)------------- =  9
SYSTEM OF UNITS (NUNIT)------------=  0

Length and displacement in in., stress and modulus in psi
unit weight in pcf, and temperature in F

THICKNESSES OF LAYERS (TH) ARE : 4.7  5.9  5.9
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35  0.35  0.35  0.35
VERTICAL COORDINATES OF POINTS (ZC) ARE:  13.55
CONDITIONS OF INTERFACES (INT) ARE :  1  0  0

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE :    1  4.000E+05    2  3.000E+04
   3  1.725E+04    4  7.000E+03

LOAD GROUP NO. 1 HAS 1 CONTACT AREA
CONTACT RADIUS (CR)--------------- =  5.945
CONTACT PRESSURE (CP)------------- =  80.991
RADIAL COORDINATES OF  4  POINT(S) (RC) ARE :  0  2.9725  5.945  8.9175

PERIOD NO.  1   LOAD GROUP NO.  1

  RADIAL      VERTICAL      VERTICAL     VERTICAL     RADIAL    TANGENTIAL    SHEAR
COORDINATE   COORDINATE   DISPLACEMENT   STRESS       STRESS      STRESS      STRESS
                                        (STRAIN)     (STRAIN)    (STRAIN)    (STRAIN)
  0.00000     13.55000      0.03176       7.688       1.206       1.206       0.000
 (STRAIN)                              3.968E-04  -1.106E-04  -1.106E-04   .000E+00
  2.97250     13.55000      0.03130       7.345       1.199       1.130       0.546
 (STRAIN)                              3.786E-04  -1.025E-04  -1.078E-04   .855E-04
  5.94500     13.55000      0.03004       6.460       1.170       0.938       0.875
 (STRAIN)                              3.317E-04  -8.230E-05  -1.004E-04   .137E-03
  8.91750     13.55000      0.02822       5.358       1.105       0.702       0.920
 (STRAIN)                              2.740E-04  -5.889E-05  -9.046E-05   .144E-03
```

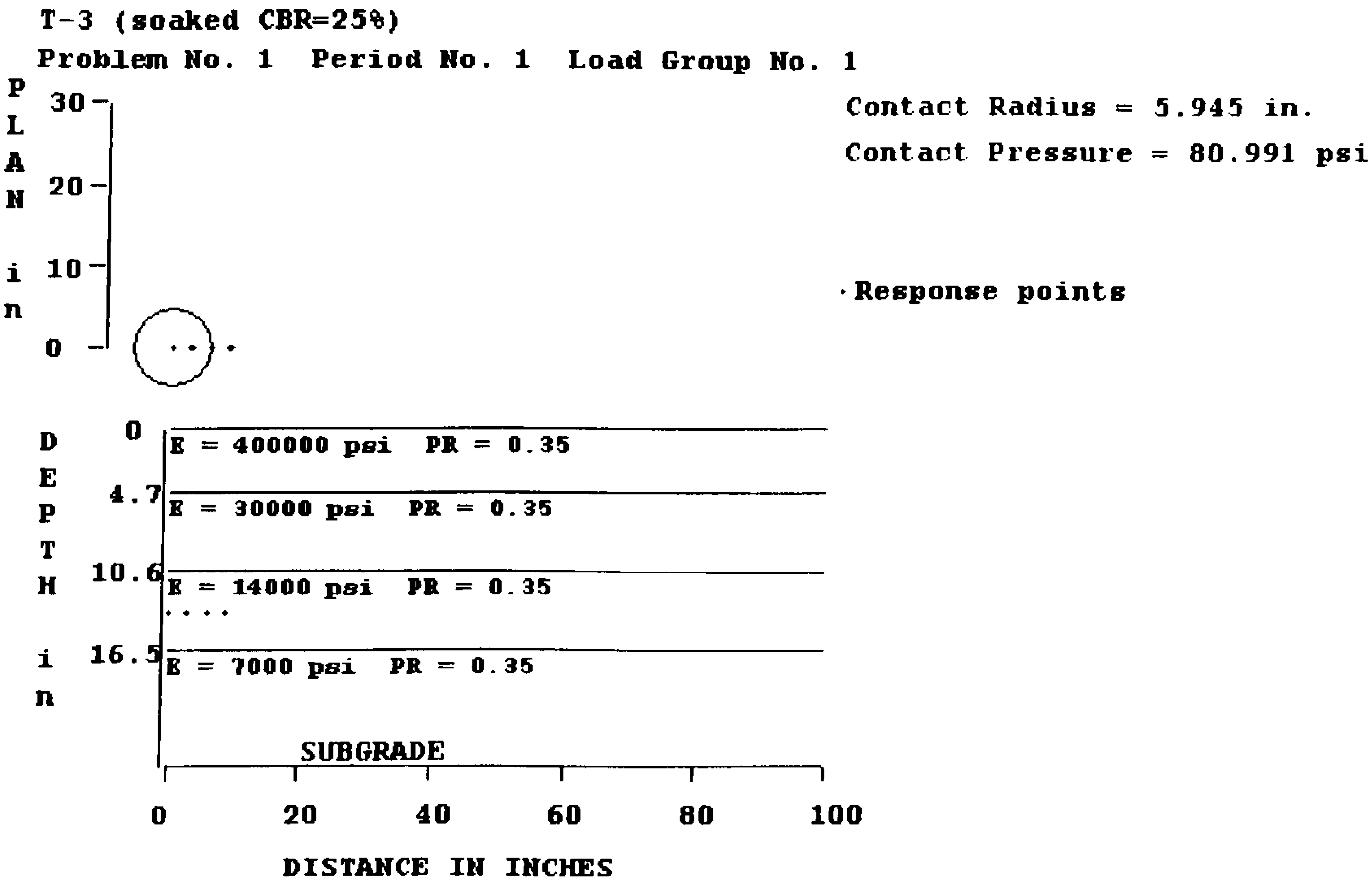

Figure B.6 Details of a typical pavement with subbase layer of T-3(soaked condition)

```
INPUT FILE NAME  -h:\kenlayer\T-3 (soaked CBR=25% unbounded 2- 4poits).DAT

NUMBER OF PROBLEMS TO BE SOLVED =  1

TITLE -T-3 (soaked CBR=25%)

MATL = 1 FOR LINEAR ELASTIC LAYERED SYSTEM
NDAMA = 0, SO DAMAGE ANALYSIS WILL NOT BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) =  1
NUMBER OF LOAD GROUPS (NLG) =  1
TOLERANCE FOR INTEGRATION (DEL) -- =  0.001
NUMBER OF LAYERS (NL)------------- =  4
NUMBER OF Z COORDINATES (NZ)------ =  1
LIMIT OF INTEGRATION CYCLES (ICL)- =  80
COMPUTING CODE (NSTD)------------- =  9
SYSTEM OF UNITS (NUNIT)------------=  0

Length and displacement in in., stress and modulus in psi|
unit weight in pcf, and temperature in F

THICKNESSES OF LAYERS (TH) ARE : 4.7  5.9  5.9
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35  0.35  0.35  0.35
VERTICAL COORDINATES OF POINTS (ZC) ARE:  13.55
CONDITIONS OF INTERFACES (INT) ARE :  1  0  0

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE :    1  4.000E+05   2  3.000E+04
   3  1.400E+04   4  7.000E+03

LOAD GROUP NO. 1 HAS 1 CONTACT AREA
CONTACT RADIUS (CR)--------------- =  5.945
CONTACT PRESSURE (CP)------------- =  80.991
RADIAL COORDINATES OF  4  POINT(S) (RC) ARE :  0  2.9725  5.945  8.9175

PERIOD NO.  1   LOAD GROUP NO.  1
```

RADIAL COORDINATE	VERTICAL COORDINATE	VERTICAL DISPLACEMENT	VERTICAL STRESS (STRAIN)	RADIAL STRESS (STRAIN)	TANGENTIAL STRESS (STRAIN)	SHEAR STRESS (STRAIN)
0.00000	13.55000	0.03184	7.475	1.177	1.177	0.000
(STRAIN)			4.751E-04	-1.322E-04	-1.322E-04	.000E+00
2.97250	13.55000	0.03138	7.153	1.170	1.107	0.447
(STRAIN)			4.540E-04	-1.229E-04	-1.290E-04	.861E-04
5.94500	13.55000	0.03012	6.325	1.142	0.926	0.714
(STRAIN)			4.001E-04	-9.970E-05	-1.206E-04	.138E-03
8.91750	13.55000	0.02829	5.285	1.081	0.701	0.748
(STRAIN)			3.330E-04	-7.247E-05	-1.090E-04	.144E-03

APPENDIX C

Further controlled or fieldwork would provide high quality results regarding performance-based tests covering a variety of RCA mixed with NA and RAP from different sources. The fieldwork should be carried out at appropriate trial area under controlled conditions where layers thickness, water content, density and stress state such as traffic loading are known. Sample of the materials tested should be collected for controlled testing with using the laboratory assessment procedures and the laboratory results compared with field data, to further establish its ability to predict the likely the subbase materials. It would also be suitable to perform the laboratory assessment test of samples before it is used on trail area and then measure its performance in the field. However, the subgrade would still need to be assessed in the usual way as a part of the design process as illustrated in Figure B-1. During research period a collaborative plan was conducted with TRL on performance-related tests on a trail area which could be recommended for the future works. Following stages supposed to be executed:

a-1) In the Pavement Test Facility at TRL, it was supposed to be selected two test sections each 1.8m wide by 10m long. The reason for doing this trial in the Facility was to provide a uniform subgrade on which to lay the granular material so the measurements on top of the sub-base were not biased by variable subgrade strength. Each strip would be divided in to two 5m lengths then the purposed mixes would be laid and compacted as shown at the aggregates as supplied moisture content (Figure 8.1).

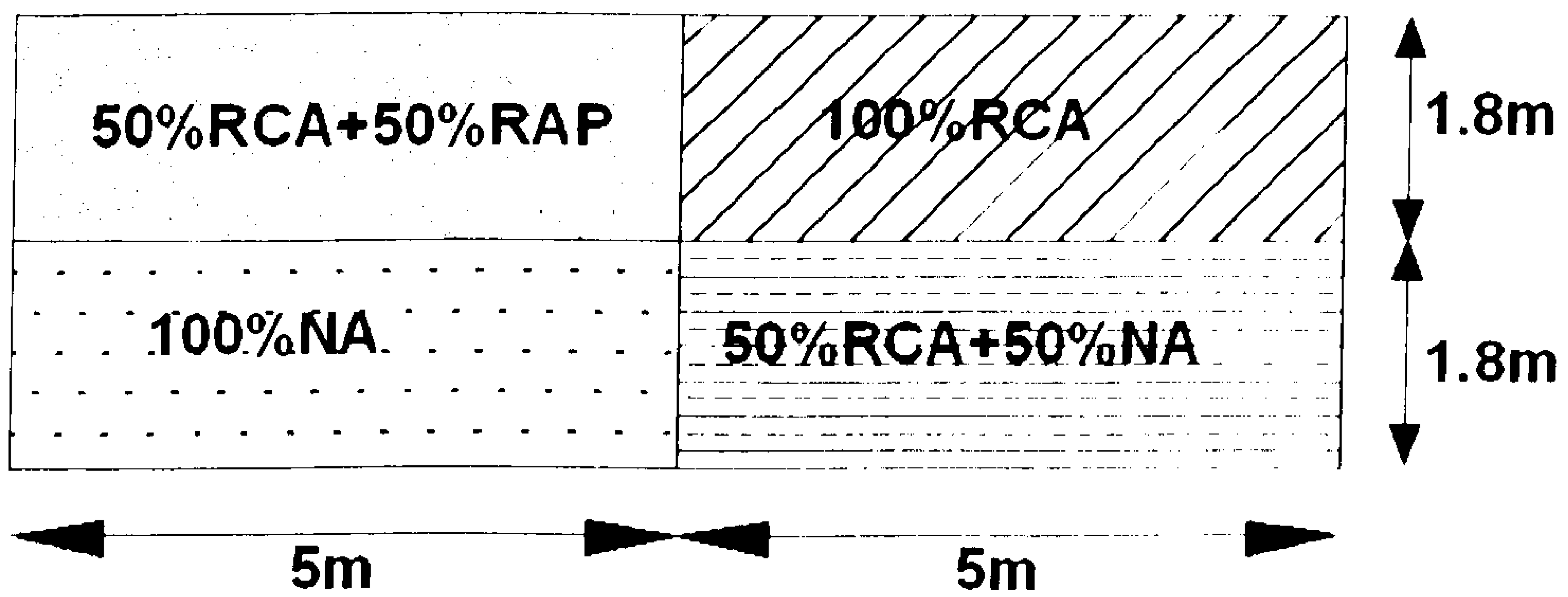

Figure C-1 Trial area

a-2) The materials will then be subject to FWD and PRIMA testing. MEXE cone penetrometer readings will be taken on the subgrade to establish its

CBR and optical levels will be taken during construction to establish layer thickness for the different sub-base materials. It would be assumed laying a nominal 300mm thick layer. The test could be performed for different thickness of subbase.

a-3) Once the testing has been completed the material will be excavated then lay another 4 sections using the material at its optimum moisture content. The same measurements as before i.e. MEXE, optical levels, FWD and PRIMA will then be taken. On completion the materials will be excavated and disposed.

a-4) In both cases in-situ moisture content samples and gradings for the different materials will be taken.

a-5) It was anticipated the entire process to take 10 working days, this consists of 1 day preparation, 1 day laying the first 4 materials, 2 days testing, 1 day clearing then 5 days to repeat this process.