PROJECT REPORT

Guidelines on the selection and use of road construction materials

by Transport Research Laboratory

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APPROVALS

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CONTENTS

1 INTRODUCTION

1.1 General Background

The frequently distinct engineering behaviour of naturally occurring construction materials within sub-tropical and tropical regions, as compared with those in temperate zones, has been identified as a key factor in determining the long-term engineering success or failure of road projects in developing countries (Millard ,1990).

The engineering behaviour of near surface sub-tropical and tropical soils and rocks is a function of the impact of their interaction with the road environment and the weathering processes. Both the tropical weathering process and key aspects of the road environment (eg climate) are radically different from those in temperate regions, where the great majority of the research and development of materials standards, specifications and construction procedures have originated. In the tropics and sub-tropics roads will tend to have non-standard responses to the impacts of environment and traffic that will not be picked up unless the approach to investigation and assessment of construction materials is specifically tailored to that environment. There is also a greater need to view the application of specifications and construction practices in terms of a whole road unit, including earthworks and drainage, rather than in terms of individual pavement layers.

It follows that there has been a need to derive and implement design and construction procedures specifically for the tropical and sub-tropical regions. Overseas Road Note 31 (ORN 31), which is now widely used in developing countries, provides such guidance on the structural design of sealed roads within tropical regions and defines the construction material requirements in terms of standard properties. ORN 31 does not seek, however, to provide details relevant to the selection, testing and appropriate use of these materials. Neither does it present information on alternative or non-standard approaches to material assessment and the conflicts that may arise out of employing inappropriate test procedures.

The report comes at a time of change. Roads departments and related Ministries are, by necessity, becoming more commercial with the establishment of road agencies etc. Also there are moves to unify standards and procedures in Europe, which inevitably will impact on specifications and standards used in the developing world that are based on British or other existing European national standards.

The report also comes at time of heightened environmental awareness both in terms of protection against adverse impacts and in the more effective husbanding of non-renewable natural construction materials that are in some areas are becoming increasingly difficult to identify. It is now increasingly viewed as vitally important to use materials appropriate to their role in the road, that is, to ensure that they are neither sub-standard nor wastefully above the standards demanded by their engineering task..

The above comments serve to highlight the need for identification, selection, processing and construction procedures that can be allied to relevant materials standards and specifications to ensure that appropriate construction materials sources are utilised to maximum long-term advantage.

1.2 Purpose of the Document

This document comprises the technical report on research undertaken under a TRL Knowledge and Research (KaR) contract (R6989) with DFID to produce guidelines on the selection and use of construction materials in developing countries. The general purpose of this document is to provide guidance on the selection and use of road construction materials and in doing so make a contribution to reducing the costs of constructing, rehabilitating and maintaining road infrastructure, and vehicle operations

The report aims to provide guidelines on the above issues and, in doing so, can be viewed as the first stage in the provision of a companion document, or series of documents, to ORN 31. It also seeks to raise and promote discussion on key selection and utilisation issues within its overall objective. Research for this report has highlighted areas within ORN 31 in terms of methodology and terminology that could be incorporated within future revisions of that document. In addition there are a number of terms and concepts within ORN 31 that would benefit from further explanation and discussion to enable them to be more fully understood by the practising engineer.

This report therefore covers topics of wide interest and as such is aimed at a range of end-users, such as engineers, planners, materials and laboratory engineers and technical staff, academics and staff trainers, consultants and contractors.

1.3 Document Strategy

The research leading to the drafting of this report was based on a comprehensive review of ongoing or completed TRL research and of reported projects by other recognised research bodies and consultants in the general field of road construction materials in tropical and sub-tropical regions. It also incorporated a review of ORN 31.

Initial research indicated a number of key themes that formed the basis for developing the project and the framework around which the subsequent report could be structured. These targeted themes were:

- <u>Definition of the engineering geological environment.</u> Many of the construction materials encountered in tropical regions have distinct behaviour characteristics related to their geological origins and many of the assumptions and empirical relationships that govern the use of temperate materials require to be challenged as to their appropriateness for tropical areas. It is important, therefore to have available a basic engineering geological framework for characterising materials sources and for understanding their likely behaviour patterns in an engineering context.

- <u>Management of information.</u> Resources available for gathering relevant information on construction materials may be limited in many developing countries, where, in addition there may not be the wealth of historical data that is commonly available to engineers in more developed countries. The effective management of information gathering and collation procedures is therefore of critical importance.

- <u>Material selection</u>. The decision processes whereby materials are selected or rejected for particular tasks are at the heart of appropriate materials management. It is frequently necessary to go beyond merely abiding by current specifications and take full cogniscence of the material characteristics, the road environment and the engineering impacts of the in-service roles the materials are required to perform. In doing this it may be necessary to query the relevance of existing specifications and assumptions.

- <u>Appropriate and well-managed materials testing programmes.</u> The prediction of likely engineering behaviour is fundamental to construction material assessment. The production of reliable and quality assured test data is an absolute necessity in any laboratory or field testing programme. Testing methods and governing standards have largely been developed on the basis of the behaviour patterns of temperate soils and rocks in temperate climatic environments, i.e. materials subjected to physical rather then chemical weathering processes. There is a need to re-evaluate the relevance of these established testing procedures, particularly so in the light of current moves to produce pan-European standards and freeze modification of individual national standards. This may be an opportune time to consider the selection of appropriate ”Tropical Engineering” standards.

Additional topics dealing with material excavation and processing, statistics and material stabilisation were incorporated into the programme in support of the principal themes. The key issues of earthworks and drainage, although included within the overall ORN 31 mantle, were not considered within this document.

This document may be viewed primarily as an interim step in the production and dissemination of an Overseas Road Note or series of technical notes acting in support of ORN 31. In a wider strategic context the research associated with this document also fits well with parallel work, such as that on the promotion of marginal, or non-standard, construction aggregates, in the increasingly important area of low-volume, low-cost and access road construction.

1.4 Scope of the Document

Following this introduction Chapter Two presents the geological background to the nature and occurrence of road construction materials in tropical and sub-tropical regions and outlines links with engineering performance in terms of their general material nature, namely

- Hard rock
- Weak rock
- Sands and gravels
- Duricrusts
- Residual soils.

Chapter Three describes the sets of information, both background and specific that assist in the selection of natural road construction materials. A logical framework for information collection is presented that may be integrated with other activities for more general route alignment or earthwork investigations.

Chapter Four discusses the key issues concerning the extraction and processing of materials for road construction, with particular reference to tropical countries. Factors discussed range from aspects of excavation and processing to the management of environmental matters, disposal of unwanted residues and reclamation of exploited land.

Chapter Five deals with the selection and use of naturally occurring materials for pavement construction and earthwork embankments, including in situ sub-grade, imported capping layer and drainage materials. Guidance on the selection process for key road construction materials is presented through a series figures and associated tables.

Chapter Six outlines key aspects in the design and undertaking of material test programmes, both laboratory and in the field, and discusses in detail potential problems associated with specific test procedures in the tropical and sub-tropical environment. Particular emphasis is placed on the selection of appropriate tests and the need for effective quality management throughout the whole testing, reporting and analysis process.

Chapter Seven provides guidance on the sampling of materials and on the statistical tools that may be commonly used in the assessment of test data, while Chapter 8 provides guidance on the stabilisation of soils and gravels for road building by chemical or mechanical means.

2 THE GEOLOGICAL BACKGROUND

2.1 Introduction

The geological background of natural road building materials used in all areas of road construction has a profound effect on the engineering performance of these materials. It is important to have a clear understanding of the geological processes that lead to the formation of rocks and other materials if they are to be used successfully. A knowledge of geology and geomorphology can provide a useful framework for identifying material sources and understanding their likely behaviour patterns in an engineering context.

General soil and rock behaviour can be considered a function of:

- mineralogy of the constituent particles
- morphology of the constituent particles (texture)
- physical relationship of the constituent particles to one another (fabric)
- nature of any discontinuities

In an engineering context soils and rocks behave either at a material scale or a mass scale. Construction materials may be taken as performing largely at the material scale, although aspects of their *in-situ* occurrence have to be considered at the mass scale, Table 2.1

This chapter presents the geological background to the nature and occurrence of road construction materials in tropical and sub-tropical regions and highlights general characteristics of expected engineering performance.

2.2 Principal Rock Groups

2.2.1 General Groups

The Earth's crust is composed of three principal rock groups:

- Igneous rocks; formed from the solidification of molten magma originating from within the earth's crust or the underlying mantle.

- Sedimentary rocks; formed from the consolidation, compaction and induration of the eroded and weathered products of existing rocks.

- Metamorphic rocks; formed by the influences of heat and/or pressure on pre-existing igneous, sedimentary or metamorphic rocks.

2.2.2 Igneous Rocks

Igneous rocks are commonly characterised in terms of their mode of formation and chemistry, the former influencing grain size and the latter governing mineralogy (Figure 2.1). The grain size of igneous is largely a function of the time taken to cool from a molten state, the slower the rate of cooling the coarser the grain size. Plutonic rocks, for example, are typically coarse grained where they form as large intrusions at depth within the earth's crust. Where they form as minor intrusions however (e.g. dykes and sills), they tend to have a more fine to medium grained texture. Additionally extrusive igneous rocks that solidify rapidly as lava flows have a fine to glassy texture.

The chemical characterisation of igneous rocks is done through their silica content (SiO_2) which varies between 40 and 75 % and is its most abundant constituent. Silica content forms the basis of the four way division of igneous rocks into the following categories:-

- Acid
- Intermediate
- Basic

- Ultra-basic

Acid or silica rich igneous rocks typically contain silica in the form of free quartz and abundant pale feldspars. Intermediate and basic rock types have progressively less quartz and greater percentages of dark coloured ferromagnesian, or mafic, minerals. Ultra-basic rocks are composed almost entirely of mafic minerals.

An additional group of igneous rocks known as pyroclastics are formed when molten magma and debris are ejected during volcanic activity in the form of dust, ash or larger agglomeratic fragments (Table 2.2).

2.2.3 Sedimentary Rocks

The formation of sedimentary rocks is done by the consolidation of material eroded from existing rocks, and can be simply defined as a combination of the following activities:

- Erosion of existing rocks into particles or chemical ions.
- Transportation of eroded material as particles or in solution.
- Deposition of transported material by settling of particles or precipitation of solutions.
- Diagenesis of deposited material into rock by processes of compaction, consolidation and cementation.

The nature of the sedimentary rock formed by this process is therefore a function of the mineral composition of the source rock or material and the interaction between the environment of transport, deposition and diagenesis.

Sedimentary rocks are generally considered to fall within three major subdivisions:-

- Detrital Clastic Rock: formed primarily from rock or mineral particles.
- Chemical Rocks: Formed either from chemical deposition or evaporation or combinations of both.
- Bio-Chemical Rocks: Formed by the accumulation of organic debris aided by chemical deposition.

These three subdivisions have further classifications depending on the type of sedimentary rock formed. Detrital rock classification is based on the grain size of the constituent particles while chemical and biochemical rock classification is based on the most abundant component of the rock (see Tables 2.3 & 2.4)

2.2.4 Metamorphic Rocks

As stated earlier, metamorphic rock is formed by action of heat and pressure on pre-existing rock. Depending on the rock being metamorphosed, the rock formed can vary greatly. In generally however metamorphic rocks are divided into three groups, two major and one minor, based on the metamorphic process applied during formation as follows:

- Major metamorphism process:-
  - (a) Thermal metamorphism. Thermal metamorphic rocks are formed by the imposition of high temperatures with only minor pressure effects. They are commonly associated with igneous intrusions and are often described as the products of contact metamorphism. Contact metamorphic rocks are generally localised as concentric shells, or aureoles of rock material surrounding an igneous body intruded into the in the upper levels of the Earth's crust.
  - (b) Regional metamorphism is a large scale process associated mainly with mountain building and large scale tectonic movements. It involves high temperature and pressure, with the latter being dominant. This form of metamorphism tends to occur over large areas and generally shows a progressive decrease in alteration away from the heat or pressure centre.

- Minor metamorphism process:-

Dynamic metamorphism, occurs when intense localised stress break-up existing rock to produce breccias and cataclastic rocks.

Geological classification of metamorphic rocks can vary in complexity depending upon whether the classification is based on the end-product of the metamorphic process or the original metamorphosed rock. The most commonly used procedures are based on the following:

- nature of the original rock.
- diagnostic mineral or metamorphic grade.
- association of mineral assemblages.

In an engineering context it is logical to deal with the end-product terminology. In general terms metamorphic classification can be considered as function of mineralogy, texture and fabric, Table 2.5. This essentially descriptive approach, which is based on the discernible characteristics, is most applicable for every day engineering classification.

2.3 Weathering

2.3.1 Processes

Weathering is the breakdown and alteration of material at or near the Earth's surface yielding products that are more stable in with the prevailing physio-chemical conditions. Weathering can cause some displacement of the resultant debris, however substantial removal of this material from the weathering site is referred to as erosion.

Weathering is a combination of physical degradation of a rock and chemical alteration of its constituent minerals. Biological activity is sometimes considered a separate category of weathering. However, as it contains aspects of physical disintegration and chemical alteration, it is more appropriately considered a subsidiary activity within the two main processes. Table 2.6 and Table 2.7 summarise the key physical and chemical weathering activities and Figure 2.2 illustrates their significance in terms of weathering products.

The distinction between physical and chemical weathering is somewhat arbitrary as they rarely operate in isolation. In the majority of instances, particular in tropical and sub-tropical environments, they interact in the overall weathering environment. The relative importance of these two mechanisms within a weathering environment is a function of climate, (and to some degree altitude) particularly temperature and rainfall (Figure 2.3). In tropical and sub-tropical areas chemical decomposition dominates the weathering process (Figure 2.4).

The end product of the chemical weathering of rocks is an engineering material. This material reflects the combined effects of present and past climates, vegetation, human activity and the lithology of the parent materials. Physical degradation although playing a lesser overall role, does play a significant part, particularly in the initial stages of tropical weathering in structured rocks. For example, discontinuities in the rock, such as cracks and joints, promote the chemical weathering process. Little (1969) briefly outlined the sequence of tropical weathering as follows:

> "Near to the surface the joints are open and water flows readily along them carrying the biochemical products of vegetable and corrosive compounds in solution. These solutions attack the rock; weathering starts along rock joints and spreads from them into the body of the rock"

Fookes et al (1988) summarised the three simultaneous processes involved in chemical weathering as:

1. Breakdown of the parent material structure with the concomitant release of the constituent elements as ions or molecules
2. The removal in solution of some of these released constituents
3. The reconstitution of the residue with components of the atmosphere to form new minerals that are in stable or metastable equilibrium with the environment.

The relative stability, or resistance to chemical decomposition of the rock-forming minerals, is indicated by

Goldrich's adaptation of the Bowen Reaction Series (Bowen 1928) and is shown in Figure 2.5. This has important implications on the nature of material sources and the relative durability of aggregate materials within an engineering time-scale. The timescale within which tropical weathering takes place is highly variable, and is dependant on a range of factors including rock type, climate, geomorphology and geological stability. In some cases weathering can take place over a relatively short time-scale, particularly within some igneous materials. In contrast, some processes, such as the formation of silcretes, may involve long periods of geological time.

2.3.2 Transported Materials

Erosion and transportation remove the products of weathering to new sites of deposition. The weathering products may be transported either as solid particles or as chemical ions in solution. Gravity and water are the principal agents of erosion and transportation, although in some dry environments wind can play a significant role.

Solid particles are transported until the energy levels within the transporting medium drop sufficiently to allow their deposition. A gradual decay of energy in, for example, a stream or river allows heavier or coarser material to be deposited first followed by progressively finer material. In extreme cases this results in the formation of very single-sized material resources. In contrast a sudden drop in energy will result in the deposition of unsorted material with a wide range of grain sizes. From a construction materials viewpoint sorting and shape modification of solid particles is an important consequence of the transportation phase.

Eroded particles also undergo physical degradation due to the attritional effects of impact and grinding. Angular particles will become progressively more rounded and equi-dimensional in shape with increased transport attrition effects. Some materials, however, with inherent anisotropy are resistant to these rounding effects.

In finer sediments physical degradation can be accompanied by mineralogical sorting due to the unique internal structures and relative strength of individual mineral types. Quartz, for example, is a highly resistant mineral that tends to form well rounded sediment whereas feldspar will tend to break down and decay.

The engineering properties of materials derived as a result of erosion, transportation and deposition, is a function of the following geological factors:

Parent Geology: The original bedrock mineralogy and structure have a fundamental influence on material properties.

Sediment transport: The duration and intensity of the transport impacts upon clast sorting and shape

Depositional environment: Defines the morphology of the sediment mass. Changes in environment will result in changes in the nature of deposited materials. A rapidly changing environment may lead to variable layered deposits.

Geological history; Factors such as changing climate, tectonic movements and variations in sea levels may influence the formation and characteristics of deposits

2.3.3 In Situ Materials

Tropical weathering of a soil-rock mass leads to the decomposition of the constituent minerals to stable or metastable secondary products, usually in the form of clay minerals (phyllosilicates), oxides and hydroxides. The type of soil that forms in these regions are totally different to those formed in temperate regions which tend to be geologically young soils dominated by fine grained quartz, illite/muscovite and 2:1 layer silicates (Mitchell 1993). Mature, well drained tropical residual soils tend to be dominated by kaolinites (including halloysite), gibbsite, hematite and goethite. Poorly drained soils are likely to be dominated by smectite clays.

Weathered products at depths not removed by erosion, can form in situ profiles ranging from fresh bedrock to residual soils at the surface and may, under appropriate conditions develop pedogenic duricrust deposits such as laterite, calcrete or silcrete (Table 2.8). Where the rate of predominantly chemical weathering exceeds the mass erosion rate, tropical and sub-tropical environments provide suitable conditions for the development of residual soil-rock profiles, as shown in Figure 2.6.

A climatic index (N) value of less than 5 indicates that conditions suitable for the formation of a weathering mantle (Weinert 1980). In the Southern African environment where N lies between 2 and 5 montmorillonite is commonly developed as an end product from the weathering of feldspars and mafic minerals. If $N < 2$, montmorillonite is no longer stable and decomposes to kaolinite and when $N <1$ kaolinite itself tends to breakdown to bauxite.

Although the formation of pedogenic duricrusts involves the movement of chemical ions either within or adjacent to the bedrock, they are generally considered to have formed in situ. Some formational processes include disintegration and downslope movement of existing duricrust fragments, however, in engineering terms, they are not considered as separate from other duricrusts.

A fundamental requirement for the development of a residual soil-rock profile is that the rate of formation must exceed the rate of erosion. High erosion rates may modify the profile, particularly on steep slopes or within land systems subject to recent uplift and denudation. Tropical weathering patterns may be highly variable both horizontally and vertically.

The geotechnical character of a tropical weathered soil-rock mass is a function of the chemical weathering process allied to any relict characteristics. Within the residual soil-rock horizon there is usually a variety of minerals resulting from one or more of the following:

- Variable rates of parent mineral alteration to new minerals depending on the silica content, the concentration of ions present, the soil pH and the amount of leaching.
- Inherited clay minerals from parent material.
- Clay minerals formed under previous chemical weathering environments.
- Relict primary minerals.

Quartz, and to lesser extent muscovite mica, are the most resistant of the common rock forming minerals to chemical weathering (Figure 2.5) and as a consequence they tend to remain within soil profiles. The concentration of residual quartz can result in either the occurrence of quartz beds within a residual soil profile or the enhancement of any associated pedogenic gravels.

2.4 Structure

2.4.1 General

The geological structure of rock encompasses a wide range of geometrical forms and arrangements. Figure 2.7 illustrates components of structure in relation to scale. Key elements of structure that influence the selection and use of construction materials at both the mass and material scales include:

- Folds
- Discontinuities
- Texture
- Fabric

2.4.2 Folds

Folds are formed in rock strata that respond in a ductile plastic manner to compressional stresses. Folds are commonly defined with respect to their geometric cross-sectional form. A fold with limbs dipping outward from a central axis is termed anticlinal, whilst a fold with inward dipping limbs is termed synclinal.

Further definition is possible in terms of the angle between the limbs, the attitude of the fold axis and the relative behaviour of the strata making up the fold (Figure 2.8)

Folds influence construction resources most obviously at the mass scale, where they determine economic resource boundaries by governing outcrop patterns. They can also contribute to quarry slope stability in conjunction with other structural elements, (cf chapter 4)

2.4.3 Discontinuities

Discontinuities can be considered as planes of weakness, either open or tight, that separate elements of soil-rock material or mass. Discontinuity scales run from very large regional faulting to microscopic mineral cleavage. Table 2.9 summarises the relevant range of discontinuity types as they influence construction material resources.

Within a tropical weathered soil-rock mass, discontinuities are usually inherited features in the form of joints, bedding, faults, etc. Some new-formed fissuring can be present as the result of formational soil movement, particularly in swelling vertisols, as a result of alternating reducing and oxidising conditions in tropical monsoonal climates. .

2.4.4 Texture

Texture operates largely within the material scale and may be considered as defining the physical character of a soil or rock's constituent particles or minerals. The common elements of texture are particle size, angularity and shape (Figure 2.9).

2.4.5 Fabric

Fabric may be defined as the spatial relationship of a soil or rock's constituent elements. Elements of fabric, although more generally operating at the material scale, can also be used to define a soil-rock mass. Table 2.10 describes elements of fabric as they impact upon construction materials.

2.5 Classification and Characterisation of Materials Sources

2.5.1 General Classification

There have been a number of attempts to classify construction materials into broad groups, based either solely on soil-rock type (BS 812 1975) or in combination with expected performance, (Lees 1968; and Weinert 1980). These have not been widely accepted as applicable to tropical and sub-tropical regions, where the occurrence of weathered soil-rock profiles adds additional complexity to an already intricate suite of materials

A general classification framework for material sources is useful, however, to help assess potential utilisation, recognise possible problems and programme investigations. The occurrence of technically viable sources of construction materials results from the interaction of geological, hydrological and geomorphological processes. It follows that a basic classification of such sources may be based on their general geological character, such as:

- Hard-rock sources
- Weak-rock sources
- Transported sediments (or soils)
- Residual Materials.

Table 2.11 provides a definition and further detail of these groups. Where required, suggestions by the Working Party Report on Aggregates (Geological Society, 1993) provide more detailed descriptions along with suitable geological petrographic names.

2.5.2 Hard Rock Sources

The nature of hard-rock sources is largely governed by basic geological principles of rock mineralogy and structure. Table 2.12 gives some guidance on the character of common igneous, sedimentary and metamorphic rock types within this group.

Road construction materials excavated from hard-rock sources are generally won by standard quarry, drill and blast excavation procedures and will normally require crushing and sizing (cf Chapter 4). to quarry development costs, make these materials expensive and this should be reflected in the care taken with pre-development investigations and the subsequent use to which they are put.

2.5.3 Weak Rock Sources

There are a wide range of weak-rock materials that may be utilised in road construction. In tropical and sub-tropical regions these include igneous, sedimentary and metamorphic rock types whose original character has been altered by weathering. Table 2.13 describes some common weak-rock types together with some general guidance on their suitability.

In general it is likely that weak-rock materials would only be used in lower volume roads where their cheaper development and excavation costs and wider availability in many developing countries makes them attractive in low-cost infrastructure development.

2.5.4 Transported Soils

The majority of transported materials used for road construction in tropical and sub-tropical regions are sands and gravels of fluvial origin, such as channel, flood plain, river terrace and alluvial fan deposits. Their occurrence may be related to previous climatic and geomorphological environments. High rainfall climates are known to have occurred during Pleistocene times, and hence alluvial sands and gravels are encountered in areas which currently have dry environments

Channel deposits: In strong flowing streams or rivers the fine sediment load may be washed out of the sand and gravel The sand and gravel may subsequently be deposited in channels to form potential sources of construction material. These, generally elongated masses, may be seasonally replenished by fresh deposits, particularly in large river systems adjacent to eroding mountainous areas. Winning of material from river channel deposits has to take into account factors such as potential flooding of the area and any environmental impact by pollution.

Flood plain deposits: During floods, sand and gravel, as well as fines, may be spread over river flood plains. These deposits are likely to be composed of bedded units, each of which may become progressively finer towards the top.

Terrace deposits: Terrace deposits are the dissected remains of former flood plains left behind on the valley flanks as the river erodes down into its channel. The extent of their formation may have been enhanced by changes in sea level in the recent geological history.

Alluvial fans: Alluvial fans are formed when a stream or river carrying a large sediment load a sudden change in slope and suffers a rapid drop in bedload capacity. These fans tend to be variable in thickness, and tend to contain well graded angular fragment. Their formation may be enhanced by tectonic uplift, in recent geological time

In coastal regions sands and gravels accumulate as beach deposits on coastal margins between low tide and top of storm levels. Constant wave and tide action leads to the accumulation of well rounded sand and gravel deposits. These may be limited vertically but can be extensive along shorelines. Any utilisation of these materials should take into account the environmental consequences of sediment extraction.

2.5.5 Residual Soils and Duricrusts

The tropical weathering process results in the development of residual soil profiles. These materials which occur at or near the surface, have low development and excavation costs and are therefore commonly used in road construction. They are normally worked as shallow borrow pits, therefore the environmental implications of extraction need to be carefully considered.

Table 2.14 presents general information on these materials under the sub-headings of saprolitic soils; residual soils, residual gravels and pedogenic duricrusts. The latter can potentially be used as sub-base and roadbase materials and in some low volume roads as surfacing aggregates. Their suitability for use in road construction is strongly influenced by the strength of the cemented particles.

Scale Level		Scale of Influence
Mass		In situ character of pit or quarry. Stability of quarry slopes or road side slopes (boundaries of resource). Character of pavement or embankment layers
Material	Hand Specimen	Intact rock samples; compacted soil or aggregate
	Visible Particle	Individual aggregate particles, e.g. character of sand or gravel clasts.
	Microscopic	Character of minerals or fine particles (e.g. clays) either within larger samples or individually.

Table 2.1 Levels of Geotechnical Behaviour

Pyroclastic Type Particles (or grains) size (mm)	**Description**	
	Unconsolidated	**Consolidated as rock**
60+	Volcanic bombs & ejected rock blocks	Agglomerate Volcanic breccia
2-60mm	Fine strands and droplets of lava ejected into the atmosphere	Lapilli tuff
0.06-2	Volcanic ash	Tuff
0.002-2	Volcanic dust	Tuff

Table 2.2 Geological Classification of Pyroclastics (Blyth and de Freitas 1994)

<table>
<tr><th rowspan="2">Grain or Clast Size</th><th colspan="3">Detrital Sedimentary materials</th></tr>
<tr><th>Unconsolidated
Uncemented</th><th colspan="2">Indurated Rock</th></tr>
<tr><td>200mm</td><td>Boulder</td><td colspan="2" rowspan="3">Conglomerate</td></tr>
<tr><td>60mm</td><td>Cobble</td></tr>
<tr><td>4mm</td><td>Pebble</td></tr>
<tr><td>2mm</td><td>Gravel</td><td colspan="2">Gritstone</td></tr>
<tr><td>0.06mm</td><td>Sand</td><td colspan="2">Sandstone
(Arkose, greywacke,
Quartzite etc)</td></tr>
<tr><td>0.002</td><td>Silt</td><td>Siltstone</td><td rowspan="2">Mudstone</td></tr>
<tr><td></td><td>Clay</td><td>Claystone</td></tr>
</table>

Table 2.3 Classification of Detrital Sedimentary Rocks

Degree of Induration	Compressive Strength	Grain Size <0.002mm	Grain Size 0.002 -0.06mm	Grain Size 0.06 - 2.00mm	Grain Size >2.00mm	Carbonate Content
Non Indurated	Very Soft to Soft	Carbonate Mud	Carbonate Silt	Carbonate Sand	Carbonate Gravel	>90%
	36 –300 kPa	Clayey Carbonate Mud	Siliceous Carbonate Silt	Siliceous Carbonate Sand	Mixed Carbonate Gravel	50-90%
Slightly Indurated	Hard to Moderately Hard	Calcilutite	Calcisiltite	Calcarenite	Calcirudite	>90%
	0.3 - 12.5 MPa	Clayey Calcilutite	Siliceous Calcisiltite	Siliceous Calcisiltite	Conglomeratic Calcirudite	50-90%
Moderately Indurated	Moderatley Strong to strong	Fine Grained Limestone	Fine Grained Limestone	Detrital Limestone	Conglomerate Limestones	>90%
	12.5 - 100 MPa	Fine Argillaceous Limestone	Fine Siliceous Limestone	Siliceous Detrital Limestone	Conglameratic Limestone	50-90%
Highly Indurated	Strong to Very Strong >100 MPa		Crystalline	Limestone		>50%

Table 2.4 A Geotechnical Classification of Limestones (Based on Clark and Walker, 1977)

Geotechnical Grouping	Typical Rock Type
A. Isotropic	Massive Quartzite
	Marble
	Hornfels
B. Moderately Anisotropic	Gneiss
	Bedded Quartzite
	Amphibolite
C. Strongly Anisotropic	Shale/Slate
	Phylllite
	Schist

Table 2.5 A Basic Classification of Metamorphic Rocks

Process	Description
Stress Release	Following denudation by erosion, stresses "locked-in" to a rock mass can be released, the mass undergoes elastic expansion and forms sheet joints. Stress release can also occur at mineral boundaries. "Strained" quartz, within regional metamorphic rocks, can break down to form fine sediment on exposure.
Insolation	Occurs primarily in desert areas where high daytime temperatures are followed by low night-time temperatures. This large diurnal variation (up to 40^{O}C) results in physical splitting, spalling and flaking (particularly fine grained mafic mineral-rich rocks and those composed of minerals with very different co-efficients of expansion).
Slaking	A physio-chemical process whereby the alternate wetting and drying of rock materials, particularly mudstones and shales, results in their disintegration and the formation of debris of angular flakes or muddy sediment.
Salt Expansion	The crystallisation of supersaturated solutions of salts occupying fissures and pore spaces within rocks and imparting expansive stresses to joint boundaries and mineral boundaries.
Mineral Expansion Stress	Chemical alteration of minerals can have physical disintegration consequences, for example, the alteration of biotite to montmorillonite may be accompanied by up to a 40% increase in volume, resulting in physical fracturing of the adjacent rock fabric.

Table 2.6 Processes of Physical Disintegration

Process	Description
Hydrolysis	Thought to be the most important chemical decomposition process. In general terms: Silicate + H_2O + H_2CO_3 = Clay mineral + cations + OH^- + $HCO3^-$ + H_2SiO_4
Solution	Solution and leaching are important factors in the development of weathering profiles and are closely associated with hydrolysis. In general terms rain soaks into the ground, containing CO_2 derived from the atmosphere and from the humus zone. Ions become dissolved in the CO_2 enriched water, and the solution proceeds downwards under the hydraulic gradient, until a state of saturation is reached.
Oxidation	{The process of losing electrons with a resultant gain in positive valency.} Oxygen is the most common oxidising agent and iron the most commonly oxidised element in a profile. On being released by hydrolysis into an aerobic environment Fe^{2+} quickly oxidises to Fe^{3+}.
Reduction	Reduction is the reverse of the oxidation process. Reduction of iron from the ferric to its ferrous state takes place in anaerobic weathering environments.
Hydration-Dehydration	The process whereby an original mineral takes up water molecules to form a new mineral. An increase in volume may be associated with this type of reaction. Dehydration is the reverse process. These processes are limited to a few minerals, such as haematite which can hydrate to goethite and halloysite which can dehydrate to metahalloysite
Chelation	Chelation involves the complexing and removal of metallic ions. Chelating agents are formed by biological processes in the soil aided by lichens growing on rock surfaces and render substances more soluble under certain pH conditions.

Table 2.7 Processes of Chemical Alteration

Classification/Subdivisions	General Description	Common Terms
SILCRETE a. Grain supported fabric b. Floating fabric c. Matrix fabric d. Conglomeratic	Indurated deposits consisting mainly of silica, which may have been formed by lateral or vertical transfer. Subdivision on the basis of fabric	Silcrete
CALCRETE a. Calcified soils b. Powder calcrete c. Nodular calcrete i Nodular ii Concretionary d. Honeycombe calcrete i Coalesced nodules ii Cemented e. Hardpan calcrete i Cemented honeycombe ii Cemented powder iii Recemented iv Coalesced nodules v case hardened calcic f. Laminar g. Boulder	Variably indurated deposits consisting mainly of Ca and Mg carbonates. Includes non-pedogenic forms produced by fluvial or groundwater action, otherwise by lateral or vertical pedogenic transfer. Subdivision usually on basis of degree and type of cementation.	Calcareous soil becoming hardpan, calcrete or dolocrete with increasing concretionary growth.
LATERITE a. Water table cuirasses i Local ii Plinthite iii Petroplinthite b. Plateau cuirasses *Or* a. Pisolitic b. Scoriaceous c. Petroplinthite	A form of indurated deposit containing accumulations of sesquioxides, mainly iron, within one or several soil horizons. It may be formed by deposition from solution, moving laterally or vertically, or as a residue after removal of silica, alkalis etc. It may be pedogenetic by retention or accumulation of minerals and by segregation within vadose profiles. May also be reworked or detrital. Subdivision on the basis of degree and type of induration.	Ferricrete/Latosol (red) soil or plinthite becoming hardpan or laterite with increased concretions or induration
ALUCRETE a. Pisolitic b. Scoriaceous c. Petroplinthite	A form of variably indurated deposit containing Al and Fe in residual laterite deposits, with Al in sufficient quantity to be of commercial use. Otherwise similar to ferricretes	Bauxite,Alucrete

Table 2.8 Classification of Duricrusts (after Geological Society,1996)

Discontinuity	Definition	Construction Material Impacts
Faults	Brittle fractures produced by tensional or compressional forces within a rock mass along which there has been an observable displacement. May be normal, reverse or wrench in character.	Operating mainly at the mass scale, faults have impacts on the nature of source areas in terms of defining geological boundaries and quarry slope stability. Faults and associated shear/shatter zones can act as hydrological conduits and may promote hydrothermal alteration and weathering of the rock
Joints	Brittle fractures produced by tensional or compressional forces within a rock mass along which there has been no observable displacement. Likely to occur in parallel groups called sets and combinations of sets called systems.	At the mass level, joint systems can influence methods of material extraction in hard rock quarries. They may also govern quarry-slope stability. At the material level they can act as hydrological conduits and may promote differential weathering of materials.
Bedding Planes	Planar surfaces parallel to a surface of sedimentary deposition.	Bedding planes have impacts at the mass level in terms of defining material type and quality boundaries and potentially influencing quarry slope stability.
Laminations	Very closely spaced bedding planes.	Similar impacts as bedding planes but with the additional impact of influencing aggregate shape and strength at the material scale.
Rock Cleavage	Slaty cleavage: very closely spaced planes developed in fine grained rocks as a result of intense deformation and the partial recrystallisation of platy minerals perpendicular to the direction of compressive forces. Fracture cleavage: planes produced by folding.	Major impacts on the shape and strength of aggregates and on the mechanical durability in service.
Mineral Cleavage	Planar surfaces within individual minerals formed in response to their atomic structure and crystal form. Each mineral type has consistent cleavage pattern, e.g. mica: one distinct; hornblende two and quartz none.	Influences the character and performance of materials at the visible particle and microscopic levels, e.g. the compatibility of muscovite-mica rich soils, the high mechanical integrity of quartz and the susceptibility of cleaved minerals to weathering and alteration

Table 2.9 Geological Discontinuities

Discontinuity	Definition	Construction Material Impacts
Shear Plane	Planes produced by the shearing action of rock or soil failure.	Principally impact at the mass level by influencing quarry slope stability. Often associated with poor or altered minerals May also be produced within compacted soil fills during construction and influence the constructed road at the mass level.
Foliation Planes	Planes of anisotropy produced by parallel orientation of platy minerals within rocks.	Impacts on the character and performance of materials by influencing natural and processed aggregate shape and durability. They can also enhance weathering.
Banding/Layering	Metamorphic: produced by the segregation of minerals under conditions of high temperature and pressure, e.g. banded gneiss Igneous: produced at the boundaries of individual lava flows (e.g. basalts) or internally within highly viscous lava types (e.g. rhyolite)	Metamorphic: Impacts on the character and performance of materials by influencing natural and processed aggregate shape. Igneous: banding/layering can impact at the mass scale by influencing slope stability. Also at the material scale by introducing variability in mineralogy and hence durability
Fissures	Similar in nature to joints but the terminology is generally reserved for use with soil-like materials	May impact at the material scale by allowing the introduction of deleterious inclusions. May impact on the finished road at he mass level by allowing ingress of water into embankment fill and sub-grade.

Table 2.9 Geological Discontinuities (Continued)

Fabric Group	Description	Definition	Impacts
Types	Crystalline	Tightly interlocked crystals, characteristic of medium to coarse grained igneous and some metamorphic rocks. After initial weathering this can degrade to a separated crystalline fabric. Further chemical alteration can lead to a Relict Crystalline when the fabric retains the crystalline form but the minerals are substituted by weaker clay minerals.	Tight crystalline fabric will impart high strength (e.g. dolerite) A separated fabric gives rise to significantly weaker and less durable materials Relict crystalline fabric may be subject to collapse.
	Granular	Separated identifiable clasts or minerals either enclosing interstitial fine material (grain supported) or contained within a finer matrix (matrix supported)	Tend to be less strong and less durable than crystalline fabrics. Matrix supported materials in particular may be liable to poor grading and durability.
	Blocked/Fissured	Material separated by a system of small fissures in blocks or "peds". Typically encountered in residual soil materials. May be termed "fissure-block" fabric	Impacts on the amount of soil break-down under compaction and the relationship between laboratory and field compaction characteristics
Orientation	Strong	> 60% of particles oriented with long axis within 30^{O} of each other	Inherent fabric orientation will impact on the shape characteristics of processed aggregate
	Moderate	40-60% of particles oriented with long axis within 30^{O} of each other	
	Weak	20-40% of particles oriented with long axis within 30^{O} of each other	
	Random	No apparent orientation in visible particles	Random orientation is likely to enhance aggregate clast strength.
Relative Grain or Mineral Size	Equi-granular	Clast or grains predominantly of one size group,	Inherent clast size likely to influence aggregate grading, particularly in granular fabric types
	Inequi-granular	Wide range of clast or grain sizes	
	Porphyritic	Distinctly larger mineral crystals or clasts within finer matrix.	

Table 2.10 Fabric Description

Resource Group	Description	General Material Impacts
Hard-Rock	Strong to very strong igneous, sedimentary and metamorphic rock types normally requiring drill-and blast quarrying techniques for excavation.	Materials require crushing and classifying before being utilised as road aggregates. Relatively high cost quarry development and material processing.
Weak-Rock	Weak to very weak igneous, sedimentary and metamorphic rocks that may be excavated by mechanical means, including ripping where necessary. This group includes rocks that have been weakened by weathering processes.	Materials may require some processing before being utilised for road pavements.
Residual Soils and Duricrusts	Soil-like materials that have been formed largely in situ by tropical and sub-tropical weathering processes. Materials generally excavated by borrow-pit techniques. Occasionally indurated duricrust may require ripping.	Depending on the in situ character these materials may be utilised as-dug or classified for fill, capping layer, sub-base or roadbase. Would generally not be considered for surfacing aggregates with the possible exception of processed duricrusts and residual quartz gravels.
Transported Soils	Soil-like materials such as sand and gravel that have undergone processes of erosion, transportation and deposition in addition to weathering. Materials generally excavated by borrow-pit techniques	Sound gravel and cobble materials can be processed to produce high quality aggregates. The sorting action of erosion and transportation may result in materials lacking in some particle sizes.

Table 2.11 Resource Group Classification

Rock Type	General Description	Str	Du	Sh.	Ad.	Potential Problems
Granite	Coarse grained, light coloured acid plutonic rock. Contains quartz, feldspars and possible micas	4	4	4	2-3	Susceptible to deep and variable weathering e.g. kaolinisation of feldspars
Diorite	Coarse to medium grained, intermediate plutonic rock composed chiefly of plagioclase feldspar and hornblende with minor orthoclase feldspar and quartz.	5	5	5	4	
Gabbro	Coarse grained, dark basic rock composed largely of plagioclase feldspars and augite. Hornblende olive and biotite may also be present.	4	4	4	5	Possible internal mineral variability, with an impact on weathering (smectites) and hence durability.
Peridotite	Coarse grained, dark ultra-basic intrusive, composed chiefly of olivine and pyroxenes (augite) with some iron oxides	4	1-2	4	5	Mafic mineral alteration with an impact on durability
Dolerite	Medium grained tightly crystalline dark basic minor intrusive composed largely of plagioclase feldspars and augite	5	2-3	5	5	Variability at intrusion edges.
Rhyolite	Light coloured fine grained acidic lava composed essentially of quartz and plagioclase feldspar	4	4	3	4	Flow banding, anisotropic character and poor shape.
Andesite	Fine grained intermediate lava composed essentially of plagioclase feldspar and mafic minerals (hornblende, biotite, augite)	4	4	3	5	Some possible flow banding, anisotropic character and poor shape
Basalt	Fine-grained dark basic lava. Composed largely of plagioclase feldspars and augite and sometimes olivine. Can contain infilled vesicles.	4	1-3	3	5	Varieties rich in olivine/chlorite susceptible to rapid deterioration and disintegration. Aggregates can be susceptible to disintegration problems in service

Table 2.12 Hard-Rock Material Types: a) Igneous Rocks

Notes: St: Aggregate Strength. Du: Durability. Sh: Likely processed shape. Ad: Likely bitumen adhesion
1: Very poor 2: Poor 3: Moderate 4: Good 5: Excellent

Rock Type	Description	Str	Du	Sh.	Ad.	Potential Problems.
Quartzitic Sandstone	Medium grained detrital sedimentary rock with clasts composed of quartz particles, fabric may be cemented by silica, iron oxides or carbonates.	3	2-4	4		Great variability, a function of fabric and matrix. May be interbedded with weaker materials.
Arkose	Medium grained detrital sedimentary rock with clasts composed of predominantly feldspar particles, fabric may be cemented by silica, iron oxides or carbonates.	3	2-3	4	3	Feldspar may be altered. Feldspar inherently weaker than quartz. May be interbedded with weaker materials.
Greywacke	Frequently dark coloured compact detrital sedimentary rock composed of poorly sorted angular fragments of quartz, feldspar rock within a fine matrix.	4	4	2-3	-	Generally a good strong material when fresh. May contain deleterious minerals in matrix.
Conglomerate	Coarse grained detrital sedimentary rock. Generally composed of boulders, cobbles and gravel sized fragments in fine matrix.	3	3-4	4	4	Very variable. Processed grading a function of clast-matrix relationships.
Siltstone	Similar to sandstone but with predominantly silt-sized particles.	3-4	2-3	3	-	Tends to be interbedded with other sedimentary materials, including mudstone.
Crystalline Limestone	Consist essentially of crystalline calcium carbonate. If magnesium carbonate then the term Dolomite is appropriate. May contain minor amounts of non-carbonate detritus.	4	4	5	5	

Table 2.12 Hard-Rock Material Types: b) Sedimentary Rocks

Notes: St: Aggregate Strength. Du: Durability. Sh: Likely processed shape. Ad: Likely bitumen adhesion
1: Very poor 2: Poor 3: Moderate 4: Good 5: Excellent

Rock Type	Description	Str	Du	Sh.	Ad.	Potential Problems
Shale/Slate	A very low-grade metamorphic rock in which cleavage planes are pervasively developed throughout the rock.	2	1	1	-	Poor durability and shape. Marked tendency to split along cleavage planes (fissile).
Phyllite	A low-grade metamorphic rock characterised by a lustrous sheen and a well-developed foliation resulting from the parallel arrangement of sheet silicate minerals. Slightly coarser than slate.	3	3	2	-	Poor particle shape. Possibility of free mica being produced during processing.
Schist	A medium to high-grade metamorphic rock characterised by the parallel alignment of moderately coarse grains usually visible to the naked eye. The preferred orientation described as schistosity.	4	3	1-2	-	Poor particle shape. Possibility of free mica being produced during processing.
Amphibolite	An essentially bimineralic dark green rock made up of hornblende and plagioclase feldspar. Mostly formed from basic igneous rocks (metabasites)	4	3	2-3	4	Tendency for the aggregate to be elongate in shape. May contain deleterious minerals.
Gneiss	Medium to coarse mineral grains with a variably developed layered or banded structure, minerals tended to be segregated, e.g. quartz, feldspar and mafic mineral banding. Described as gneisic.	5	4	3	3	Some potential shape problems – though less than phyllite/schist.
Quartzite	A contact metamorphic rock formed from a quartz rich sandstone or siltstone. Contains more than 80% quartz.	5	5	4	2-3	Strained quartz mineral grains may break down to give silica rich fines. Abrasive to construction plant.
Hornfels	A fine to medium-grained metamorphic rock granofels formed in contact aureoles (zone of metamorphism); and possessing a tough and not easy to break character. Commonly formed from fine grained sedimentary rocks.	4	4	4	4	May contain deleterious minerals.

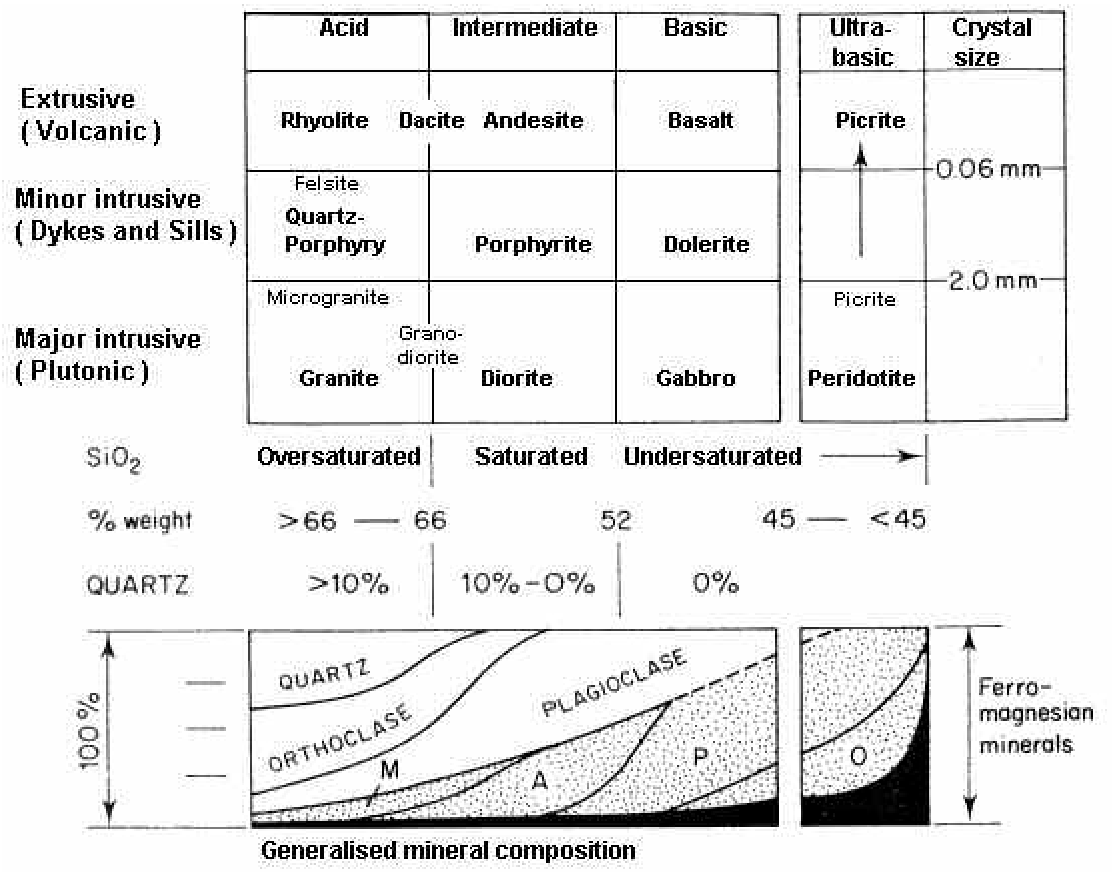
Table 2.12 Common Hard-Rock Material Types: c) Metamorphic Rocks

Rock Type	Potential Uses	Potential Problems
Mudstone	As embankment fill and possible selected fill/capping layer material	Very low particle strength. Potential for slaking and swell/shrink in wet climates.
Shale	As embankment fill and selected fill/capping layer material. Possible use as sub-base material in dry climates.	Potential for slaking and swell/shrink in wet climates. Requires care in compaction for embankment fill as breakdown of material in a voided rockfill could lead to in service settlement.
Weak limestones	As embankment fill and selected fill/capping layer material. Possible selected use as sub-base or roadbase material for low volume roads.	Possible poor as-dug gradings. Low particle strength and in service deterioration.
Weak Sandstones	As embankment fill and selected fill/capping layer material. Possible use as sub-base or roadbase material in dry climates.	Possible poor as dug grading. Low particle strength and potential for in service deterioration.
Pyroclastics	As embankment fill and selected fill/capping layer material. Possible selected use after processing as sub-base or roadbase material in lower volume roads.	Low particle strength, high material void ratios and water absorption. Potential for deleterious mineral inclusions
Weathered Hard Rocks	As-dug: As embankment fill and selected fill/capping layer material. Roadbase material for low volume roads Processed: Potential use as sub-base and roadbase materials	Problems highlighted in Table 2.12 will be accentuated by weathering. Particular problems associated with the rapid deterioration of weathered basic igneous materials.

Table 2.13 Typical Weak Material Types

Rock Type	Description	Potential Use
Saprolitic soil	Soil-like material within the weathering profile that has retained the relict structure of the parent rock.	Generally used for common fill. Relict structure can cause problems resulting from over-compaction and break-down of material fabric. Saprolitic soils developed over phyllites and schist may have problems resulting from high mica content.
Residual Soil	True residual soil has developed a new-formed fabric to replace the relict forms in saprolitic material. Generally overlies Saprolitic layers.	Used for common fill. Generally less problems than with saprolitic soil.
Residual Gravel	Concentrations of weathering resistant quartz within residual soil profiles.	Usability as-dug is a function of the ratio of fines to gravel. Commonly used as sub-base and, if processed or stabilised, as roadbase material
Duricrusts	Silcrete, calcrete and laterite, as described in Table 2.2.8 "As dug" materials highly variable in strength, size and durability	Commonly used as sub-base and stabilised roadbase. Potential problems reported with calcrete stabilisation. Higher plasticity materials will be subject to significant loss of strength on saturation and so use often restricted depending on climatic conditions, and design traffic

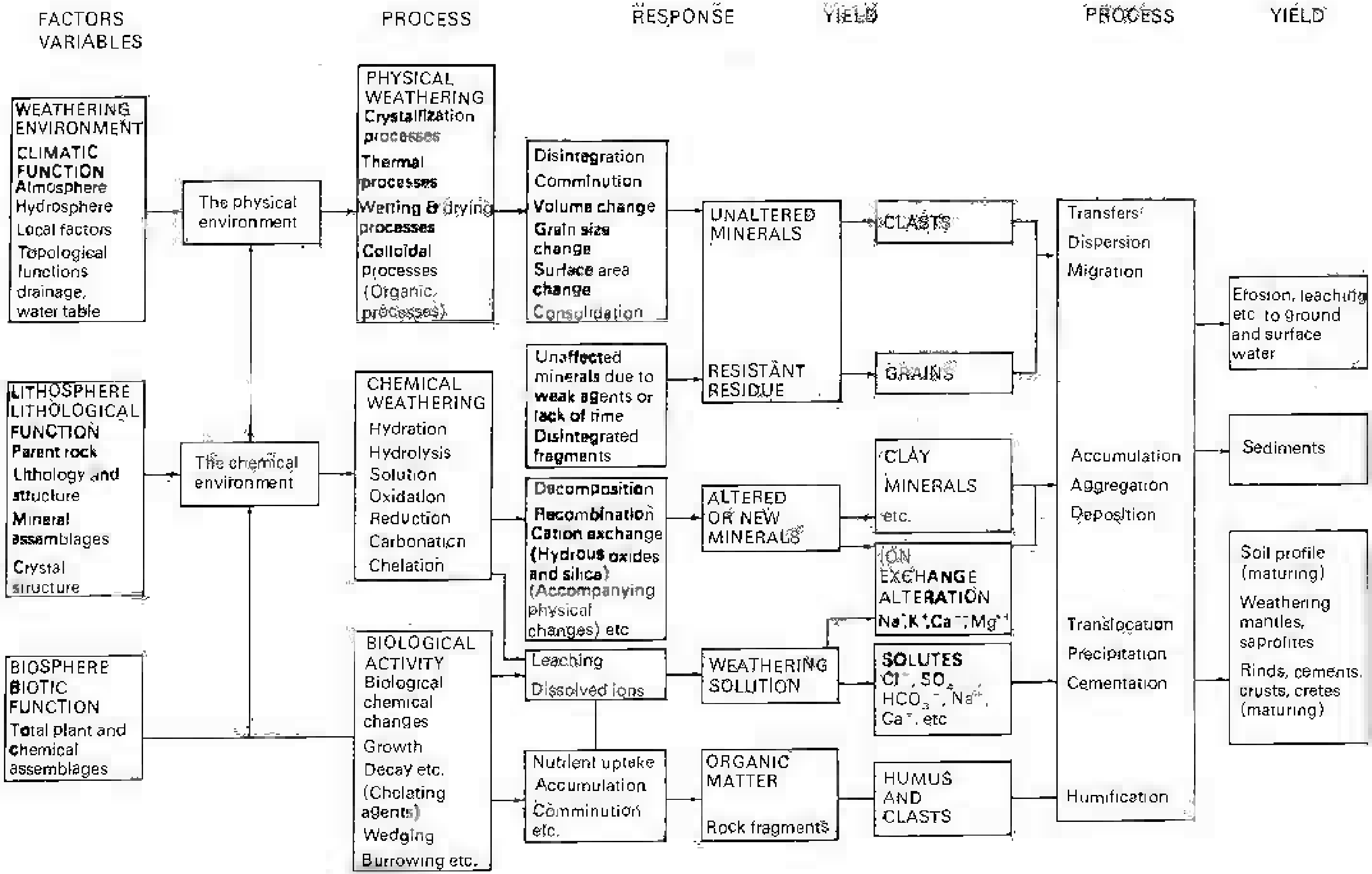
Table 2.14 Residually Weathered Construction Materials



M= ***Mica*** **A =** ***Amphibole*** **P =** ***Pyroxene*** **O =** ***Olivine***

Figure 2.1 Classification of Igneous Rocks (After Blyth and de Freitas 1984)

Figure 2.2 Tropical Weathering Processes and Products (After Brunsden 1979)



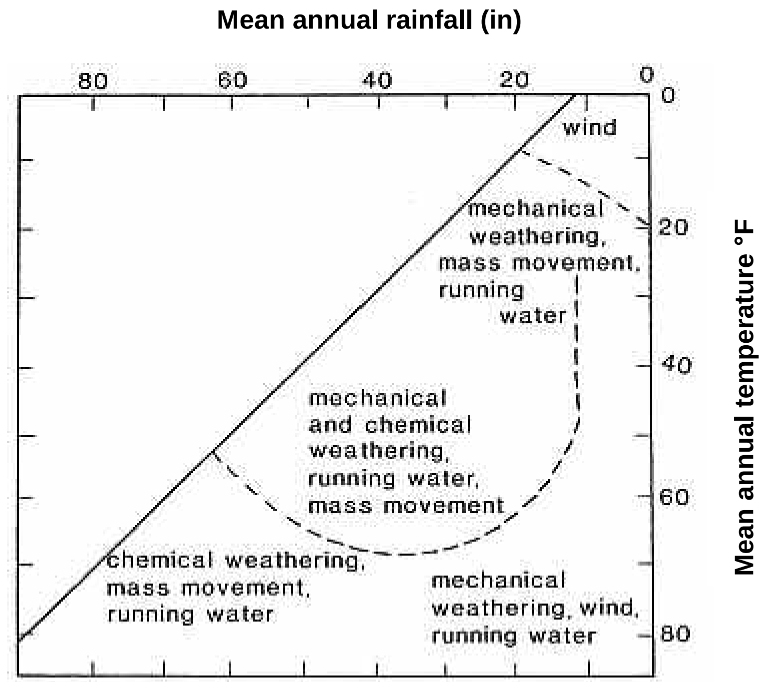


Figure 2.3 Climate and Dominant Weathering Processes, (After Leopold et al, 1964)

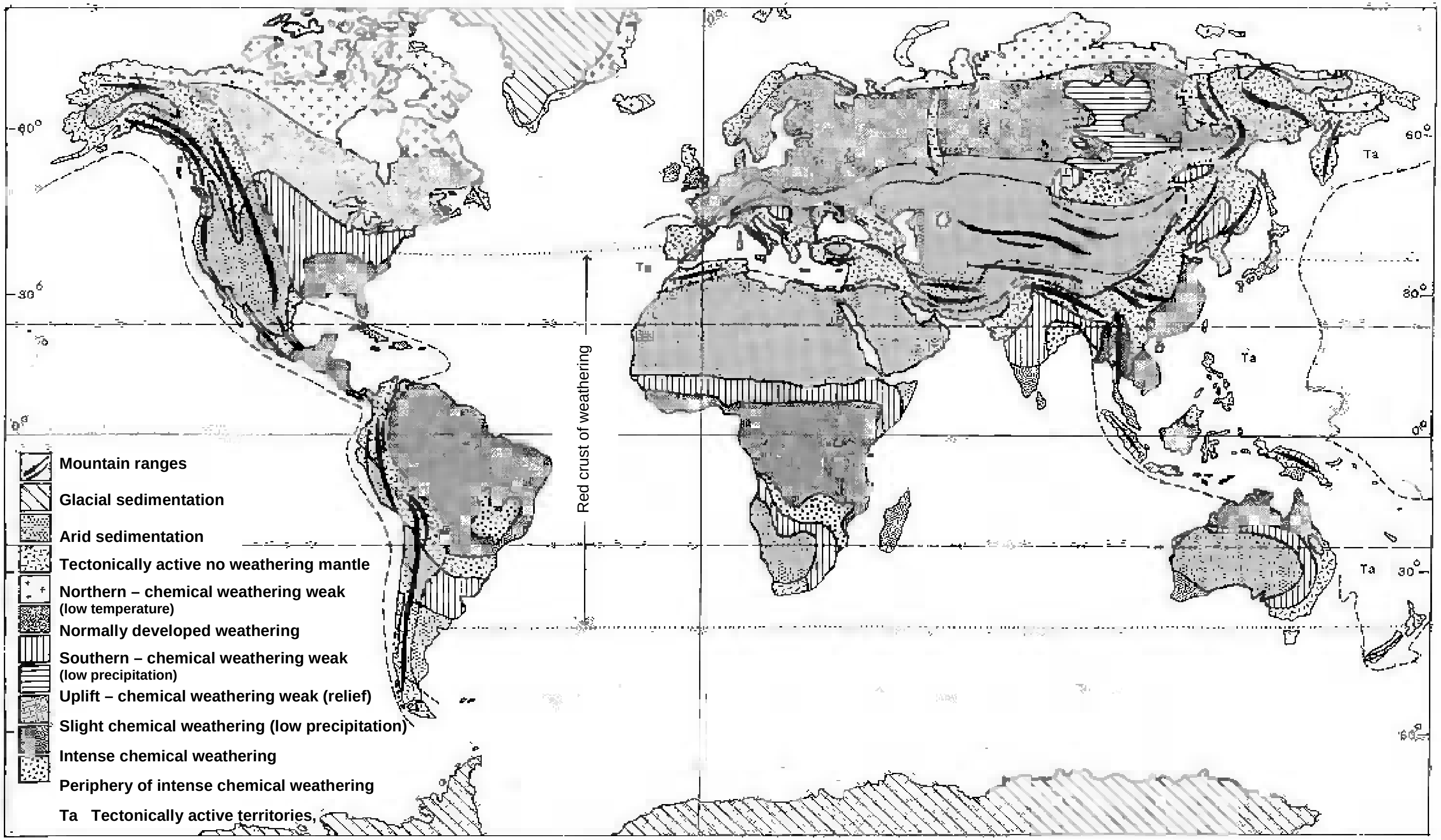


Figure 2.4 World-Wide Weathering Zones (After Strakhov 1964)

Dark Minerals	**Light minerals**	
Olivine		***Least Stable***
	Calcic plagioclase	↓
Pyroxene		
	Calcic-alkalic plagioclase	
Hornblende	**Alkalic-calcic plagioclase**	
	Alkalic plagioclase	
Biotite		
	Potash feldspar	
	Muscovite	
	Quartz	
		Most Stable

Figure 2.5 Bowens reaction series (After Bowen, 1928)

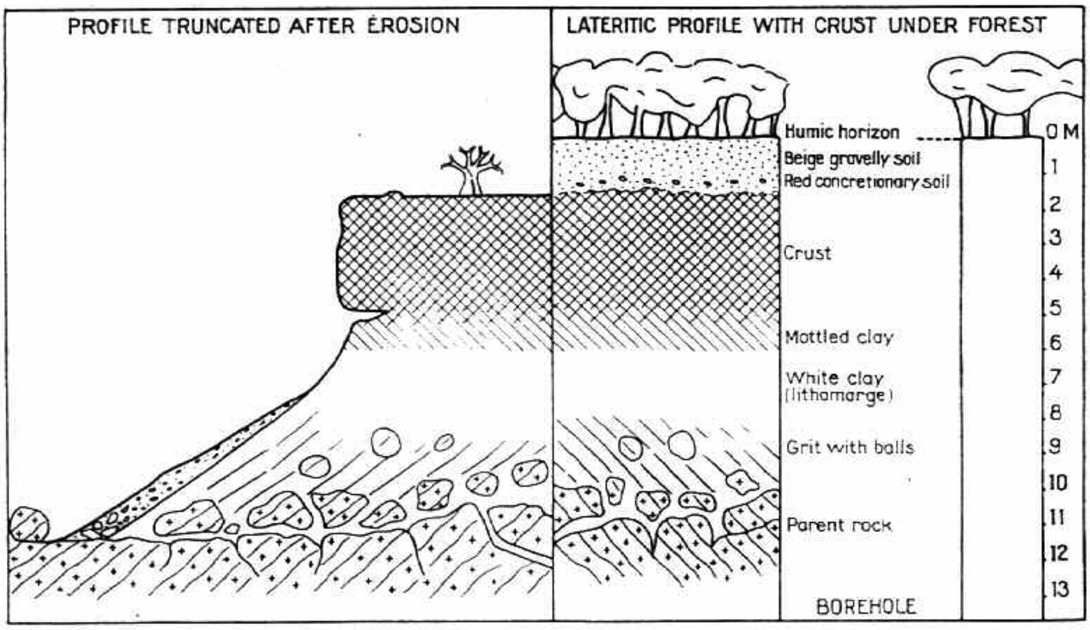
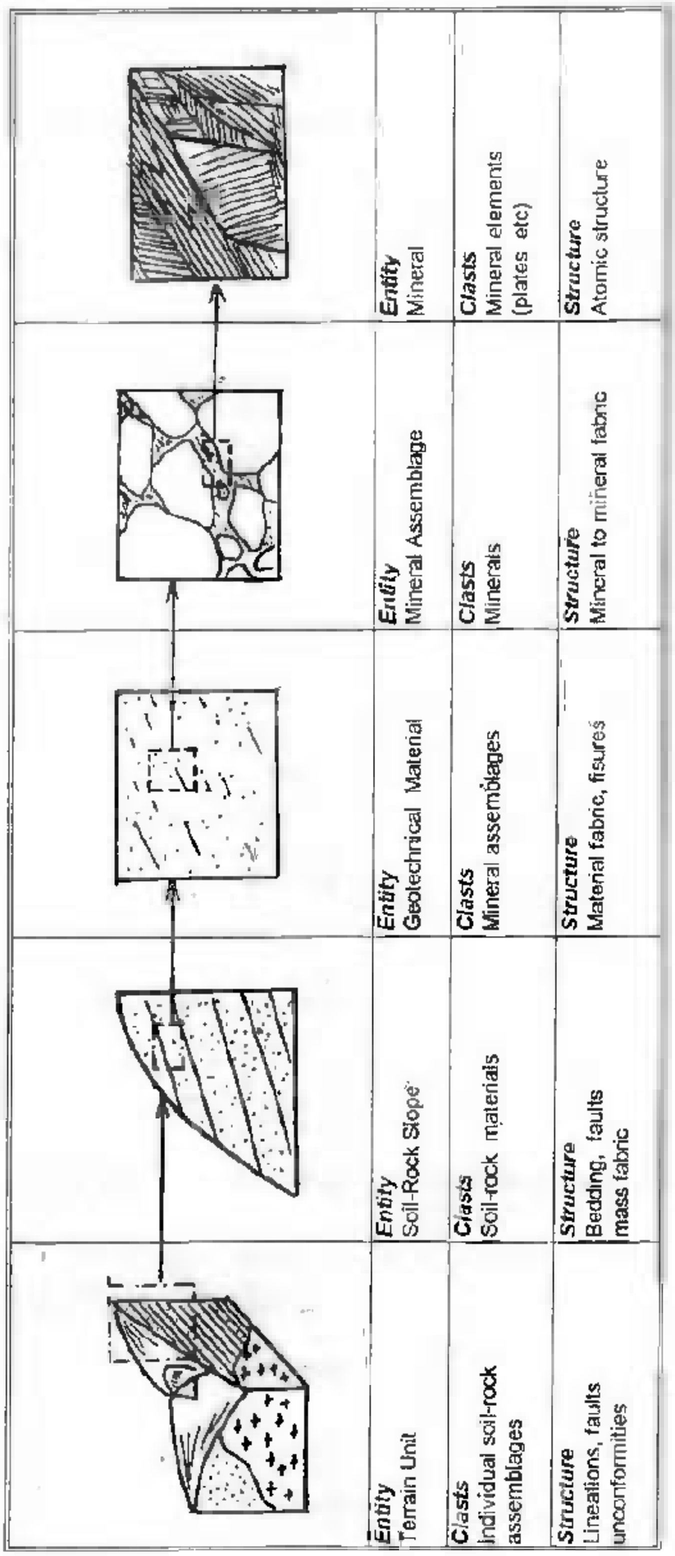


Figure 2.6 Typical units within tropically weathered soil rock profiles



Note: If mass is viewed as being an entity composed of a number of clasts in set structure, then the mass-material approach to characterisation may be employed at a number of geotechnical scales

Figure 2.7 Mass material scale (After Cook 1997)

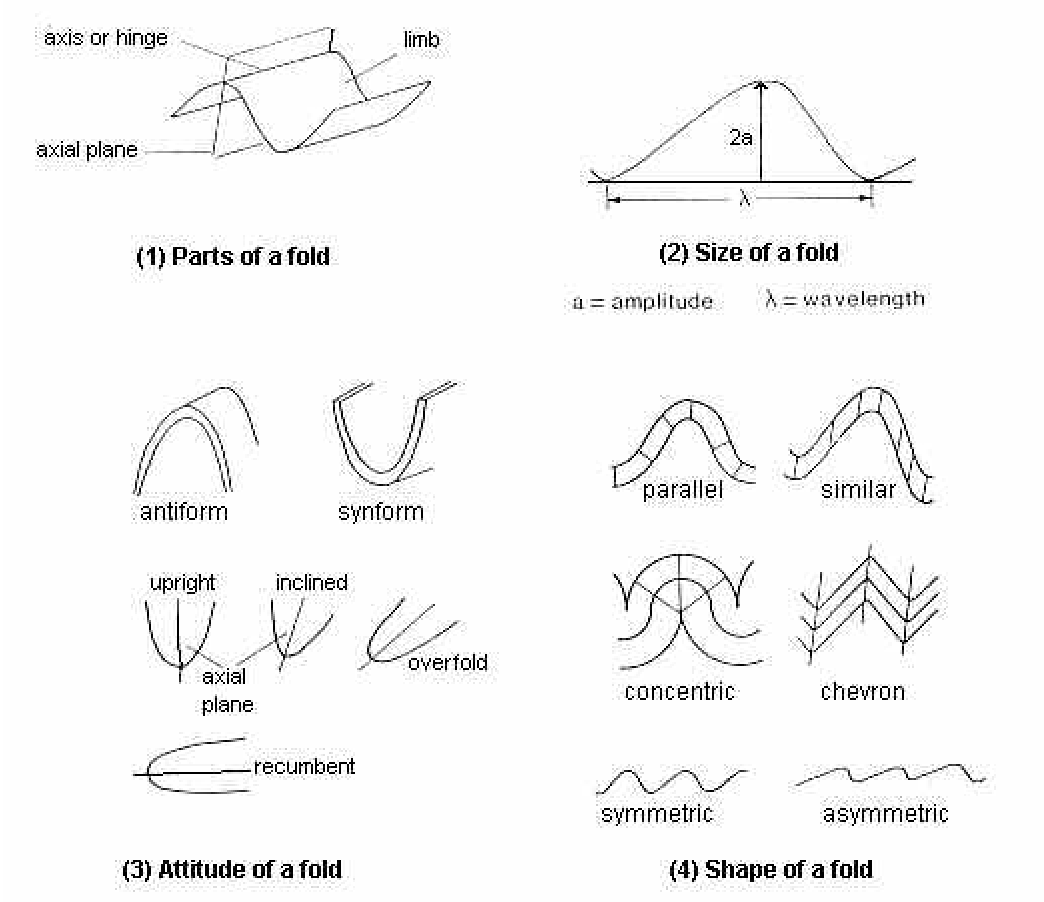


Figure 2.8 Definition of Fold Geometry

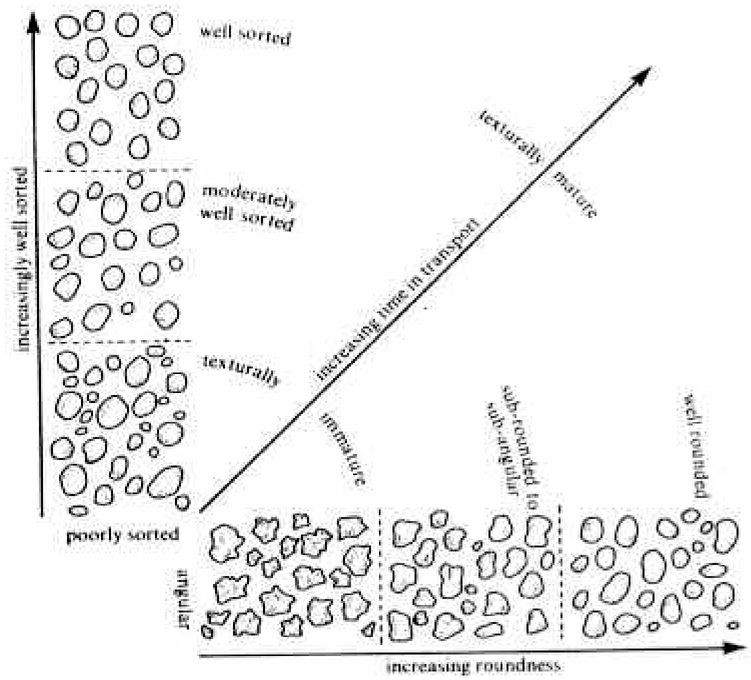


Figure 2.9 Aggregate Shape Definitions (After McNally, 1998)

3 THE ACQUISITION AND UTILISATION OF ROAD CONSTRUCTION MATERIALS INFORMATION

3.1 Introduction

Information to be collected on construction materials can be categorised by one of two basic types, project specific or general. Project specific information is that collected specifically for a current project. General information is existing information that may have been collected for another project but is also currently applicable. General information may be collected during what is commonly referred to as a "Desk-study" phase, whilst project specific, or new, information may be recovered during the project exploration and ground investigation phases.

This chapter describes the data-sets of information, both general and project specific that assist in the selection of natural road construction materials. A framework for information collection is presented that can be integrated with other information collection activities for general route alignment, earthwork and other geotechnical investigations.

3.2 General Objectives

The principal objective in acquiring information about road construction materials is to identify geotechnical materials that are capable of meeting the engineering, economic and environmental requirements of the project. Information relevant to this objective may be generally grouped as listed below:

- Source locations
- Geological environment
- Geotechnical character
- Volumes of material
- Project specifications
- Economic factors
- Environmental factors

The acquisition of materials information may be viewed as a process of gradually increasing the certainty by which decisions can be made regarding construction materials within the project area and how best to utilise them. Figure 3.1 illustrates this by defining four levels of geotechnical or engineering certainty in association with a clear distinction between existing "reserves" and technically unproven or uneconomic " resources". This distinction results from a number of factors including the cost of extraction and processing; source location and ownership and access, as well as geotechnical and environmental influences.

The need to “prove” sources highlights an important aspect of materials as natural deposits. Until a source is proved suitable as a construction material in all relevant aspects it cannot be designated a “reserve”; and until then it remains a “resource”. For example:

- A deposit may be designated a resource if it is geotechnically not suitable, but a change of specification or design could make it suitable with shift it to reserve status.

- Due to the economic implications of increased haul distance, a materials deposit may be a designated reserve (1A) on one road but may only be considered a resource (1B) in relation to another road nearby.

Information at level 1 is both directly relevant and specific to the project. In most cases, general background information, collected from other sources, will only be partially or indirectly relevant to project design and therefore must be applied with care. In other words, all information interpreted from maps, and most coming from other materials sources, will allow classification into levels 2, 3 or 4. Some existing project information may be up to level 2 or 3 but little or none will fall into 1. It is important to take account of this situation when deciding how best to use this general information and how much effort to put into its collection and interpretation.

The objective of acquiring information, as setout in Figure 1, is to be able to progress potential sources of road construction material to category 1A in as cost-effective a means as possible. As part of this process it is important to be able to define the level of certainty that has been achieved, as information is accumulated. It may be necessary, for example, for an engineer to make decisions on material sources before level 1A has been achieved. If the engineer is aware of this risk and is also aware of the implications for cost, design etc. of the actual level achieved, this can be taken into account in his decision making process.

3.3 Data Acquisition Programming

3.3.1 Information Groups

Information relevant to the selection and use of naturally occurring construction materials may be considered under a number of headings. Table 3.1 lists key information sets and summarises their importance with respect to construction materials. The importance of topics such as source location, size, geology, geotechnical character and material specification is obvious, however, there are some information sets that are frequently given less attention than is desirable.

Economic factors should include information on costs of winning materials including extraction, processing and transport to site. To be fully effective, data on any long term, or whole-life, cost implications of using particular materials should also be accessed; this may include relative maintenance and repair costs, and also long term cost implications of using inappropriately high grade resources. Information on the relative costs of stabilisation options for utilisation of lower quality materials would, for example, be required to balance judgement with the technical assessment.

The input of the prevailing climate on the in service performance of road materials and sub-grade can be significant with respect to their potential weathering and moisture susceptibility. It also impacts on their traffickability and placeability during construction.

The increasing importance given to environmental impact means that information on factors such as loss of habitat and pollution, are very relevant to materials investigations. This may be particularly applicable for some materials, for example the development and processing of technically hard quartz-rich rocks may incur risks due to production of silica dust. There is also the need to consider the environmental and economic implications of quarry or borrow pit restoration. These factors are discussed further in Chapter 4.

3.3.2 Programme Planning

A clear investigation design is normally desirable to achieve the delicate balance between budget, technical requirements and time constraints. Although the investigation programme will be a function of the overall project type and its constraints, it is recommended that the basis for the acquisition of relevant data should be a phased approach that takes full account of projects technical, economic and environmental requirements.

Materials investigation and design should not be limited to a list of possible activities but should be an argued statement of objectives complemented by a presentation of the means by which they can be achieved. A framework for staged materials investigations is presented in Figure 3.2 and forms the basis for the following sections. Figure 3.2 outlines the overall framework of assessment that may be linked with a decision-making procedure for identifying the requirements for the acquisition of fresh information.

Construction materials information needs to be accessed prior to the commencement of construction and ideally in advance of any tendering or construction planning phases. The acquisition of such information should therefore be commenced at an early stage in any road project.

3.4 Existing Information Acquisition

3.4.1 Scope and Objectives

The desk-study phase of a construction materials investigation is undertaken in order to assemble existing information not only on construction material resources but also information on alignment soils, sub-grades, and on the selection of appropriate specifications and construction methods. A crucial first step in the data acquisition process is the clear identification of the road project requirements. These may be summarised as follows:

- Type of project - new road, road rehabilitation.
- Stage of project - planning, design, construction
- Volume and types of material required
- Specification requirement for material
- Project constraints - economic, time, environmental, political.

Existing information relevant to the identification of construction materials may vary from general background data on geology, topography and climate to detailed direct data on engineering properties and in-service behaviour. Crucial steps in the existing data acquisition process are as follows:

- Identify sources of data
- Identify useful data in these sources
- Extract the relevant data
- Collate and analyse the data.
- Use data in the decision and design process

Tables 3.2 to 3.6 presents some queries that can be used to initiate the acquisition process, and following on from this, Table 3.7 outlines the basis for an assessment process. This assessment may lead to either a requirement for more desk study; a process of field data acquisition or a decision that enough information has been gathered already. Figure 3.3 details the sequence of desk-study data acquisition within the overall framework presented in Figure 3.2.

3.4.2 Maps as Information Sources

Table 3.8 outlines a range of map types that can contain useful construction materials information.

Topographic Maps. Commonly used world-wide, and vary in scale from 1:250,000 to 1: 5,000. Height above mean sea level is shown by contours. Information presented includes natural topographic features such as rivers, cliffs, rock outcrops and beaches. Some of these surface expressions are relevant to engineering materials, although usually these situations can only be recognised experienced geomorphologists. A simple example is a river terrace, that is identifiable by its position within a river valley, its shape in plan and the way it affects river flow, recognisable through the shape of its contour lines.

Geological Maps. Generally vary in scale between 1:5,000,000 and 1:500,000. The map is usually accompanied by a report describing the geological sequences shown on the map. Geological maps are an indispensable source of information for materials surveys, especially when searching for hard rock quarry sites.

Interpretation of geological maps is not made easy for an engineer by the fact that geological terminology can be complicated. It is necessary to understand the relationship between a rock name and the potential engineering characteristics of the rock. In other words, the name of a rock given on the map can convey information about its mineralogy and structure. This information in turn indicates the rock’s physical and chemical characteristics, from which its strength and durability can be preliminary assessed.

Agricultural Soil Maps. Commonly called soils maps vary in scale between 1:5,000,000 and 1:50,000, and are usually accompanied by a report describing the soil sequences shown on the map. Agricultural soil is defined (in simple terms) as that part of the weathered mantle so modified by chemical and physical processes that it can support plant life. The base of a soil profile includes the uppermost part of the weathered rock, but this means that most weathered rock gravels are excluded from agricultural soil descriptions. The depth of weathered material may go several metres below the base of the agricultural soil. However, soil maps can provide very useful information about the occurrence of pedogenic engineering construction materials (laterite, calcrete, silcrete) and are very useful indicators of sub-grade type, and earthfill sources.

Terrain classification or land system maps have been used quite extensively and may be discovered as a source of reference in the course of a literature search. Land system mapping is based on the premise that landscapes do not evolve randomly, but develop as integrated associations of soils, slopes and hydrological regimes. Within an association, ground characteristics have a limited range of variation and are predictable. The landscapes are called land systems. Land system maps are accompanied by an explanatory report, in a similar way to a geology map or soil map and can be used to help predict the:

- Complexity of landscape that will be encountered. This includes a subjective judgement on the amount of earthworks involved.
- Subgrade type
    - General hydrological conditions that can be expected
    - Likely occurrence of weathered materials or rock for road construction
    - Expected type of vegetation cover.

Countries in parts of Africa, the Far East and Australasia have the widest coverage of land system mapping. Land systems maps and reports give an immediate “feel” of a landscape by virtue of the drawings showing the slope relationships, and the easily-understood style of presentation of the terrain information.

3.4.3 Aerial photographs

Aerial photographs are usually taken vertically, in a strip of overlapping views (called a “run”). When two adjacent photographs from a run are viewed in a viewing stereoscope, the terrain can be seen as a three dimensional model. The normal use of aerial photographs is to make topographic maps; the three-dimensional view is necessary for the plotting of contour lines. However, aerial photographs provide a view of the ground in very fine detail, and when used in conjunction with geological and soils maps, form the best general planning tool for site investigation. Aerial photographs can be used as maps to help guide exploration teams. They show details such as buildings, footpath junctions and individual trees that will enable the party to locate its exact position.

Aerial photographs are taken at a wide range of scales, from about 1:80,000 up to 1:2,000, depending upon the mapping requirement for which they were taken. Scales between 1:30,000 - 1:50,000 are the most common. Information is extracted from the photographs by the process of air photo interpretation. An experienced interpreter can extract very useful information for construction materials planning from a photograph as shown in Table 3.9. Aerial photographs offer the most comprehensive single source of information about terrain, provided the observer learns to interpret them. They are best used in conjunction with topographic and geological maps to help build up a very complete engineering picture. Their disadvantage is that the scale of the picture is not consistent (unlike a map), so measurements taken off the photograph are only approximate.

3.4.4 Satellite imagery

Satellite imagery, as a digital image, is radioed form an orbiting satellite to a ground receiving station where it is processed and published either as a digital product on computer-compatible tape or CD (for display in a computer) or as a photograph. The images are in the public domain and are available for purchase.

The main sources of imagery are the US Landsat satellites and the French SPOT satellite. Together, these satellites have covered the whole land surface of earth. Photographic images are normally produced at scales between 1:1,000,000 and 1:250,000 (occasionally larger). Digital images can be enlarged on the computer display to any scale, although at scales larger than about 1:100,000 the individual picture elements (pixels) become visible and the image begins to break up. The pixels represent a square on the ground about 25-30m in size, so this is given as the nominal resolution of the image, although high-contrast objects smaller than this are often detectable (e.g., a gravel road running through grassy terrain). The images are in colour but are not stereoscopic.

The following types of features can normally be interpreted from satellite imagery:

- Boundaries between different types of vegetation (though not vegetation type itself).
- Geological structure and geological boundaries.
- The colour of soils and surface materials, in areas without vegetation cover.
- Drainage networks, watershed lines and river catchment areas
- Swamps and poorly drained low-lying ground.
- Terrain patterns, as described under Land System Mapping above. However, terrain pattern is largely hidden if actively-growing vegetation is abundant.
- Individual land forms if they are large or prominent or of high contrast.
- The dimensions in plan of large objects, e.g., the width of a major river.
- Climatic variations, i.e., change from drier to wetter climate, as reflected in vegetation characteristics.

Satellite images are interpreted in a very similar way to aerial photographs, requiring similar skills. This includes use at a basic level of a computerised image processing system for enhancing images. The added dimension of statistical processing of computer-based images and the availability of up to seven spectral bands for viewing can make digital image interpretation a specialised science. Help in this can be obtained from a remote sensing centre from where images are sold.

Satellite images provide general rather than specific information about terrain, on a regional basis. They are extremely useful when used in support of aerial photographs, whose qualities they largely complement.

3.4.5 Climatic Data

Climatic information relevant to the selection and use of construction materials is available in a number of forms and at a range of scales, from global seasonal patterns to local diurnal variations, Table 3.10.

Thornthwaite (1948) and Weinert (1964) developed indices to climatically subdivide regions, Table 3.11. The Weinart N value was developed in southern Africa as a predictive tool for assessing the effect of rock weathering. Information required to produce such indices include the following:

- annual and monthly rainfall
- average monthly air temperature
- relative humidity (wet bulb)
- mean wind speeds
- evaporation rates
- altitude

Climatic data can frequently be accessed through published statistics on rainfall that may also include information on variation of climate including factors such as frequency and intensity of rainfall and recurrence of rain-storms and evapotranspiration, Figure 3.4. The latter can be a crucial element in decisions on earthwork programming, including those on borrow pit development and standard of haul road provision.

3.4.6 Geotechnical and Construction Materials Data

A literature search should be an integral part of any desk-study. In addition to published bibliographies of relevant reports or technical papers, researchers now have the ability to access by electronic means libraries or other holders of information, such as geological surveys or national record offices.

Construction Records. Records for roads already built can be a valuable source of data, not only on the location of construction materials, but also on their excavation, processing, placement and subsequent performance. Potential problems with materials can also be identified. Construction records may be kept either by relevant government departments and local authorities or by road construction supervising organisations and contractors.

Materials Databases. There is an increasing awareness of the cost-effectiveness of assembling materials databases, for example, in Southern Africa (Gourley & Greening, 1999); in Indonesia (Woodbridge & Cook, 1998) and in Papua New Guinea (Bishop 1990) Databases such as these can supply early information on construction materials at little cost to a project. Resource maps showing the occurrence of construction materials are also increasingly being produced by geological survey departments in many parts of the world, and are usually incorporated in their materials database systems. The information gathered can also act as a guide for the planning of field investigations

Reports and Technical Papers: Relevant construction materials information may be contained within project reports and technical literature, including academic papers and conference reports. Reports or literature concerned with regional materials problems can be of particular importance. There may be specific reports or even databases of information specifically concerned with construction materials.

3.4.7 Desk Study Assesment

The primary outcome from the desk study is a collation of all relevant existing information on; potential materials sources, likely alignment soil conditions, the governing climatic and physical environment and the road project itself. A database of information, set up during the desk-study, should updated as increasing amounts of information are acquired during the whole investigation. It may also be the final repository of as-built-construction information. The database of desk-study ca be fed to the relevant project planning and road design personnel and would include:

- Location of existing or previously used quarries and pits in the project area
- Estimates of quality and quantities in existing sources
- Previously encountered problems with the above sources
- Likely areas for further exploration
- Range of sub-grade conditions along the proposed alignment
- Climatic details; including rainfall, rainfall intensities and evaporation
- Project materials required in terms of quality and quantity
- Project constraints; e.g. economic, contractual, environmental or time-related
- Proposed road design standards
- Likely soils and aggregate testing requirements

Table 3.12 summarises how materials information and data can be used to supplement various components and phases of the project.

It is possible that in some cases the amounts of information acquired during the desk study have rendered some expected or planned exploratory phases unnecessary. This can be the case when there are well established materials sources or where the project economic or contractual environment is such that some element of risk is deemed to be acceptable in utilising material sources that are below level 1A.

The desk-study data assessment should address the following issues:

- The amounts of construction material that have been identified, and to what levels of certainty (Figure 3.1)
- The amounts of material shortfall in each category: earthfill, sub-base, roadbase etc
- Recommendations as to what areas should be investigated for materials to cover any shortfall
- Planning of further field and laboratory material investigations to counteract the shortfalls
- Planning of investigations to fill-in any knowledge gaps with respect to sub-grade conditions
- Recommendations as any requirement for collection of further climatic data

It often possible to use existing spatial information to develops maps showing the relationship between the occurrence of existing sources, local landforms and bedrock geology and to use them as exploration tools.

3.5 Exploration Phase

3.5.1 Scope

The exploration phase is essentially a field operation backed by limited laboratory testing of representative samples. Field information is collected at potential construction material sites that will enable identification of areas for more detailed studies. The level of information collected needs to be enough to judge the relative merits of one site compared to another. This phase of work would also involve preliminary field identification of alignment soils. The exploration programme should, therefore, be set up to gather information on:

- Likely sub-surface extent and nature of deposits.
- As-dug properties of the deposit
- Excavation limitations
- Likely processing requirements

- Access requirements
- Suitability for various road-building applications.
- Alignment conditions and minimum subgrade strength

3.5.2 Procedures

The exploration phase would be done by a field team visiting and evaluating the existing, or potential, sites identified during the desk study phase. An ideal exploration team would include: a materials engineer/geologist; a road engineer/supervisor; and with support from local community representatives; who know location of exposures and ownership boundaries etc. The team should be provided with all relevant maps with sites marked on them and, where possible, with air photos.

The team should be prepared to undertake the following:

- Sketch map key geological features
- Sketch map key physical features; e.g. access tracks, source boundaries, rivers etc
- Dig shallow exploration pits or use hand augers
- Describe materials
- Enhance descriptions with appropriate field index tests, Table 3.13
- Take representative samples
- Undertake dynamic cone penetration tests (DCPs) over proposed alignment.

In some cases, particularly if different teams are visiting a large number of sites, it may be appropriate to consider the use of standard data collection forms to help eliminate operator error. Figures 3.5 to 3.7 are examples of construction materials pro-formas for the rapid collection of data from existing materials sources by relatively inexperienced teams.

Recovered samples should be tested for indicative soil and aggregate properties. In cases where geotechnical problems have been identified at the desk study phase, relevant specialist tests should be used to quantify the impact of these on the project.

3.5.3 Required Outcomes from an Exploration Phase

The principal outcome from the exploration phase of the work should be a clear identification of material resources, with a certainty level of 3, Figure 3.1. Reporting on the exploration phase should include:

- Details of resource locations
- Access information
- Estimates of available quantities
- Sketch maps of each source
- Evaluation of types of construction material available; aided by laboratory test results
- Assessment of plant required for excavation and processing
- Indicative assessment of problem, or non-standard materials
- Ownership details of existing or potential sources
- Assessment of costs
- In situ sub-grade conditions based on preliminary walk-over and test results

3.5.4 Exploration Data Assessment

The assessment should aim to identify appropriate standard and volume of resources for the road project with sites ranked in order of design, economic and technical suitability. This can be achieved by developing an outline material supply strategy, quality, quantity and costs of various materials along the road alignment. This may take the form of preliminary mass-haul diagrams.

It is possible that sufficient knowledge now exists, for example, with respect to common fill borrow areas. However, gaps in information should be identified and recommendations made with respect to further investigations. These recommendations should include further field and laboratory investigation procedures to be employed and the scale of work at each site.

3.6 Resource Investigation

3.6.1 Scope of Resource Investigation Phase

This phase of investigation is aimed at finalising the options available for selection of construction materials within a road project. The quality and quantities of available material types should be defined to at least certainty level 2, Figure 3.1. Sufficient information should be recovered finalise a material supply strategy to ensure that minimum costs are associated with supplying suitable quality materials at their required location. A materials supply strategy requires the following factors, Roughton International (2000):

- Cost per cubic metre of extraction and processing to comply with quality requirements, including cost of land, royalties, quarry/pit preparation, extraction costs, processing costs and reinstatement costs.
- Cost of haulage from source to site on road. Largely a function of distance, although the relative state of quarry access roads and socio-economic factors are also important.
- Resource size; for example, need to balance economics of developing a number of small cheap quarries over one larger one
- The relative quality of all resources being considered. Any variation in quality carries with it financial penalties in terms of design variation, and the associated maintenance and vehicle operating costs.
- Environmental impacts and associated costs. Maybe costs over and above normal re-instatement. On the other hand there may be cost benefits in terms of utilisation of quarry sites as waste disposal dumps.

Detailed investigations of alignment conditions should be undertaken at this stage, although these are most likely to be integrated with road design investigations.

3.6.2 Procedures for Resource Investigation Phase

Standard surface and sub-surface site investigation techniques are likely to be employed during this phase. These may be augmented by the use of quarry drills or borrow pit excavation plant at sites where extensions to existing reserves need to be proven, Table 3.14.

Methods of surface and sub-surface investigation must take into account the resources available (plant, labour, testing equipment) and the nature of the materials e.g. hard materials may not be hand-dug. The depth and nature of overburden may also influence investigation methods.

A comprehensive laboratory testing programme should be undertaken on recovered samples which should be sufficient to indicate both quality of the materials, likely variations in the deposit and the need for any improvement, e.g. chemical stabilisation.

3.6.3 Required Outputs from Resource Investigation Phase

Reporting of the resource investigation phase should be aimed at providing sufficient information to allow final decisions on the resources to be utilised within the framework of the material supply strategy. The following information is normally required:

- Borehole or drill hole logs
- Trial pits or trench logs
- Typical cross sections based on the above to indicate volumes available
- Quality assured laboratory test results

- Comment on the field and laboratory data, which should highlight not only the quality and quantities of the various materials but also the potential variability.
- Clear definition of problems associated with potentially difficult or non-standard materials
- Comment on suitability of stabilisation, if required.
- Likely socio-environmental impacts of resource development
- Available cost information

At this stage the three dimensional shape of deposits, the nature of the contact with surrounding materials, the thickness of overburden and underlying strata must be determined. Thickness and variation in thickness, together with removal and storage of overburden and waste are critical features in appraising a material resource. The structure of the hard rock resources needs to be to develop the approach to extraction i.e. blasting or mechanical excavation. The state of weathering or alteration also needs to be established, as this may define the use for materials; weathered materials may be designated for fill.

3.6.4 Investigation Phase Data Assessment

Information accumulated in this and previous phases should be evaluated with respect to the project in terms of material quality and quantity for the various road construction requirements. Gaps in knowledge should be clearly highlighted.

In some road projects, particularly in those where a significant excess of material of previously proven quality has been identified, investigation undertaken is sufficient, even though certainty levels may not be at the reserve level 1. However in other cases further investigation may be necessary, particularly so if:

- Large hard-rock quarries or borrow pits are to be used
- The project haslimited resource options.
- Haul diagrams indicate low margins for error
- Deposits are inherently variable
- The material is marginal in quality.

In these cases a detailed reserve or special investigation should be recommended with the details options and costs presented.

3.7 Reserve Definition

3.7.1 Scope

Reserve definition investigations should aim at proving quality and quantity of individual reserves to a certainty level 1.

3.7.2 Procedures

Procedures will be broadly in line with those employed for resource investigations. The principal difference will be that the scale of investigation must be such that material volumes and quality can be proven rather then inferred from representative sections. Any problem areas or materials within a quarry or borrow pit must be fully investigated and backed up by comprehensive laboratory testing. This would also include detailed laboratory investigation of properties required chemical stabilisation if required.

3.7.3 Required Outputs from Reserve Definition Phase

The principal outputs from a reserve investigation are detailed reports confirming construction material quantity and quality to certainty level 1. These should include reserve plans marking boundaries of proven quantities and cross-sections detailing the material boundaries, depths of overburden and limits of excavation. Clarification of any outstanding costing or technical detail relevant to the materials supply strategy and clear recommendations on methods of working, material processing material stabilisation should also form part of this reporting.

3.7.4 Final Information Assessment

Accumulated information up to and including the reserve definition phase needs to be presented in concise guidelines on the availability and utilisation of the identified construction materials reserves. Additional special investigations may be required, including:

- Special laboratory testing
- Quarry or pit production trials and associated testing
- Compaction trials
- Road trials.

Any further works should be clearly defined in terms of scope and cost.

3.8 Special Investigations

Special investigations may be required for a number of reasons. For example, materials having non-standard properties, will require a special approach

Pilot material production trials may be necessary to establish requirements for extraction plant and production of materials at the required quality and quantity. Materials extraction and production trials should be well planned and supervised with samples taken of the end products of all accepted and rejected materials. Further laboratory investigations should be made on samples collected from all stages of extraction and production pilot trials. This information can then be used to review and determine optimum extraction and production procedures for materials at the required standard or specification.

3.9 Construction Record Keeping

Records of material usage are an essential part of efficient construction materials management. Records concerning the actual use of materials during a project should be maintained during the course of a project and stored following completion. Ongoing maintenance and pavement evaluation records should also be added for futher reference.

Records that need to be kept, which can be used to construct "as built" haulage diagrams:

- Actual source of materials used to construct each section of road.
- Characteristics of each material used per section of road
- Actual costs of materials
- Estimate of residual volumes left at each utilised source.

The benefits of being able to access as built records are:

- Identification of resource deficiencies in terms of quantity for future use.
- Identification of any construction problems with particular materials; enables a crosscheck on, for example, assumptions regarding the relationship between in-situ, as-dug and service performance.
- Identification of in service performance deficiencies, which will allow amended processing requirements to be identified for future works.

Information Set	Description of Potential Information Sets	Key Implications
Location	Location of materials sources by co-ordinates, by road chainage or by representation on maps.	Identification of resources; distances for material haulage; mass haul calculations.
Quantities of. material	Amounts of potentially available material.	Reviewed in conjunction with volumes required and stockpiled quantities achievable and wastage. Requirements for further materials exploration or investigation
Geological nature	Classification: rock types, sand and gravel; duricrust etc. Morphology of the source. Amounts of weathering/overburden.	Material options identified for design. Potential problem identification. Overburden ratio calculations. Outline methods of extraction and processing.
Geotechnical character	Index or behaviour properties, either from tests on in situ source, processed material or in service road performance. In form of individual results, project reports or database files.	Material quality identified. Appropriate methods of processing or use. Also, the identification of possible problems associated with these activities.
Project specifications	Engineering requirements will generally be readily available for sub-grade and pavement materials and probably also for fill and filter media (though the last two may be less rigidly defined). Also any modification to the design standard or variation in standard practice that may be permitted.	Appropriate use of materials. Influence on design modifications required. Requirements for material processing or stabilisation. Possible impacts on construction plant selection and construction methodologies.
Economic factors	Costs of material processing; of haulage; and of any required modification. Cost limitations imposed by project budget.	Mass-haul cost calculations. Impact on designation of sources as "resources" or "reserves"
Climate and hydrology	Rainfall data – Mean annual rain fall, distribution and rain storm statistics. Local rain patterns. Evapotranspiration rates. Occurrence of watertable: depths and variation of depth with seasonal rainfall	A primary influence on performance of roads and selection of materials - related to climatic indices. Selection of design criteria, eg sub-grade design CBR. Depths of possible material extraction limited by watertable.
Environmental impact factors	Impacts on the environment: pollution - dust, noise etc; Health – water borne disease; Loss of productive land etc.	Methods of source extraction, material processing and rehabilitation. Limits imposed by groundwater levels.

Table 3.1 Key Information Sets

Basic Data Availability	Follow-on Queries
Is there an existing road ?	Is information available on the materials used to build this road ? Use Table 3.3
Has there been a local or regional materials survey ?	Where and in what form is this available (maps, paper files, computer files)? Use Table 3.6
Is the area topographically mapped ?	At what scales ?
Are there geological maps of the area ?	At what scales ?
Is there any published relevant technical literature	Where and in what form? - technical papers; consultant reports, Public Works guidelines
Is remote sensing information available ?	Air photos, and at what scale ?
Is there a regional/national road or public works laboratory or research institute	What data is held ? What are the capabilities for materials testing ?
Is there climatic data available ?	What form is data - general seasonal or detailed rainfall/evaporation figures ?

Table 3.2 General Background Data

Historical Data For Existing Road	Follow-on Action if Affirmative
Is there information on the overall road design?	Locate details of alignment and specifications used.
Is there information on road construction ?	Identify methods of material placement and compaction. Identify any problems.
Does the road have significant earthworks ?	Identify any fills over 3m - obtain details of side slopes, drainage, construction methods, etc.
Is there information on the sub-grade criteria	Obtain information on original moisture design criteria and subsequent equilibrium moisture contents - link with climatic data.
What is the pavement design ?	Obtain data on thicknesses of pavement layers and materials specifications.
Is there road performance or pavement evaluation data ?	Obtain data either on any deterioration problems due to materials or on possible over-design. Consult rehabilitation design documents
Where were the sources of road materials ?	Identify the source locations and if possible compute potential volumes remaining.

Table 3.3 Existing Road Data Queries

Current Project Queries	Follow-on Action if Affirmative
What is the road location ?	Identify how this road relates to the project in question ?
Can the geological/terrain environment be defined?	Compare the geology, geomorphology and geotechnical consequences of the relationship between the new project design and existing road
What is the pavement design ?	Obtain data on thicknesses of pavement layers and materials specifications.
Is there relevant local or regional road performance data ?	Obtain data either on any deterioration problems due to materials or on possible easing of material specifications in the light of performance.

Table 3.4 Current Project Queries

Data Queries For Other Civil Projects	Follow-on Action if Affirmative
What is the nature of the project(s)	Identify relevant construction/design details useful to current roads project.
Can the geological/terrain environment be defined?	Compare the geology, geomorphology and geotechnical consequences with the road project under consideration.
What natural materials were used construction	Identify nature of materials used and any relevant specifications
Where were the sources of road materials ?	Identify the source locations and if possible compute potential volumes that might be useable on the new project. Alternatively identify similar geological/terrain units.

Table 3.5 Data Queries For Other Civil Engineering Projects

Identified Resources Queries	Follow-on Action
What is the nature of the resource?	General classification in terms of hard rock quarries; sand/gravel pits; and borrow pits.
What is the location of the resource?	Define in relation to the road project. Preliminary haul distance calculations.
What is the expected quality and volume of the material available.?	Define the quality and quantities in relation to those required for the current project ?
What excavation methods were used?	Identify whether these methods are still applicable - could they be improved? What were the environmental impacts?
Were the materials processed?	Identify the processing methods. Is the processing plant still available.
What are the costs of the materials ?	Identify unit costs of materials including processing if necessary. Input data to mass-haul diagrams
Is there scope for further development of the identified sources ?	Identify geological and physical boundary information. Clarify ownership.
Is there scope for identifying further similar source areas ?	Identify similar geological and terrain combinations within the project area.

Table 3.6 Materials Source Data

General Topic	Queries
Source Location	Are the physical and/or geological boundaries of the source(s) identified ? Is there sufficient groundwater information ? Are haul routes available? Has land ownership been clarified ? Have adverse environmental impact factors been identified?
Material	Has the quality of the proposed material(s) been established (level 1A; Fig 3.1) ? Is the volume of material sufficient? Have any inherent problems with the proposed source material(s) been identified ? Have the processing requirements been identified and costed ?

Note: If the query answer is "no" then further data acquisition is required.

Table 3.7 Basic Materials Information Assessment

Map Type	Relevant Materials Information
Topographic	Important for use in locating previously identified sources. Most topographic maps are of limited use in providing direct information about materials because contour lines more than 10m apart do not represent the subtleties of ground shape very well.
Geological	Geological maps are an indispensable source of information for materials surveys, especially with respect tor hard rock quarry sites. Sources of both direct and background geological information. Useful for providing hints as to potential material and alignment problems
Agricultural Soils	Soils maps can provide very useful information about the occurrence of pedogenic construction materials (laterite, calcrete, silcrete). They may have particular relevance for identifying borrow areas. Soils maps are very useful indicators of subgrade type, especially with regard to subgrade drainage.
Terrain Classification	Terrain maps can useful for preliminary assessment of fill requirements and the general hydrological conditions expected. Also used for assessing the likely occurrence of weathered materials or rock for road construction.
Specialist Maps	Specialist maps can include information on useful topics such as hydrology; climate (e.g. rainfall contours); materials locations; and geotechnical hazard.

Table 3.8 Application of Common Map types

Features Useful for Construction Material Assessment	Relative Ability of Aerial Photographs to Depict the Feature
Ground shape and relief	3
Rock type	1
Geological boundaries	3
Rock outcrop	3
Geological structure	2
Rock strength	2
Depth of weathering or overburden	2
Soil/overburden type	1
General soil geotechnical character	1
Soil erodability	3
Depth to water table	1
Springs, surface water, areas of poor drainage	3

Table 3.9 Ability of Aerial Photographs to Indicate Useful Features

Scale	Description	Availability
Global	General climatic patterns and divisions, e.g. tropical,:sub-tropical, temperate etc.	Atlas
Regional	Regional climatic divisions such as: permanently dry (aridic); seasonally dry (ustic); seasonally wet (udic); permanently wet (perhumid) etc. Divisions may be made on the on the basis of climatic indices such as Weinert "N" value or the Thornthwaite moisture index (TMI).	Met Office, Agriculture etc
Local or site conditions	Rainfall and evapotranspiration patterns that may be governed by local topography and situation (eg coastal, inland mountainous etc)	Met Office, Local Stations

Table 3.10 Scales of Climatic Information

Climatic Division	Thornthwaite Moisture Index: (100s-60d)/n	Approximate Equivalent 'N' Value
Prehumid	>100	
Humid	80 to 100 60 to 80 40 to 60 20 to 40	
Moist Sub-humid	0 to 20	<1
Dry Sub-humid	-20 to 0	1-2
Semi Arid	-40 to –20	2-4
Arid	-60 to –40	>4

Key: s = water surplus
d = water deficit
n = water needed

Table 3.11 The Thornthwaite Moisture Index

Application	Description
Resource planning	Planning the utilisation of the available materials in the most cost-effective manner that is compliant with the engineering requirements of the project. Ensuring, for example, that high quality aggregate resources are not wasted as low-specification materials and that mass-haul diagrams are programmed effectively.
Processing requirements	Identifying what processing is required for various materials and ensuring that appropriate processing plant is selected (See Chapter 4).
Stabilisation requirements	Identifying requirements for stabilisation- e.g. mechanical, chemical, bitumen etc. Drawing up testing and trial programmes that establish detailed design and construction procedures.
Materials design and control	Drawing up guidelines for materials testing programmes that ensures that consistent quality material is brought to site and is placed correctly. The exact nature of the tests to be used will be a function of information acquired at an early stage in the materials investigation process. (Chapter 6).
Design modifications	Material information may lead to design modifications; for example, in the fill-slope angles or in the detailed pavement design.
Construction techniques	Material information may be used to decide on construction methods; for example some fabric-sensitive fill materials will require special handling and placement techniques.

Table 3.12 Summary of Materials Information Utilisation

Field Test	Description
Fines Content	Relative percentages of silt/clay. Diltancy test Jar settlement test (Houben and Guillaud 1990)
Schmidt Hammer	Use of Schmidt hammer on solid rock exposure or large boulder can be correlated to estimated compressive strength. (Deere and Miller 1966)
Hand Sample Index Strength	Use of small geological type hammer on hand or core sample Very weak: easily broken in hand Weak: broken by leaning on sample with hammer Mod. weak: broken in hand by hitting with hammer Mod. strong: broken against solid object with hammer Strong: difficult to break against solid object with hammer Very strong: requires many blows of hammer to break sample Extremely strong: sample can only be chipped with hammer
Field Durability	Immerse samples in still water for 30 minutes and observe behaviour: no effect noticeable drop in strength slowly breaks into pieces under light finger pressure slowly crumbles to small blocks under light finger rapidly breaks into pieces under light finger pressure rapidly crumbles to small blocks under light finger pressure rapidly crumbles to small blocks disintegrates to sediment
Aggregate Pliers Test	(Netterberg 1978) Take 100-200 pieces of air dry material in the 12.7 to 19.1mm range. The operator then attempts to break the pieces between finger and thumb. The remaining pieces are then tested by trying to break them with a pair of 180-mm pliers. The maximum strength should be applied in both experiments. The percentage unbroken by the pliers is termed the Aggregate Pliers Value and is broadly comparable to 10% Fines value of over 100 kN.
Feldspar Condition	Used to assess condition of feldspar-rich hard rocks Fresh - hard: Cannot be grooved with a pin/knife Moderately decomposed – gritty: Cut by knife, grooved by pin. Highly decomposed – powdery: Crushable to silt by finger pressure Completely decomposed – soft: Moulded very easily by finger pressure
Field Plasticity	(Houben and Guillaud 1990) Prepare a ball 2 or 3 cm in diameter. Moisten so that it can be modelled without being sticky. Roll to a 3mm thread adding water if necessary. At 3mm the material should start to break, then remould into a ball and carry out the following: - Ball is hard to crush – does not crack/crumble = high clay content. - Tends to crack/crumble = low clay content - Impossible to make a ball = high sand or silt content, very little clay - The ball has a soft or spongy fell = organic soil

Table 3.13 Rapid Field Index Tests

Field Activity	Description
Surface Mapping	Both geological and geomorphological mapping and may include the recovery of hydrological, vegetational and climatic data and the mapping of earthwork exposures. Surface data gathering can comprise a formalised materials inventory approach
Exposure and Sample Description	The systematic recording of material characteristics including the use of in situ behavioural testing. All samples and exposures should be logged using these guidelines.
Boreholes	These may be sunk by a number of percussion, or rotary methods. The techniques employed should be chosen to take into account the type and condition of material involved. In some locations options may be restricted by economic or access constraints
Trial Pits	May be either hand or machine dug. Particularly cost effective in the examination and logging of material fabric and the delineation of mass structure. Caution should be exercised in geotechnical interpretation of duricrust masses by test pitting alone where weaker material may underlie stronger. Very useful for obtaining bulk samples.
Augering	Can range from hand augering to machine driven hollow stem augering with undisturbed sampling.
Probing	Relatively inexpensive procedure that can be effective in delineating boundaries to soft or weak materials and in the recording of general in situ material condition. Quarry drills may be used in conjunction cored holes for correlation.
Hydrology Data	Installation of piezometers to measure watertable depths.
Climatic Data	Possible set-up of climate apparatus - thermograph, hygrograph an anemometer.
DCP	Utilised for calculation of CBR values for in situ sub-grade
Engineering Geophysics	Seismic refraction the most generally used procedure. Best utilised to interpolate or extrapolate in situ conditions in conjunction with boreholes. Caution required in environments where stronger material may overlay weaker.

Table 3.14 Standard Field Techniques for Materials Investigations

	Identified Sources			
	1 Proven Deposit	2 Indicated Deposit	3 Inferred Deposit	4 Potential Source Areas
A Reserves	1A	2A	3A	
B Resources	1B	2B	3B	

NOTES

B to A — Shift from resources to reserves is based on specification requirements and economic/technical constraints

4 to 1: — Gradation increases with geological and engineering certainty

Proven.
Level 1: — Tonnage and quality calculated from detailed investigation and sampling techniques in conjunction with a well defined geological appreciation of the deposit and a clear understanding of the materials' relation to the project specification requirements.

Indicated.
Level 2: — Tonnage and compliance with likely specification requirements computed using data from widely or irregularly spaced sampling points, from limited production data and an appreciation of the geology of the deposit. The deposit will not be outlined completely nor the grade (quality) established throughout.

Inferred.
Level 3: — Quantitative estimates are based largely on a broad knowledge of the geological character of similar deposits or the deposit itself from which there only a few, if any, samples or measurements. Suitability of the deposit is largely estimated from a combination of previous experience and engineering judgement.

Potential
Source Areas
Level 4: — Areas where potential sources of construction material are likely to be found based on general geological information only; e.g. from maps, air photos, satellite imagery, local experience.

Figure 3.1 Identification of Material Deposits Status, Modified McElvey Diagram

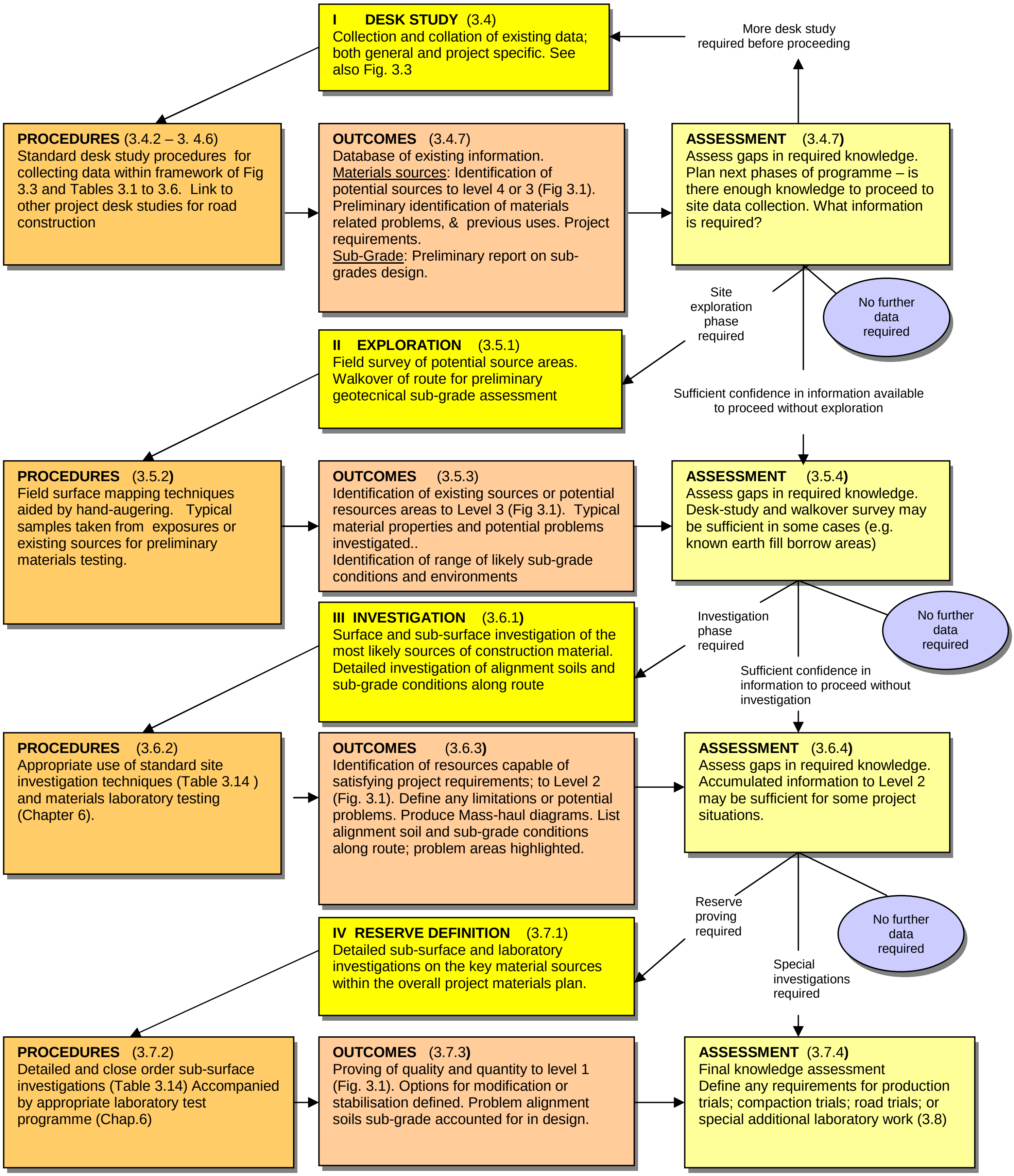


Figure 3.2 Construction Materials Investigation Programme Framework

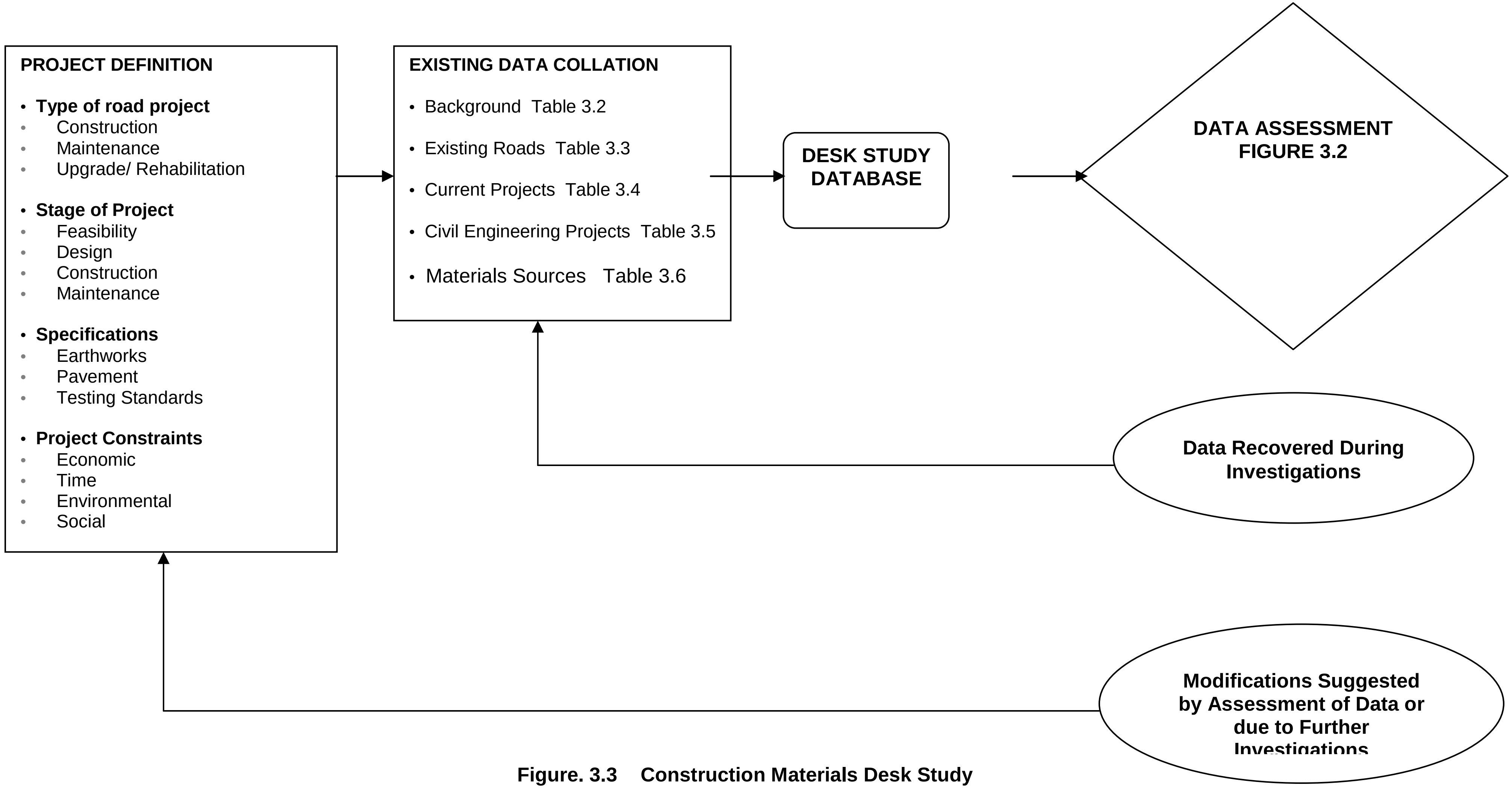


Figure. 3.3 Construction Materials Desk Study

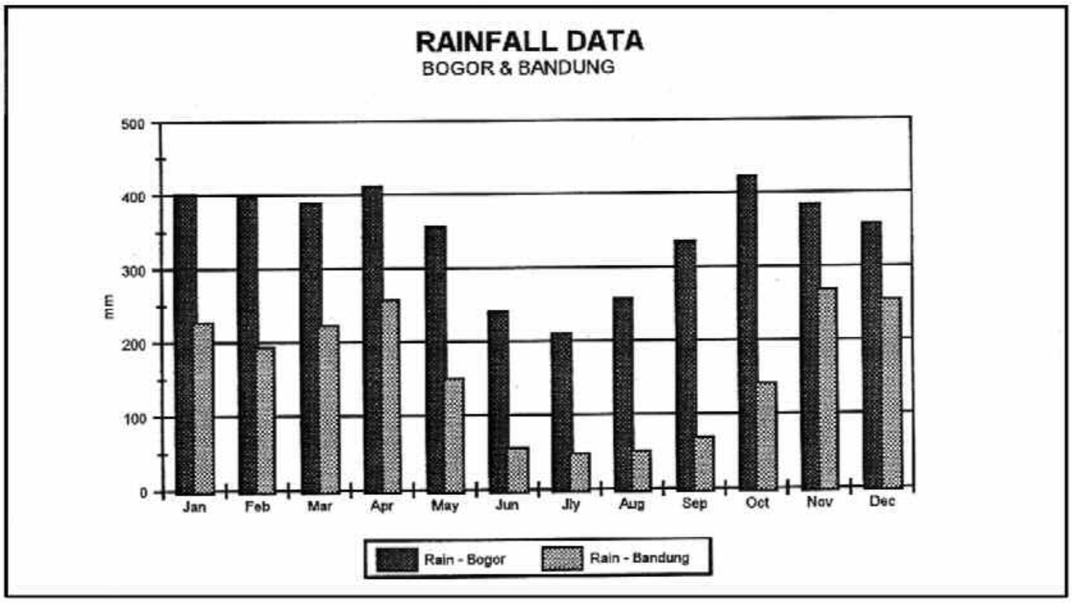


Figure. 3.4 Typical climatic data for comparison purposes

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CONSTRUCTION MATERIALS INVENTORY: FIELD FORM

LOCATION

1 Location No

2 Province

3 District

4 Subdistrict

5 Desa

6 Village

7 Rute

8 Road from

10 Road no

9 Road to

11 Road Status

1. National
2. Provincial
3. District
4. Village

12 Kms from (8)

13 Road Condition

1. Asphalt
2. Gravel
3. Earth

14 Access road distance

15 Access road condition

1. Asphalt
2. Gravel
3. Earth

16 Coordinates E N

17 Ownership

1. Private
2. Concession

18 Operating status

1. Fully operational
2. Partially operational
3. Not operational
4. Under development

19 Operating method

1. Manual
2. Mobile plant

20 Utilities available

1. No utilities
2. Electric power
3. Water
4. Water & electricity

ENVIRONMENT

1 Geomorphology

1. River
2. Coastal
3. Valley side
4. Hill
5. Mountain
6. Plain

2 Land Use

0. None
1. Housing
2. Industrial
3. Plantation
4. Agriculture
5. Forestry
6. Mining
7. Other

3 Environmental impact

0. None
1. Village
2. Industrial
3. Plantation
4. Agriculture
5. Forestry
6. River
7. Tourism

4 Quarry Restraints

0. None
1. Housing/buildings
2. Flooding
3. Environmental impact
4. Physical

Figure 3.5 Construction materials pro-formas (Location and Environment 1/3)

QUARRY MATERIAL DATA

1 Physical State
1.
2.
1. Loose
2. Massive
3. Compact

2 Material Type
1.
2.
1. Sand & Gravel
2. Rock
3. Conglomerate/Breccia
4. Soil

3 Material definition
1.
2.
1. Alluvium, Terrace deposit
2. Andesite, Granite, Diorite Metamorph
3. Conglomerate, Breccia
4. Limestone, sandstone, shale

Reserve estimate, m3
1.
2.
1. <10,000
2. 10,000-25,000
3. 25,000-50,000
4. 50,000-100,000
5. 100,000-250,000
6. 250,000-500,000
7. 500,000-1,000,000
8. 1,000,000-2,500,000
9. >2,500,000

5 Field estimate of size percentages (for alluvium/terrace)

(>200mm) Boulder 1. 2.

(76-200mm) Cobble 1. 2.

(2-76mm) Gravel 1. 2.

(0.06-2mm) Sand 1. 2.

(<0.06mm) Fines 1. 2.

Overburden type
1.
2.
1. None
2. Loose
3. Mixed loose & hard
4. Compact

7 Overburden Thickness
1.
2.
1. <1m
2. 1 - 2m
3. 2 - 4m
4. >4m

8 Material Use
1.
2.
1. Surface dressing
2. Asphalt aggregate
3. Roadbase
4. Subbase
5. Concrete aggregate
6. Fill
7. Housing & others

QUARRY PRODUCT DATA

1 Type
1.
2.
3.
4.

2 Size, Max & min (mm)
1.
2.
3.
4.

3 Production rate (m3/minggu)
1.
2.
3.
4.

4 Price (Rp/m3)
1.
2.
3.
4.

Figure 3.6 Construction materials pro-formas (Quarry Material Data and Product Data 2/3)

SAMPLE DESCRIPTION AND LABORATORY TESTS REQUIRED

1 Material Type	1.	2 Sample derivation	1.
	2.		2.
	1. Loose 2. Massive 3. Compact		1. Test pit 2. Borehole 3. Surface 4. Side section 5. [illegible]
3 Sample Size (kg)	1.		
	2.		

4 Laboratory Tests required

Water Absorption		Elongation	
Specific Gravity (bulk)		Flakiness	
Specific Gravity (ssd)		Stripping	
Specific Gravity (app)		Sulphate Soundness	
Los Angeles Abrasion		Atterberg Limits	
10% Fines (dry)		Compaction	
10% Fines (soaked)		Grading	
Impact Value (dry)		CBR	
Impact Value (soaked)		Swell	
Polished Stone Value		Organic Content	

REMARKS

1 Photo Nos		Notes
2 S[illegible]		
3 Survey Date		

Figure 3.7 Construction materials pro-formas (Sample description and laboratory tests required, Remarks 3/3)

4 EXTRACTION AND PROCESSING

4.1 Introduction

Removing naturally occurring material from the ground and transforming it a product useful for road construction involves processes of extraction and, possibly, of modification through processing operations. The extraction and processing of road construction materials can be large components of cost in the overall project budget. The efficient integration of extraction and processing techniques is an essential pre-requisite to maximising the benefit from efficient use of natural non-renewable resources.

This chapter discusses the key issues concerning the extraction and processing of materials for road construction, with particular reference to the conditions met in the tropics. For the purposes of these guidelines, extraction and processing operations are discussed in terms of their general material nature as defined in Chapter 2:

- Hard rock
- Weak rock
- Sands and gravels
- Duricrusts
- Residual soils

Factors discussed range from aspects of excavation and processing to the management of environmental matters, disposal of waste and reclamation of exploited land.

4.2 Material Extraction

4.2.1 General

There are five main types of material extractive operation:

- quarrying: extraction in drilled and blasted material, e.g. hard rock
- borrow pitting: extraction of unconsolidated material, e.g. gravels and weak rocks
- cut to fill operations along a road alignment
- mining: underground material extraction, either by shaft or adit
- dredging: extraction of unconsolidated material from under water

Materials for road construction are very rarely derived from mining operations, except as a by-product such as tailing etc. Recovery of marine aggregates from dredging operations is becoming more common in the developed world where local shortages of aggregates exist, but is not so common in developing countries because of the high cost. Recovery operations for road construction materials, therefore, normally come under the general heading of quarrying pitting and cut to fill.

Some materials, particularly common fill or capping layer fill, may be won from cuttings and excavation along the route. In deeper rock cuts it is sometimes cost-effective to utilise mobile plant to process good quality rock. Although this chapter deals primarily with materials won from off-line sources the principles regarding extraction method and processing hold true for alignment won materials. The main additional consideration is that the extractive operation must be concerned primarily with forming a stable cut-slope.

The use of the terms “quarry” and “pit” is not fixed and may be used interchangeably although they are generally used to distinguish between rock and soil-type operations. In quarries the ratio of material extracted to area of land effected is generally much larger than for pits. Table 4.1 summarises material types with respect to extractive operations.

Methods of excavation and equipment selection are influenced by the physical environment and material type in conjunction with the technical and economic framework of a project. Earlier site investigations will have determined the nature and thickness of overburden, the properties of the material and the hydrological and environmental conditions requiring consideration.

4.2.2 Hard Rock Quarrying

Crushed stone for roadbase or surfacing is normally won by drill-and-blast quarrying operations. Labour intensive procedures can be employed in low-cost projects.

Within tropical and sub-tropical areas it is likely that the exposed rock directly beneath the overburden (known as “top rock”) will be weathered to some extent. This can often preclude its use if high quality stone is needed, although it may be still be selected for a less demanding purpose. Weaker layers such as these would normally be worked separately to avoid contamination of the better quality material below. The weaker material may be soft enough to be separately excavated by excavator, or ripper, as discussed in later sections. Drainage water often accumulates at the interface between the overburden and the exposed rock. Adequate provision should be made where necessary to intercept and guide the flow of water.

The important features in determining the approach to exploiting hard rock and its surface weathered materials will depend on the underlying structural features of the deposit. Structural and other features that require consideration include :-

- Rock type
- Regional tectonics of the area
- Joint sets and discontinuities
- Lithological contacts between layers
- Bedding/banding
- Foliation surfaces and infillings
- Shear or fracture
- Water conditions.

In the development of most rock quarrying operations it is advisable to establish a benching system for material separation, ease of control and stability of quarry sides, and this is documented as part of the excavation plan, Figure 4.1.

A working sequence should delineate the boundaries for the excavation process and the programme for working within it. The working detail determines when the various design features to address the geotechnical problems are introduced. This includes access and removal of useable products and waste, de-watering the excavation, the sequence for drilling and blasting and other excavation techniques, and when and where they are to be applied in the workings. Stabilising the slopes and the temporary safe storage of waste materials, should also be an integral part of the working sequence.

It is generally advisable to establish long (> 50m) and low (<15m) benches. High, narrow faces restrict quarrying options and are potentially dangerous. It is also important to ensure that quarry floors are as level as possible, to minimise mobile plant wear and, for site safety. Rock mass structure, including bedding, jointing or faulting, should be taken into account when developing an overall strategic development plan for a hard rock quarry. Both the efficiency of extraction methods and the safety of quarry benches are likely to be influenced by these structural elements. Figures 4.2 to 4.4 illustrate the influence of structure in typical hard-rock situations.

Knowledge of the rock material properties is also necessary to determine the optimum drilling and blasting patterns. Every rock formation requires its own blasting method but fragmentation is essentially determined by a combination of two factors:

- explosive energy creating new fracture surfaces (also termed shattering energy)
- breaking of rock along existing planes of weakness (also termed heave energy)

The main considerations in quarry blasting are to generate minimum oversize with good fragmentation whilst satisfying environmental and safety restrictions on dust, vibration and "fly" rock. This is largely achieved by limiting the amount of explosive fired at any one instant and by carefully distributing the charge within the blastholes.

The effectiveness of the fragmentation is controlled by the following factors:

- Shot hole depth:
- Shot hole diameter:
- Burden:
- Shot hole spacing:
- Weight of explosive charge
- Type of explosive
- Detonation and blasting sequence

Some of the terms frequently used are explained in Figure 4.5. A suitably qualified and experienced quarry master is required for drill and blast operations. There may need to experiment in the initial stages of quarrying to obtain the optimum yield.

Two drilling methods are used for drilling shot holes:

- Down-the-hole, rotary percussive for medium to hard rock
- Top hammer, rotary percussive holes for hard rock

Top hammer drill rigs, known as ”drifter” rigs, are lighter and more manoeuvrable than down-the-hole rigs and also have the added use for exploration, in the right conditions. Down-the-hole rigs are more common in large quarries where speed and accuracy of drilling is at a premium. Modern drilling machines employ hydraulic rather than pneumatic systems because of the greater penetration rate and lower power costs of the former.

Explosive materials are available in two main groups: high explosive, such as dynamite and gelignite; and blasting agents, such as ANFO (94% ammonium nitrate and 6% fuel oil). ANFO is cheap, energetic and safe to transport, store and load. Increased shattering power is achieved by the addition of aluminium dust (ALANFO). High explosives are dangerous and expensive and not always easy to obtain or control however they are powerful and can be used in wet conditions. Blasting agents are cheap, although less powerful and are generally preferred in aggregate quarries. Table 4.2 summarises their properties.

Delays may be inserted into detonating circuits in order to limit the weight of explosive firing at any instant. Blasts may therefore be initiated in a pattern to improve fragmentation. Typically a "V" pattern will throw rock blocks together towards the centre and maximise block-block impacts. Delays are usually of the order of 17-35 milli-seconds.

Poor blast design may result in poor fragmentation, flyrock or backbreak; the latter being the fracturing of the rock mass behind the rear shot-holes, resulting in potential face instability.

Discontinuity condition is an influencing factor in rock blasting. Open discontinuities can act as free faces and confine the strain energy within blocks and hence they become excessively shattered, leaving surrounding ones undamaged. Open discontinuities waste energy by venting gases, though closely packed tight joints will aid fragmentation.

A properly managed primary fragmentation method will minimise the work necessary for the secondary fragmentation phase. When required, three methods of secondary fragmentation are commonly used:

- secondary blasting
- drop balling
- hydraulic breakers, or rock poppers

Ripping can sometimes be used for primary fragmentation in hard rock materials where the rock is naturally fractured such as in steeply dipping sedimentary sequences. Where it is cost effective this technique avoids the environmental problems associated with blasting. Mechanical excavation is discussed further in the section relating to weak rock quarrying.

Low-cost options available to excavating hard rock in small locally run operations include the barring-down of blocks in a jointed rock mass and the use of fire and water quenching methods to split the rock.

4.2.3 Weak Rock Quarrying

Weak rocks are generally capable of excavation without the aid of explosives. Mechanical shovels or back-actors and ripping are the most commonly utilised methods, although labour intensive operations may also be employed.

Figures 4.6 and 4.7 summarise the parameters governing mechanical excavation and indicate the technical boundaries to its use. Excavation of a weak rock depends on its geotechnical properties and on the type and size of equipment used. In mechanical excavation the cutting parts of the equipment must be capable of being either inserted into the discontinuities of the rock mass or forced into the fabric of a weak or unconsolidated material. The discontinuity spacing and orientation in conjunction with the strength of the intact rock material are key factors. The separation, infilling and wall strength of the discontinuities are also important factors. As there may be high rate of cutting wear in abrasive materials, the mineralogy and, in particular, the amounts of quartz are also important.

Conditions are likely to be particularly unfavourable for ripping where the dominant discontinuities are either sub-horizontal or sub-vertical, or where the “run direction” is parallel to the rock structure. Excavation can be made easier where the excavation plan incorporates benches, so that rock blocks can have greater degree of movement.

It is usually cheaper to break up rock masses by ripping rather than by blasting, although this may have to be balanced by lower production rates and poorer fragmentation. The latter may be a particular factor in rocks where discontinuities so widely spaced that large blocks are produced, which cannot be handled by the crushing plant.

Modern large hydraulic excavators have proven to be effective in ripping moderately weak massive rock, (Karpuz 1990). Similarly large scrapers may be able to excavate weak rocks without resorting to ripping.

Where a rock mass cannot be broken up satisfactorily by ripping or direct digging then other means may be required such as hydraulic rock breakers or light blasting.

Figures 4.8 and 4.9 indicate some of the geotechnical problems that may be associated with the stability of weak rock quarries. By their inherent nature they are likely to be significantly less stable than hard quarries unless slope faces and benches are appropriately designed.

4.2.4 Borrow-Pitting

When compared to quarrying, borrow pits require a higher rate of land use owing to the shallower depth and the need to exercise greater environmental control. For convenience operation can be divided into dry pits and wet pits, the difference being the working method employed.

In wet pits the degree of contamination by plastic fines is a crucial factor in their viability. Working depth is restricted to about 10 metres and the recovery of material is about 75% at best. Wet pits may be excavated by:

- Small dredger using a cutting and suction pump
- High-pressure monitors and suction pump
- Small drag-lines
- Extended arm bucket excavators

Dry pits are more common then wet pits and a wider variety of materials can be worked. Modern excavating and loading plant comprises hydraulic shovels or backhoes loading into dumper trucks but in smaller operations wheeled front end loaders can perform both tasks over short distances, providing the material is loosely consolidated and the access routes are well maintained.

In dry seasons it is important to provide measures to minimise dust generation both for the efficient working of machinery and to reduce the impact of dust on the local environment. In wet seasons efficient working will be more difficult and access routes will deteriorate more quickly. Excavation in dry pits is usually carried out using conventional earth-moving plant such as bulldozers, scrapers and front-end loaders and trucks. Face shovels and hydraulic excavators are used in some alluvial pits. Cemented or densely packed gravels can be loosened by ripping, or possibly in extreme cases by light blasting. The main geotechnical problems in dry pit operations are face collapse, variability of the deposit and the occurrence of large boulders and the presence of cemented layers.

The selection of an excavation methodology for soil, sand and gravel deposits needs to take into account a number of key factors that are likely to be related to their mode of origin, such as source geometry, groundwater conditions, and variability, Table 4.3.

4.3 Processing

4.3.1 General

Processing of as-excavated resource is undertaken to produce construction materials that meet required specifications by means of either mechanical alteration or physical selection. In general terms, materials utilised for common fill, would normally require no processing in contrast to high quality hard-rock aggregates which can be subjected to several phases of crushing and sorting. The amount of processing required is a function of the relationship between the as-extracted character and the required mechanical, chemical and physical properties.

Processing plants can be fixed or mobile.. Fixed plant is more common in large, established quarries, while semi-mobile plant is more appropriate for major construction projects where the life of the quarry is directly related to the duration of the project. Sometimes very light mobile plant is utilised in small-scale operations where minimal amounts of processing are required.

Processing is used to:

- reduce the excavated material to suitable sizes of aggregate
- group the sizes together where required into appropriate gradings
- remove unwanted fines
- reduce oversize

4.3.2 Crushing

The process of material size reduction in a quarrying operation is known as comminution. In a typical hard rock operation blasting may produce rock spalls with a size of around 300-1000mm, which can be fed into the primary crushing operation for reduction to around 100-200mm. Material is then passed onto secondary and even tertiary crushers for further size reduction and shape improvement.

There are a wide selection of crushing plant available and careful selection is necessary, depending on the characteristics of the stone and the purpose required. Crushers are of two basic types:

- Compression crushers: These subject the stone repeatedly to a squeezing action as it passes down through the machine, reducing it in size until it is able to pass out of the crushing chamber.

- Impact crushers: These subject the stone to a rapid hammer action of repeated blows until it has become broken enough to pass out of the crushing chamber.

Important key factors when choosing crushing plant are summarised in Table 4.4. Typical examples of the major types of crusher are shown in Figure 4.10

Compression-type crushers (jaws and gyratories) have maximum reduction ratios of 8:1 to 10:1. However, they normally operate at about half this because of the excessive production of fines. Impact type crushers generally operate satisfactorily at 10:1 or 20:1. Most crusher types are available in either primary or secondary versions, but jaw and gyratory crushers are usually primary crushers, while impactors and cones perform best as secondaries.

As well as comminution it is important to control the shape of the particle produced. The general aim is to ensure not more than about 35% of flaky particles are produced, to keep within the limits of most specifications. Shape is controlled by the following factors:

- Type of rock: limestone and dolerite produce better shape than slate or quartzite
- Reduction ratio: the ratio of the crusher feed particle size to the crusher aperture size: the lower the reduction ratio, the better the shape
- Rate of feed into the crusher: crushers perform best when fed at a maximum rate
- Crusher type: impact are better than compression for producing good shape

Secondary crushers have three key functions:

- Comminution of primary crusher output to useable sizes
- Correction of particle shape (make particle more cubic)
- Disintegration of weak porous particles

Tertiary crushers are employed to further improve the shape of flaky aggregates and to increase fines percentage. They generally have a reduction ratio of only about 2:1.

Crushing is considerably less significant in gravel pits than rock quarries since oversize may only be 100-200mm. If these are not discarded then it is quite common for there to be only a single stage crushing operation, usually gyratory or cone. Due to excessive shear impact crushers are generally avoided if siliceous material is th be processed.

Hand crushing is frequently employed in small-scale labour intensive road construction or maintenance projects in some countries. It is most frequently used to break-down cobble sized alluvial material into coarse aggregate. Fookes (1968) reported that well-supervised hand crushing tended to produce better shaped aggregate from metamorphic rocks in Nepal than a single jaw crusher operation.

4.3.3 Sizing and Screening

Multi-sized crushed material will normally need to be separated into a range of sizes for either separate use or for recombination into a range of sizes according to demand. This is commonly done by a process of scalping, washing, scrubbing or screening. Scalping is done over bar screens in order to separate coarse spalls, which will be passed to the primary crusher, from the undersize that can be routed straight to the secondaries.

Washing is the term used to cover stripping off of relatively small quantities of silt adhering to gravel-sized particles. The quantities of water and degree of agitation required are quite small and the process may involve no more then directing jets of water onto material as it passes across a vibrating screen.

Scrubbing is a more energetic form of washing, used when the fines are more clay-rich and more difficult to remove. Drum-scrubbers are the commonest form of plant used in this process. These are inclined revolving steel cylinders lined with projecting steel shelves to lift and turn clayey gravel in a continuous water shower, Figure 4.11. In this example the sand slurry is agitated by the helical screw, which also elevates and drains the coarse sand product.

Screening is the means by which run-of-crusher materials down to about 3mm are separated into useable aggregate fractions, (Figure 4.12). In addition to this primary sizing task, screens are used for scalping oversize, waste removal and dewatering. The basic requirements of a screen are to:

- agitate the material sufficiently to enable the undersize to pass through easily
- transport the material quickly enough to clear away the oversize

A wide variety of screens are used in aggregate processing, but the most common is the vibrating multi-deck. These are usually arranged in vertically stacked decks, becoming progressively finer, with screening taking place by rejection off each screen (rejection screening). Alternatively, horizontally vibrating screens can be used with the vibration inducing movement along the screen. Typically these are single decks of increasing aperture size, thus screening occurs by selection through the screen (selection screening). Rejection screening is generally preferred. Deck inclination is usually in the range 12-20^{o} and the set-up consists of two to four screens, typically 40mm down to 3mm. Screens may be of round, square, hexagonal or slotted punched steel plate; woven wire or welded steel rods; hard rubber or polyurethane

Particles are repeatedly presented to the apertures and separation is a function of particle shape, size and duration. Screens are designed to be 95% efficient but this is influenced by the incidence of “pegging”, or the blocking of screen apertures by individual particles, and “blinding”, or the agglomeration of fine particles by moisture, which then stick to the screen. Both problems can be minimised by screen vibration.

In the case of weak rocks the complexity of the processing operation depends on the quantity of oversize (+40mm especially) and undersize (-5mm) material, particularly the silt and clay fraction. The quantity of oversize dictates whether a secondary crusher is required. The quantity of undersize influences the decision whether or not to install washing equipment. The proportion of fine material is often crucial to the viability of the operation because more complex equipment, such as hydrocyclones are necessary to separate out the clay, followed by settlement in lagoons to precipitate out again the fine material before it reaches the outside environment. Typical processing plant flow-sheets for hard and soft rock are shown in Figures 4.13 and 4.14

For the finer aggregate sizes screening is replaced by separation through particle settling velocity, or classification. Sizing by classification is cruder than the cut obtained by screening, with separation efficiencies of only 30-80% as compared with 90-95% when screening coarse aggregates. Classifiers are of very varied design, but all have two features in common; a means of agitating a sand slurry and separating a suspended overflow of fine sand and silt from an underflow of bottom load of coarser grains

Low-cost and labour intensive operations are illustrated in Figure 4.15.

4.4 Quality Control

Materials production from quarries and large borrow-pit sites requires positive management to ensure that the desired quality of materials is maintained. This may involve the appointment of a person responsible for implementing the necessary sampling and testing regimes required for production control. Details of sampling and testing procedures are discussed in later sections.

It is also important that the production process is controlled to ensure optimum and cost-effective plant performance. Larger quarry production plants usually have a generalised plant flow-sheet and a system of recording the output of various materials and deliveries to site by way of a Quality Assurance programme. Typical features of such QA systems are:-

- A quality management structure with authority and reporting procedures clearly defined
- An evaluation of the material reserve and testing of raw materials that ensures that product quality is attainable
- A competent design of the process plant that demonstrates an ability to meet performance criteria
- A programme that ensures that instruments are calibrated and tested on a routine basis

- An appropriate operator education and training programme
  - Procedures to prevent the delivery of rejected materials
  - Defined plant maintenance and inspection procedures
  - Clear guidance on environmental mitigation issues

4.5 Environmental Issues

4.5.1 General Impacts

The extraction of road construction materials is increasingly influenced both by environmental pressures and by the requirement for land restoration after use. This has cost implications that affect the viability of an operation and needs to be included in assessments of whole-life material utilisation costs.

Key environmental impacts have been identified as follows:

- Material resources
  - – permanent loss of natural resources
- Morphological damage
  - – modification of the natural drainage
  - – increased soil erosion and siltation of waterways by disturbance of soil
  - – destabilisation of slopes
  - – compaction of soil surrounding the borrow area by plant or soil bunds
- Ecology
  - – loss of wilderness and forest
  - – displacement of species and habitats
  - – loss of potential productivity of agricultural land
- Pollution
  - – contamination of water and soil by fuel and oil spillage
  - – generation of dust during the processing, loading and transporting of materials
  - – increased dust generated by vehicles along access tracks
  - – littering
- Social and health impacts
  - – creation of habitats for disease
  - – landscape alteration and interference with natural beauty
  - – bisection of communities or farms
  - – loss of land legacy
  - – loss of antiquities, cultural heritage, areas of cultural concern, such as graves
  - – hazards to pedestrians and animals, including open or unmarked trial pits, demarcation beacons, etc
  - – safety risks to local population by exposure to heavy plant and traffic
  - – noise of blasting, traffic, plant and drilling

4.5.2 Mitigation of Impacts and Restoration

Generally quarries and pits go through a number of identifiable stages from development, through extraction and backfilling to a final restoration phase. These stages need to be part of an integrated quarry or pit operational plan. Programmed, progressive site rehabilitation reduces double handling of waste and overburden and optimises the use of plant during slack production periods. The aim should be to integrate development, extraction and restoration as closely as possible.

Although environmental management legislation may have been enacted, this may not be followed strictly, especially in the case of small scale, largely manually operated operations. For the larger, mechanised operations it is important to establish a Code of Practice allied to a clear work plan. This should seek to ensure the following:

- avoidance of numerous working faces
- avoidance of high irregular and potentially unsafe faces
- planned stockpiling of overburden or use as a screen and for the final restoration
- minimising machinery noise during face working (blasting and digging), hauling and processing
- minimising dust, especially in dry climates. All screens, for example, have a tendency to be noisy and dusty but there are measures to minimise these impacts
- planned disposal of noxious quarry wastes, e.g. oils
- careful protection of explosives and quarry plant

If it is necessary to construct access roads or tracks, these should be provided in such a manner as to minimise disturbance to the local population and environment. They should be located at a safe distance from permanent dwellings and, if necessary, fencing should be provided for dwellings to protect local people and livestock. Access tracks, if close to dwellings or cultivated land, should be watered on a regular basis to prevent unnecessary health, environmental and vegetation damage caused by dust. Where access roads are constructed, adequate provision should be made for drainage at stream and water crossings to prevent flooding or diversion of water courses. If the access track is to be retained after rehabilitation of the borrow area, it should be shaped and graded to a suitable standard as directed by the resident engineer.

Careful stockpiling of overburden soils will preserve soil quality by reducing exposure of the soluble minerals and organic matter in the soil to oxidation. The longer the soil is stockpiled, the greater will be the change in soil structure and nutrient availability due to rapid decline in the soil organic matter. The length of time that the soil is stockpiled should be minimised to prevent leaching and loss of nutrients and soluble minerals. There will be benefits in restoring borrow pits from stockpiles towards the end of any dry season since a wet soil will compact more than a dry one, thus destroying the fabric

Environmentally aware procedures have been developed by TRL for the opening, working and rehabilitation of borrow pits, (Gourley and Greening 1999). These guidelines, which may be adopted for quarry operations as well are summarised as follows:

1. During borrow pit investigations obtain information necessary to plan operations on:
   - soil profile (thickness of top-soil and sub-soils).
   - Areas to be designated for storage of top-soil and sub-soil materials
   - Amount of scrub and vegetation to be removed.
2. Scrape off the thin organic layer (usually 100 to 150mm); this can usually be distinguished by a change in soil colour; care must be taken to minimise contamination with the underlying material
3. Stockpile the top-soil material in shaped berms
4. Remove the sub-soil layers in sequence, and stockpile separately from the top-soil in shaped berms
5. Rip, stockpile, load and remove the seam material
6. Grade, contour and rip the floor of the borrow area if required
7. Spread excess or spoil material evenly to level the floor of the borrow pit
8. Spoil and waste construction materials may be dumped and levelled in the pit. Materials used for back-fill should either be chemically inert or, if not, the quarries or borrow pits should be subjected to specialist investigation and design procedures before backfilling
9. Spread the sub-soils in the reverse sequence to that in which they were removed to reinstate the layers in the correct order, restricting plant movements to the minimum necessary
10. Spread the top-soil evenly over the surface, restricting heavy plant movements to the minimum
11. Fertilise and seed, as required, or as agreed with the land owner or legislatory authority.

An incentive for dumping in operating pits is that environmental impact reduction measures may already be in place. Two general types of quarry fill: Quarry rejects – comprising overburden, inferior aggregate, crusher reject, wash tailing, stockpiled topsoil and subsoil – and bulk imported waste

Long-term slope stability of quarry walls can be a consideration where public or livestock access is to be allowed. Bench and rock face stability that is adequate for quarrying operations will probably not satisfy public access requirements.

General Source Type	Excavation Type	Typical Materials
Quarrying	Hard-Rock. Predominantly by drill and blast methods.	Unweathered strong igneous, sedimentary and metamorphic rocks (cf Table 2.12)
	Weak-Rock. Mechanical excavation; may be aided by light blasting	Mudstones, shales, weak limestones and sandstones and weathered hard-rock materials (cf 2.13)
Borrow-Pitting	Wet Pit.	Alluvial sands and natural gravels below the water table.
	Dry Pit	Alluvial, terrace, fan, beach, natural gravel duricrust deposits, above the water table;

Table 4.1 General Construction Material Excavation Types

Explosive	Description	VOD m/s	Energy Output Ratio (s/h %) - Strong rocks	Energy Output Ratio (s/h %) - Weak rocks	Comment on Use
ANFO	Mixture of 94% ammonium nitrate and 6% fuel oil. Increased shattering power is achieved by the addition of 5-10% aluminium dust (ALANFO)	3500	13/87	25/75	Cheap, energetic and safe to transport, store and load. Water-sensitive: cannot be used in wet holes and may only be partially successful in damp holes. ALANFO has increased shattering power.
Watergel	Saturated aqueous solutions with gelling agents and ammonium nitrate grains in suspension	4300-4500	21/79	33/67	As for ANFO but may be used in wet holes.
Emulsions	Ammonium nitrate droplets dispersed in an oil-wax mixture	4500-5000	29/71	48/52	Waterproof and have the highest shattering ability of the bulk-type explosives

Notes: VOD: velocity of detonation
s/h: ratio of strain energy to heave energy

Table 4.2 Typical Bulk Explosives Used in Quarrying Operations

Source Type	Key Factors Influencing Excavation
Alluvial Sands and Gravels	Material likely to be close to the water table and require wet pit working conditions. High probability of localised material character variation within generally lenticular shaped deposits. Care may be required with respect to local flooding in some environments. Environmentally sensitive.
Terrace Deposits	Most likely to be dry pit conditions although wet conditions are encountered in thicker deposits and those adjacent to existing alluvial channels. Older deposits may have some cemented layers required ripping or hard mechanical excavation.
Fan Deposits	May be wet or dry pit conditions, although the latter is usually most likely. Commonly very variable in particle size.
Beach Deposits	Most likely to be worked as dry pits above storm water high level. Material commonly well sorted , i.e. poor range of particle sizes with potential for large oversize. Very environmentally sensitive.
Residual Soils	Residual soil profiles may be up to the order of 10m thick below an unusable topsoil layer, but depth of working may be limited by pit stability or by water table, as the material is not recoverable below water table. Deposits very likely to varied vertically. Generally utilised as common fill with the option of excavation by motorised scraper. Potential problem with relict corestones, particularly in granitic materials.
Residual Gravels	Source material is likely to be in thin layers, with a distinct possibility of contamination by poorer grade materials. Dry pit conditions
Duricrusts	Generally layered deposits that can contain stronger bands that may require ripping and occasionally light blasting. Distinct possibility of contamination by poorer grade materials. Dry pit conditions.

Table 4.3 Pitting Operations

Crusher Type	General Application	Description	Use and Limitations
Jaw Crusher	Usually as primary crushers, small versions may be used as secondary crushers.	Rock is broken by slow compression-release cycles between plates, on fixed and one moving on opposite sides of a wedge-shaped chamber. This narrows downwards so that after blocks are split on the compression stroke the resultant pieces slip further down on the release stroke until on the next cycle they are released.	Double toggle machines are able to exert larger compressive forces (up to 500Mpa) and able to handle the most blocky materials (up to 3m). In most quarrying operations the single toggle machines operate satisfactorily – well suited to small and medium-sized operations, including mobile plant.
Gyratory Crusher	Usually as primary crushers; cut-down versions may be used as secondary crushers.	A gyratory crusher resembles a pestle in a narrow open-base mortar, or two cones one inverted within the other. The inner solid cone moves eccentrically around the fixed outer bowl alternately opening and closing gaps around the lower rim.	Less tolerant of oversize than jaws. Tend to be used for throughputs. Work best when hey are choke fed. Outputs more fines than jaw, particle size range is narrower than from impactor but wider than a rolls crusher.
Rolls Crusher	Primary crusher.	Machines can be fitted with single double or multiple rollers, although double rolls are most common – one fixed and the other spring-loaded.	Use of the rolls crusher is limited to weaker rock (UCS<100Mpa) and non-abrasive rock such as limestone and shale. Cheap in relation to high capacity and easily transportable – good choice for weak rock for select fill or for demolition rubble. Well suited to clay-rich feed and processing argillaceous and slabby rocks which would tend to block jaw or gyratory crushers.
Cone Crusher	Usually used as secondary or tertiary crushers.	Similar in operation to small gyratory crushers in having an oscillating inner and a static outer one - the inner cone is pivoted from below rather than suspended from above.	They have large capacities in relation to small size. Compared to impact crushers they produce a narrower range of sizes, less fines and more flaky particles.
Impact Crusher	Usually used as secondary or tertiary crushers.	Rocks broken by the action of rapidly rotating or beaters attached to a central shaft that - may be horizontally or vertically mounted. The feed particles cascade into the crushing chamber and shatter on impact with the beaters or are deflected by them to strike hardened breaker plates lining the chamber.	Relatively light and cheap for their capacity and do not require elaborate foundations. Their main disadvantage is the cost of frictional and chipping wear on breakers and plates and the consequent downtime for replacement. Generally limited to rocks with UCS <150Mpa and free quartz content <5-7%. Advantageous with wet and sticky clay-rich gravels. Product shape is good. Large impact breakers (250-500RPM) have very high reduction ratio, up to 40:1 but cannot handle as good a quality rock as gyratories. Smaller secondary impactors operate at high speeds (500-2500rpm) but with finer feed (<200mm) and generate more fines.

Table 4.4 Common Crusher Types and their Use

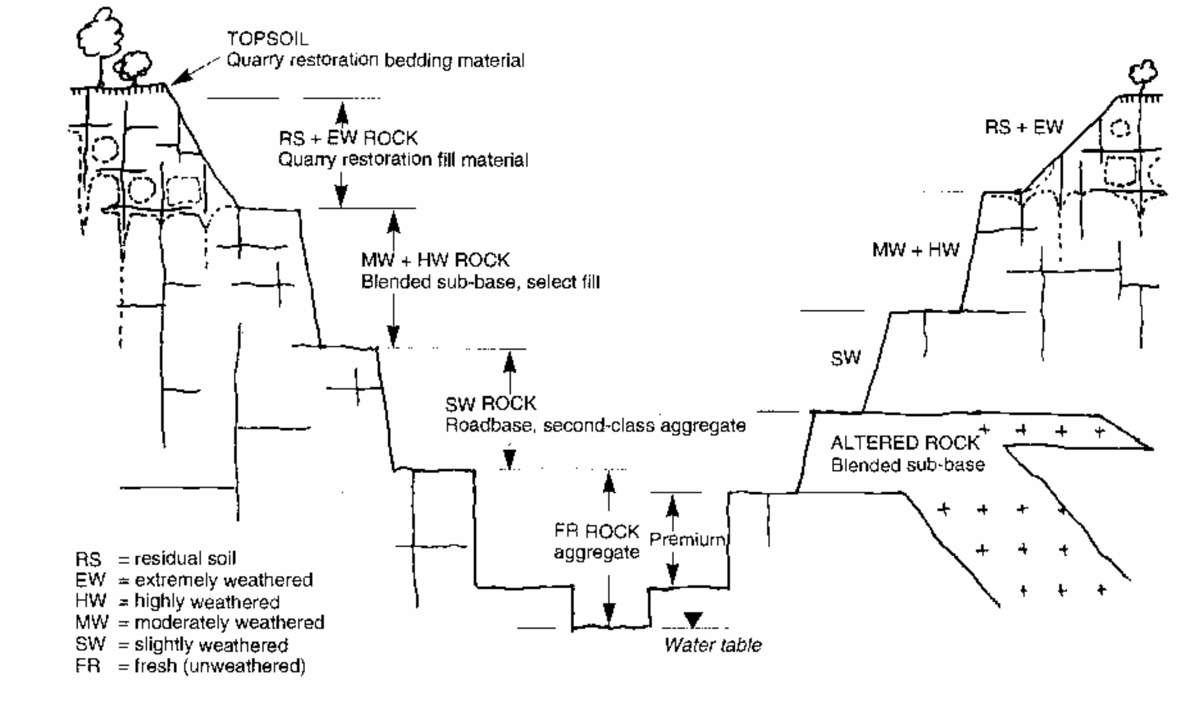


Figure 4.1 Cross section through a quarry in weathered to fresh hard rock, (After McNalty 1998).

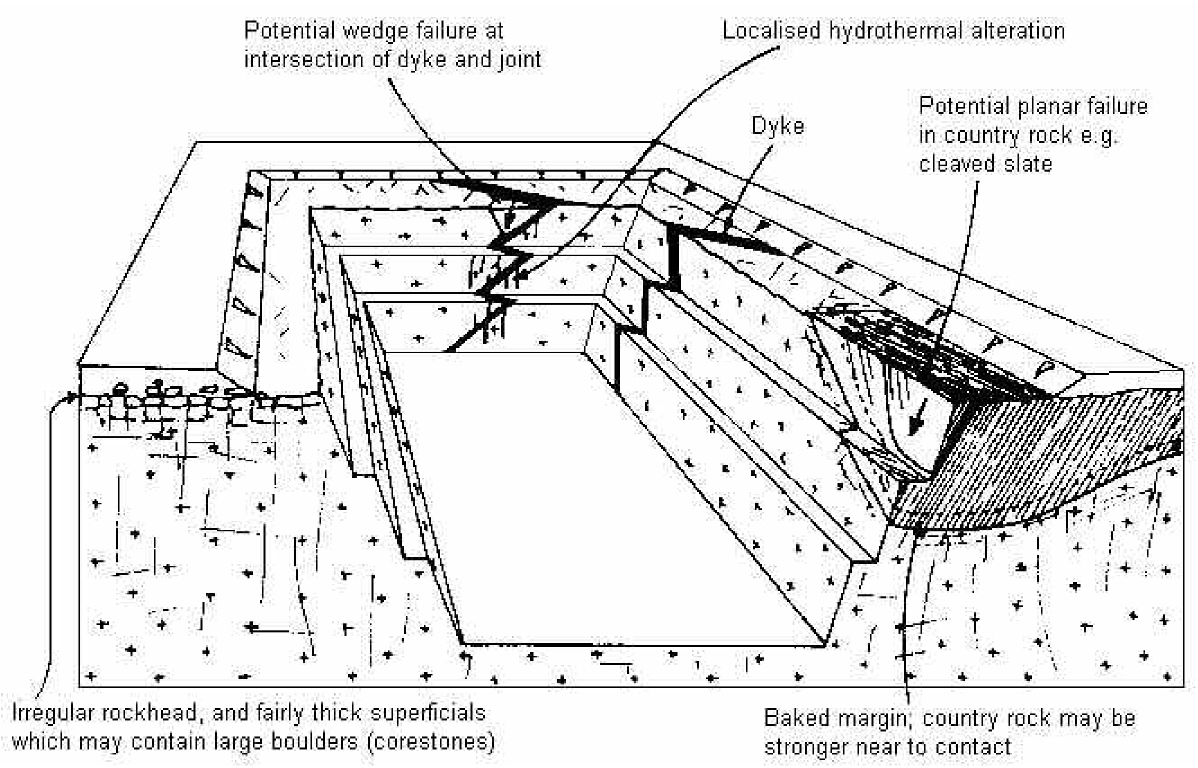


Figure 4.2 Typical geotechnical considerations associated with a large igneous intrusion quarry, (After HMSO 1998).

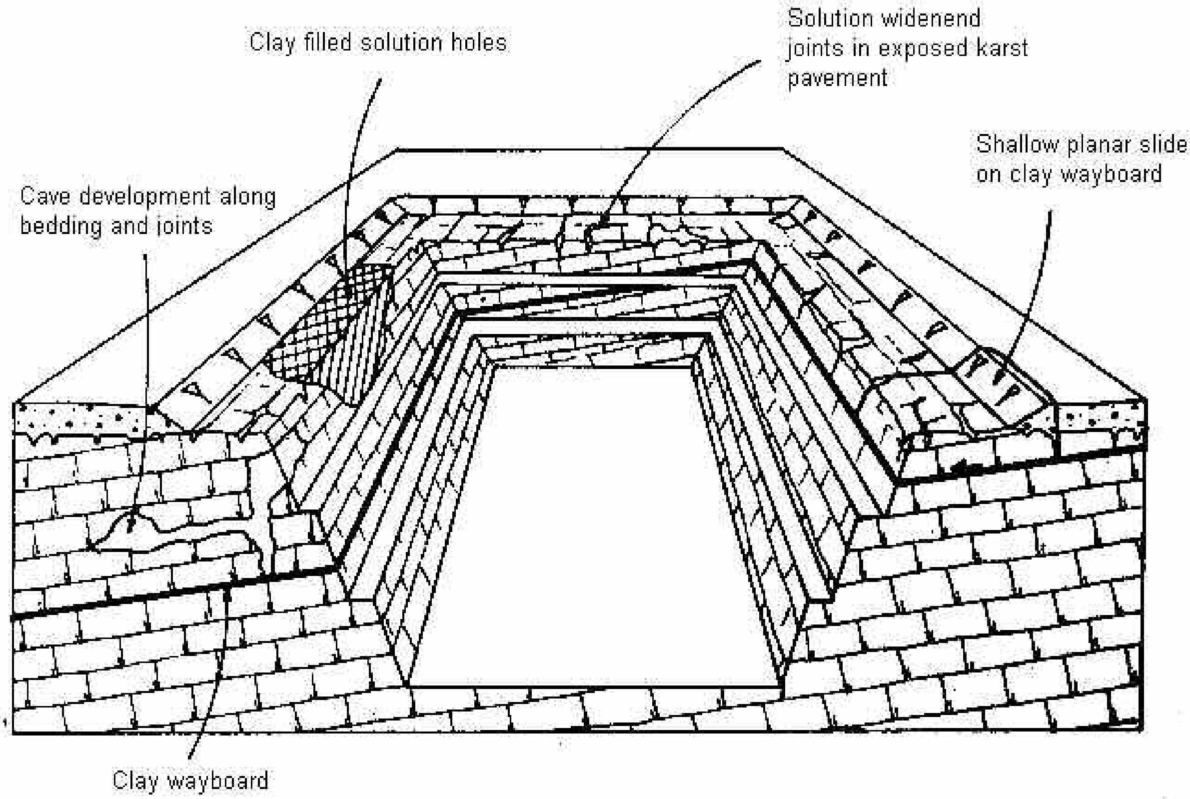


Figure 4.3 Typical geotechnical considerations associated with limestone quarrying, (After HMSO 1998).

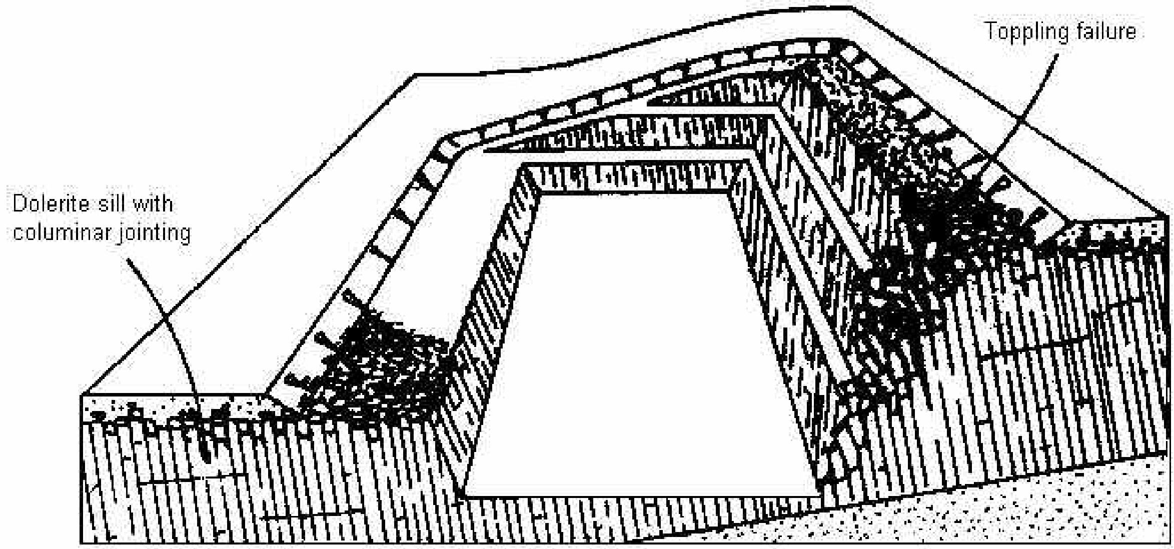


Figure 4.4 Geotechnical considerations associated with quarries in steeply dipping sedimentary rocks or jointed igneous intrusions, (After HMSO 1998).

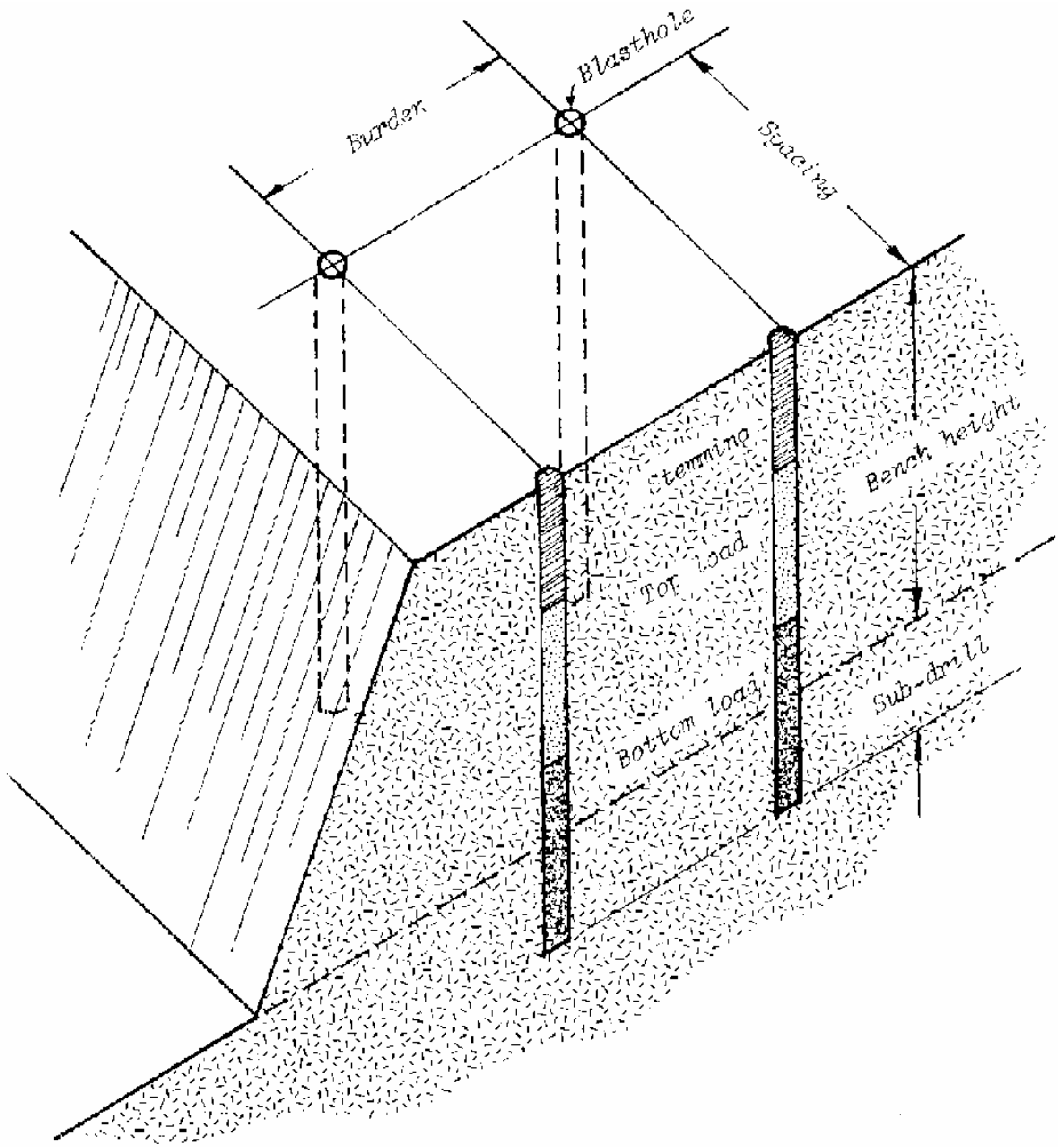
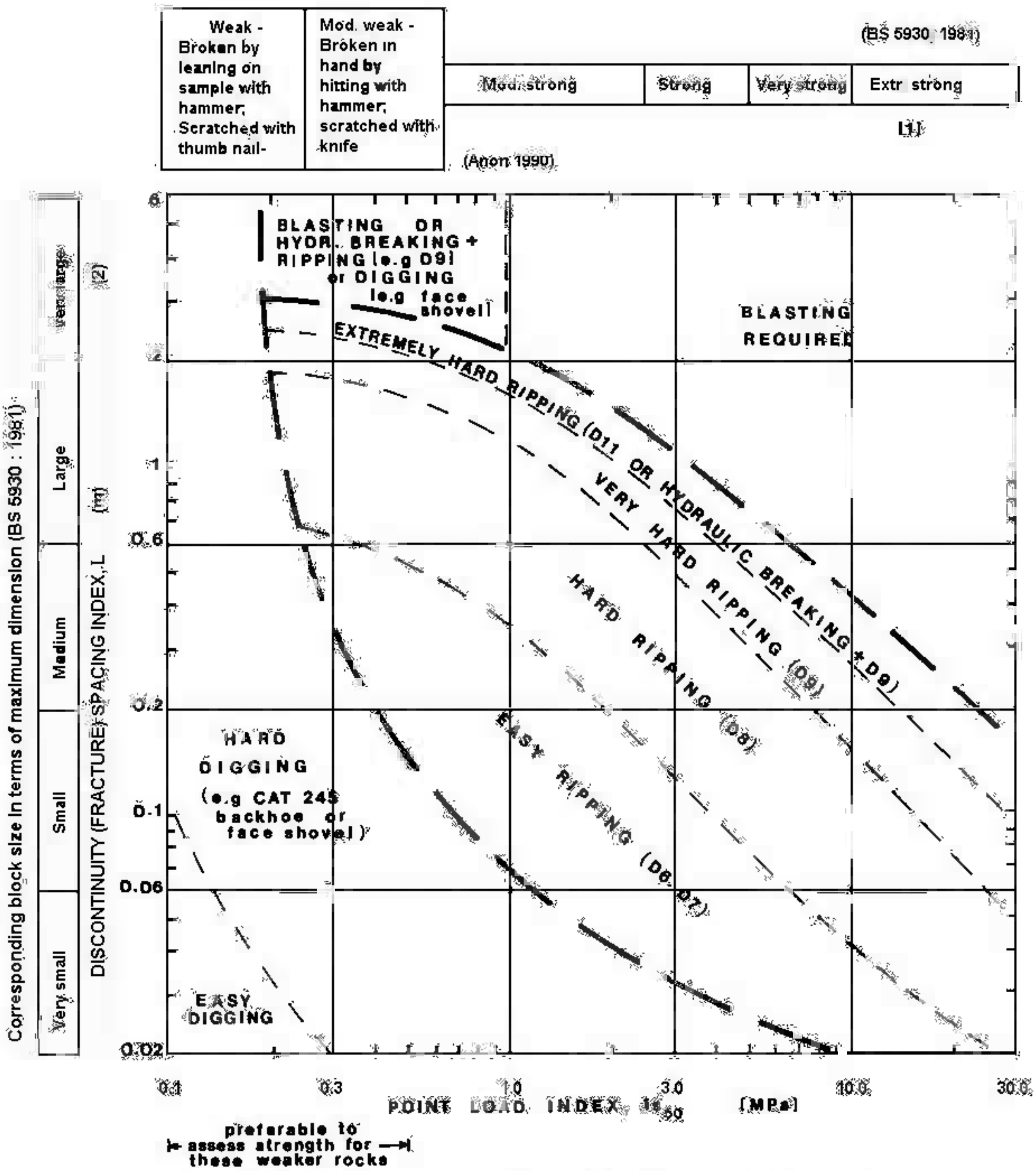


Figure 4.5 Schematic illustration of terms used in blast design (Hoek and Bray 1977)



Notes [1] Approx. correlation with point load index (UCS = 20 Is_{50}).

[2] Where the rock mass structure is unfavourable for ripping, I_f values should be increased.

$Jv = 3/I_f$, for a rock mass with three intersecting joint sets, where I_f is determined on a unit volume basis

RQD = 115 − 3.3 Jv approx. (ISRM 1981).

Figure 4.6 Excavatability (Pettifer and Fookes 1994)

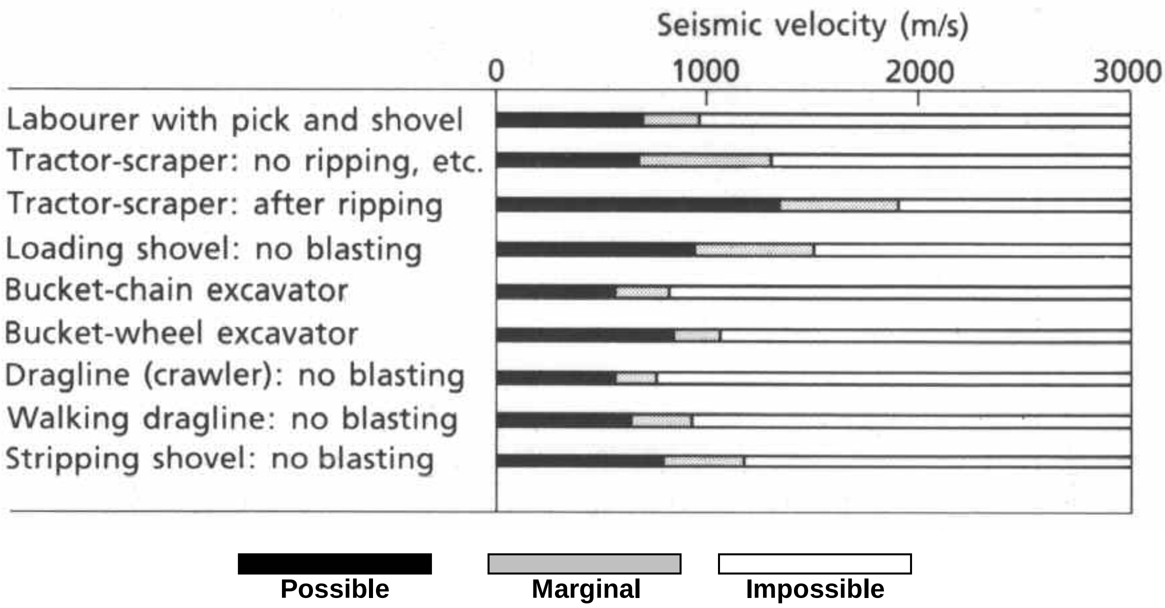


Figure 4.7 Excavatability related to seismic velocity (After Atkinson 1971)

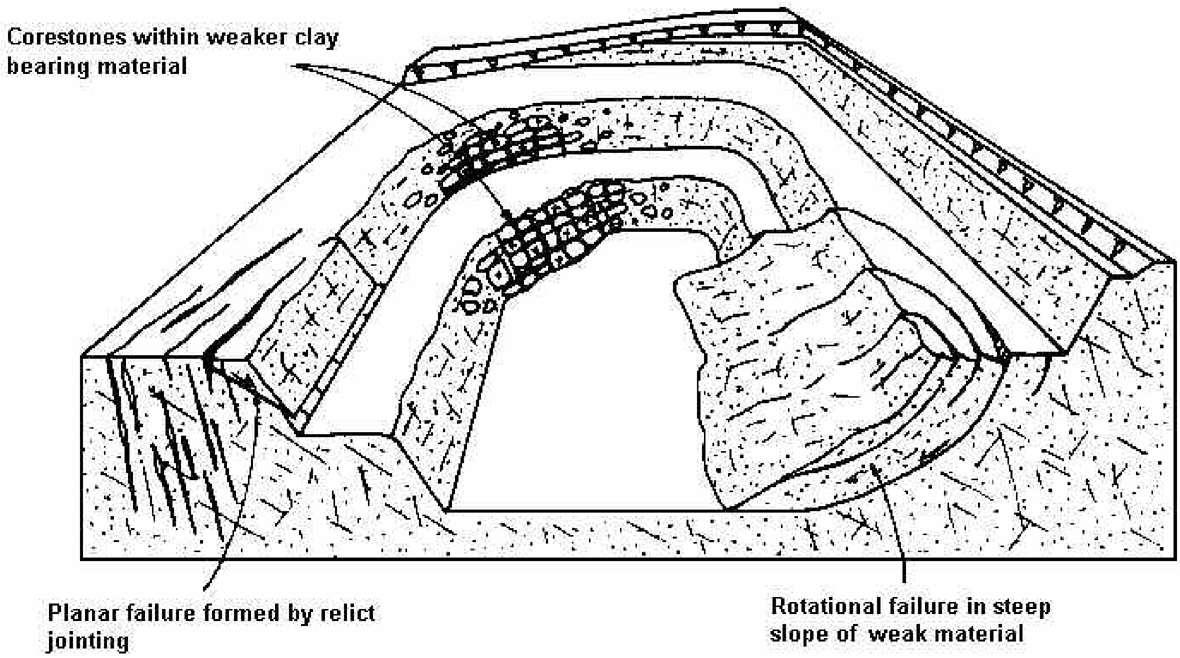


Figure 4.8 Typical geotechnical considerations associated with a quarry in weak material containing relict structures, (After HMSO 1998).

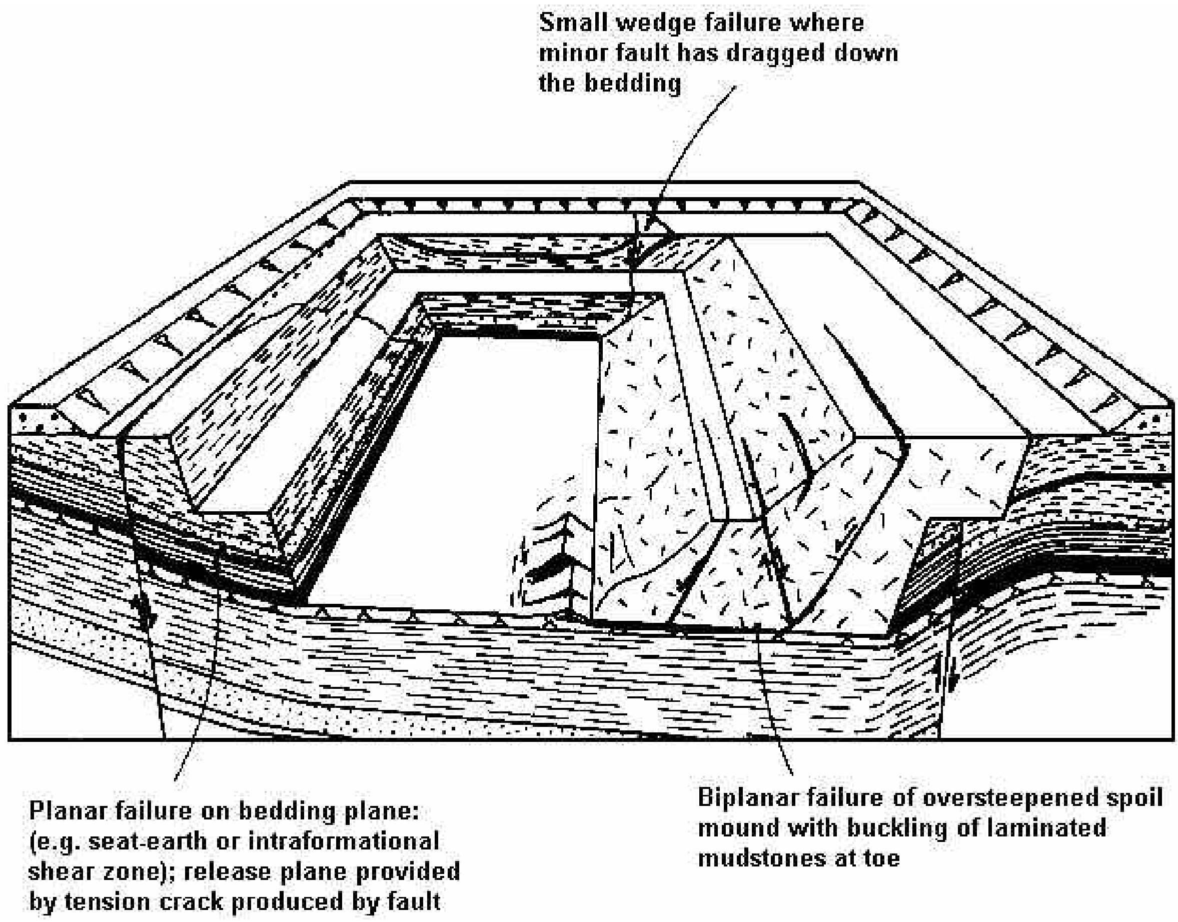
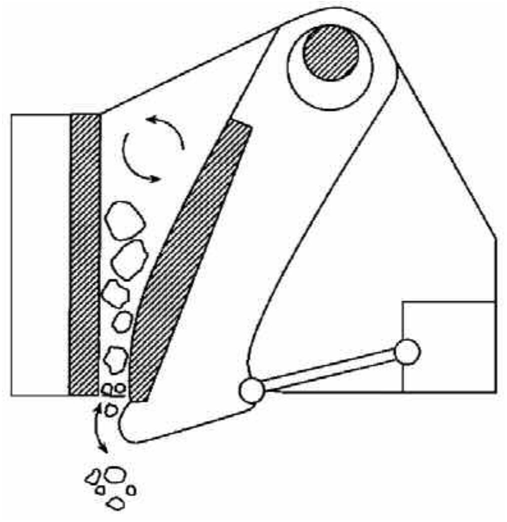
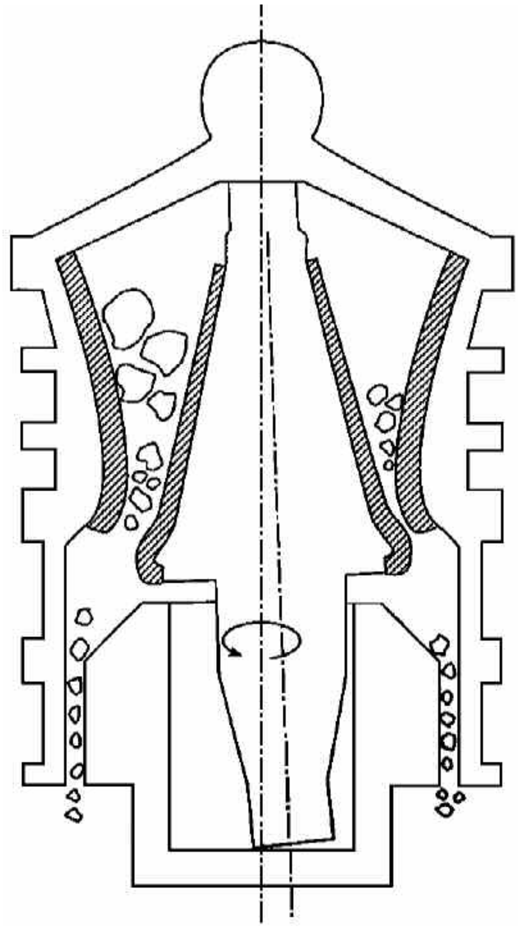


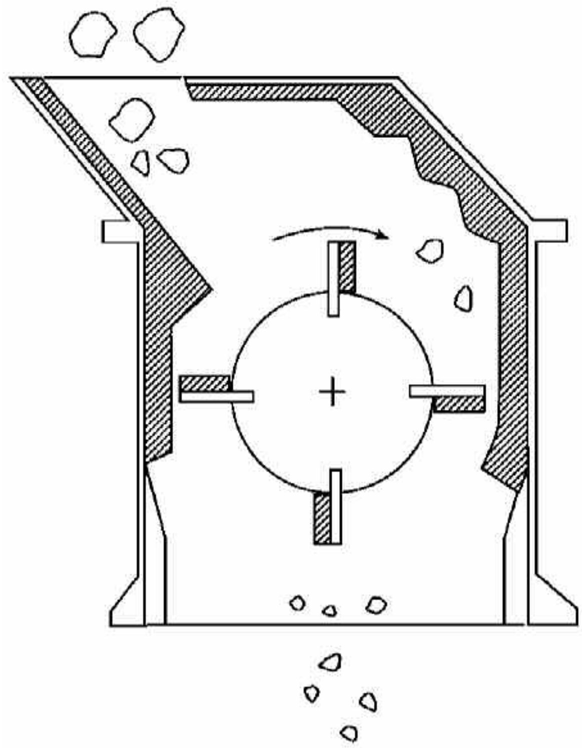
Figure 4.9 Typical geotechnical considerations associated with bedded weak sedimentary rocks, (After HMSO 1998).



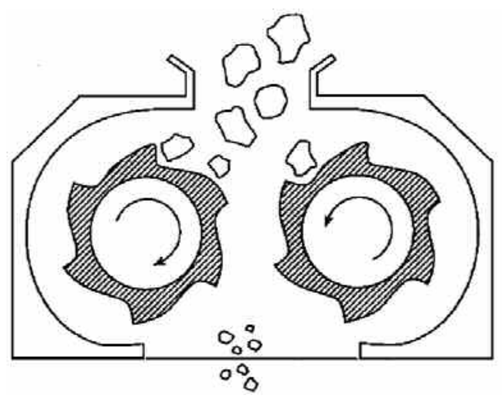
a. Single-toggle jaw crusher, schematic. The left shaded plate is fixed with the right hand plate moving in an elliptical path.



b. Gyratory crusher, schematic. The inner cone moves eccentrically but the outer plate (mantle) is fixed.



c. Impact crusher, schematic. Rock pieces falling from above are struck by fast-moving blow bars or swing hammers, and either shatter instantly or are hurled towards the armoured lining (breaker plates)



d. Double rolls crusher with interlocking teeth. Both rolls rotate towards the centre, but one is fixed and the other sprung to allow oversize blocks to pass through

Figure 4.10 Common crusher types, (After McNally, 1998).

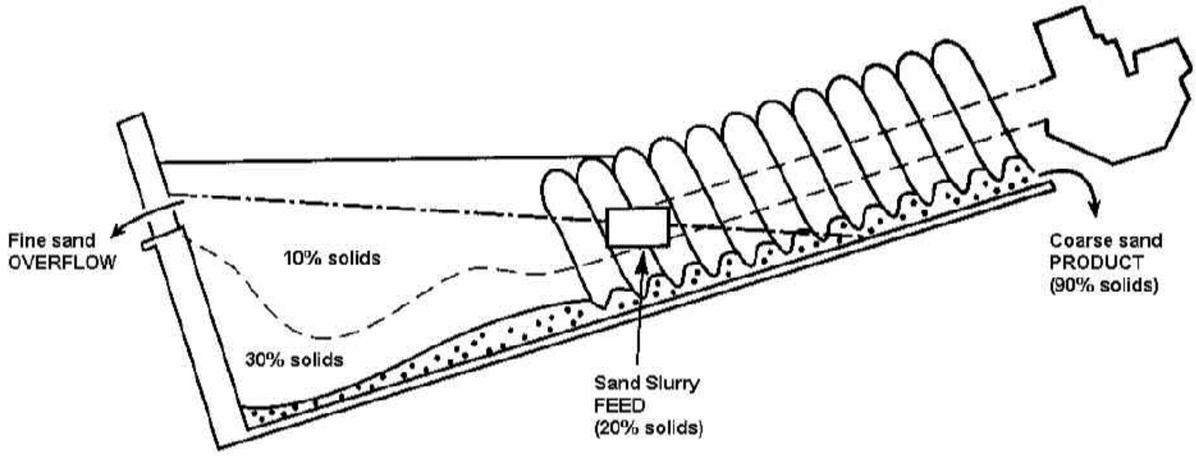


Figure 4.11 Spiral sand classifier, schematic, (After McNally, 1998)..

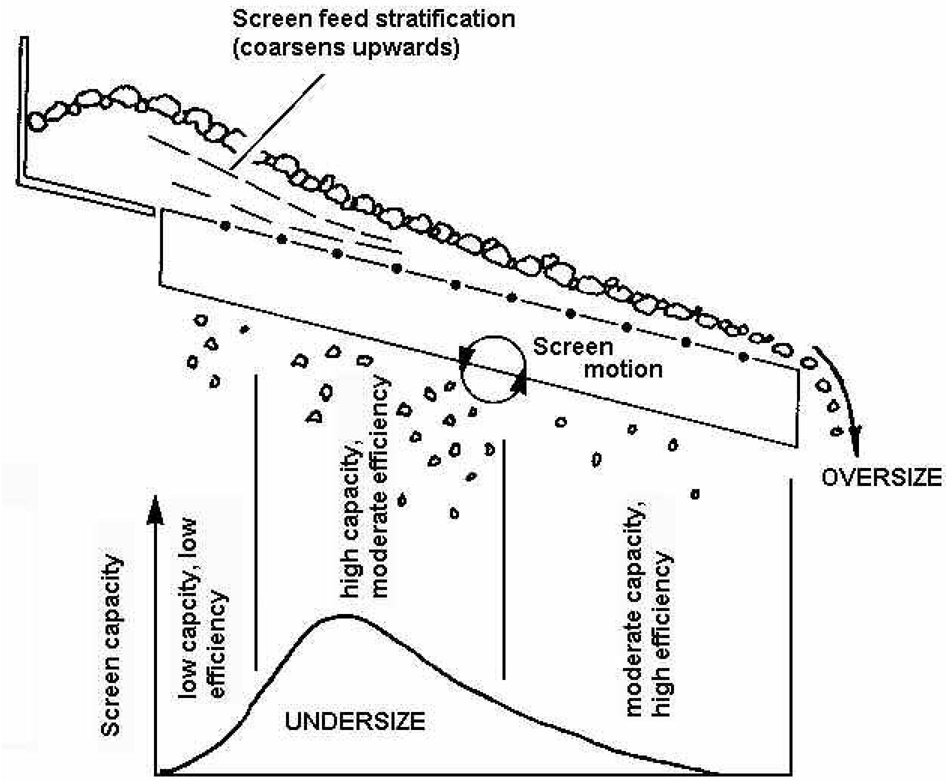


Figure 4.12 Single deck vibratory screening, (After McNally, 1998).

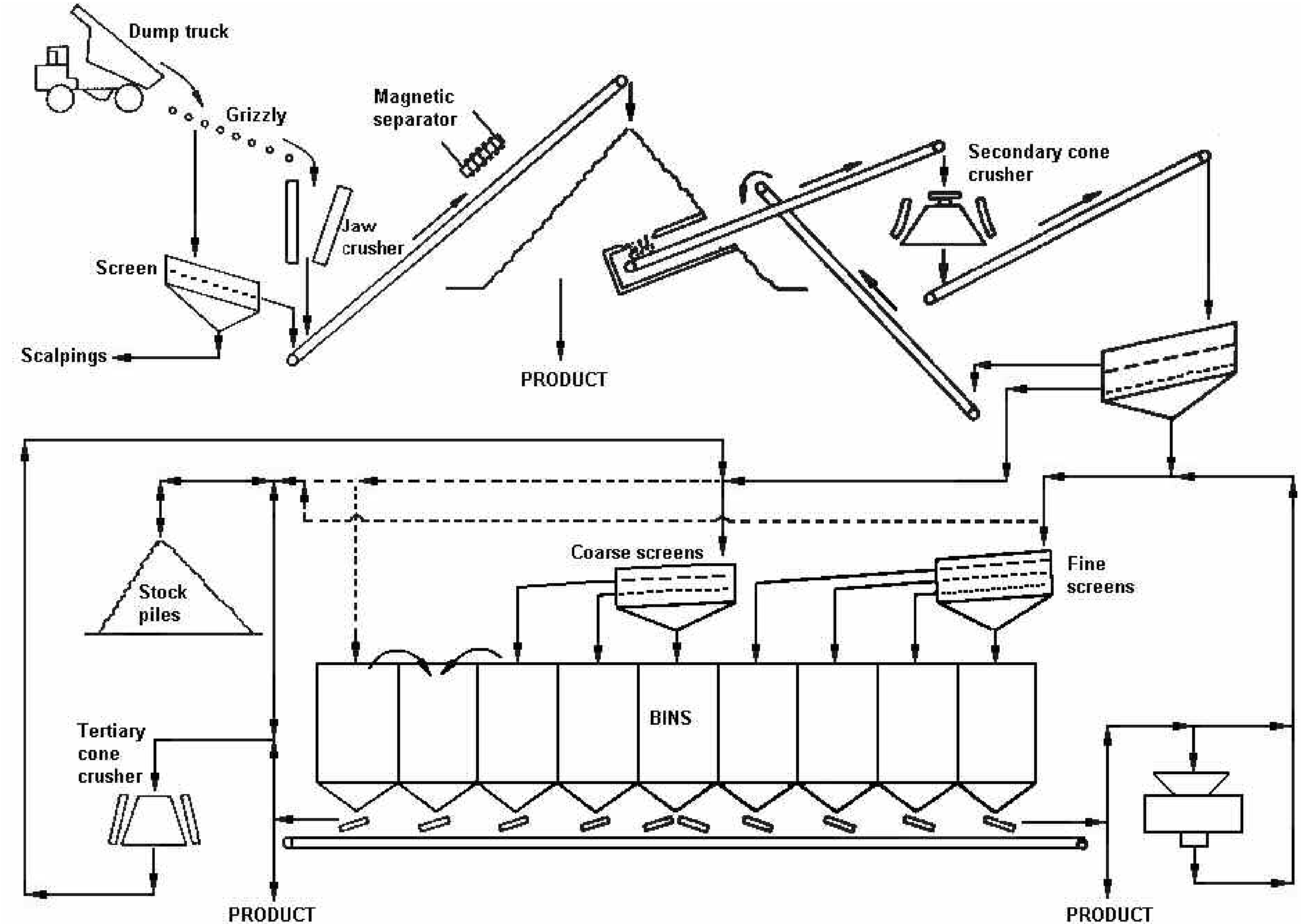


Figure 4.13 Typical Hard-rock quarry processing flow chart

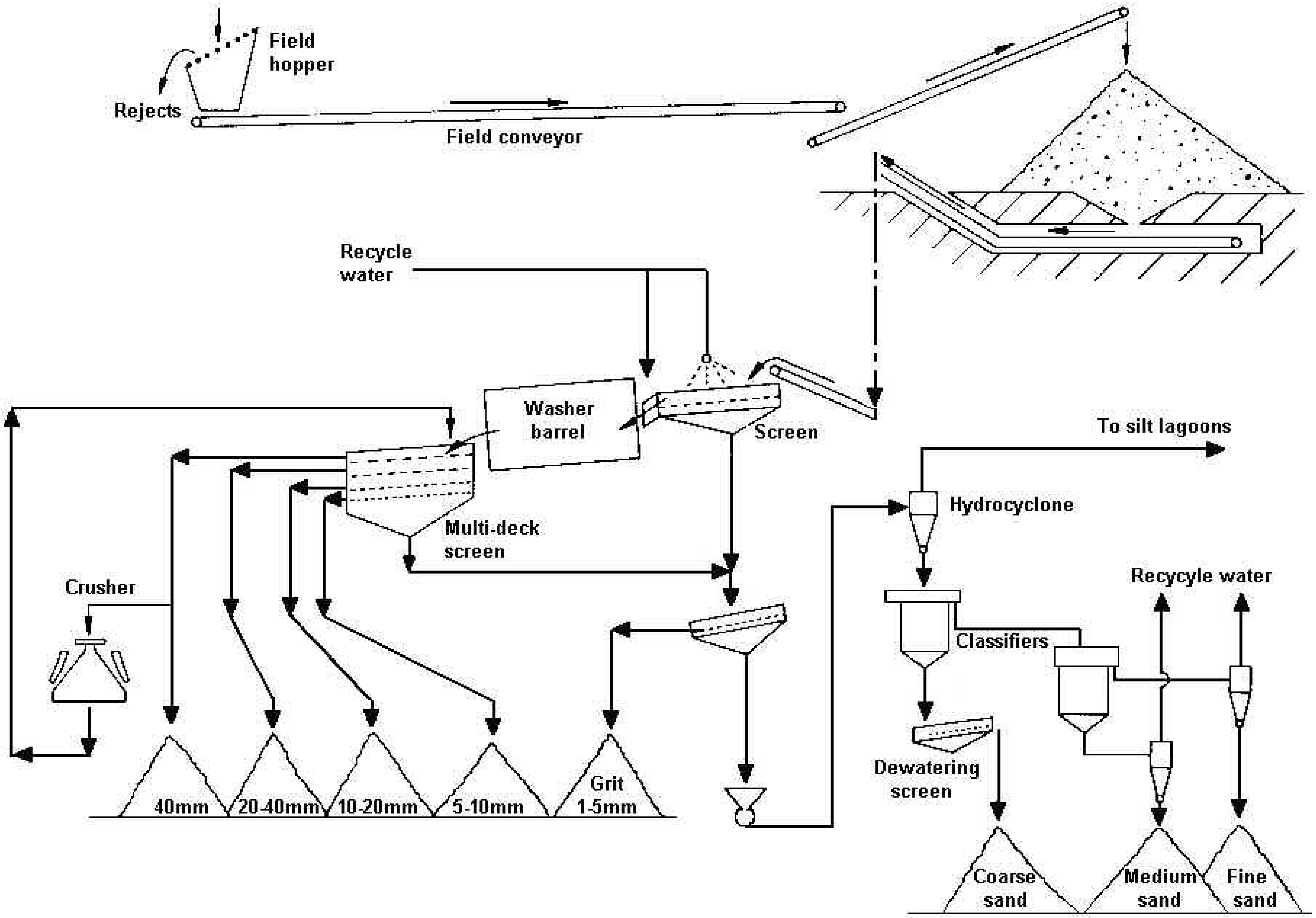
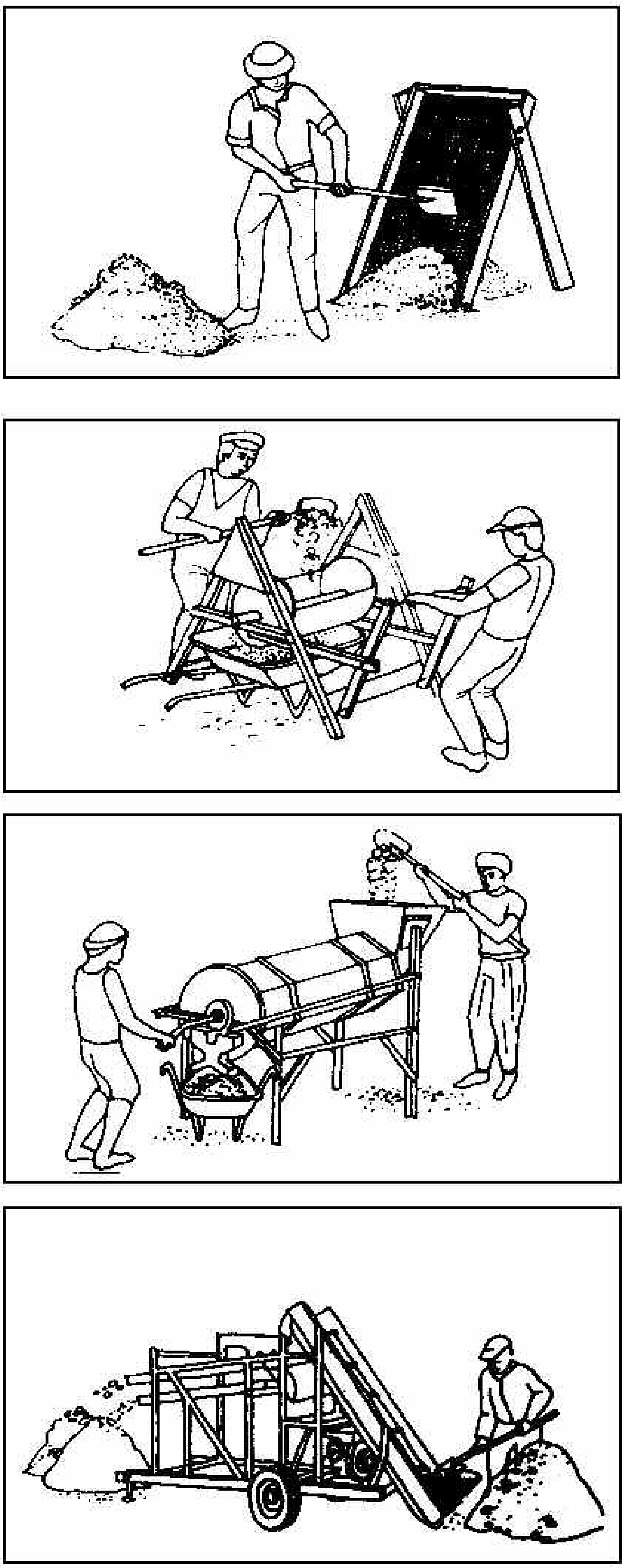


Figure 4.14 Typical sand and gravel processing flow diagram



a. Fixed screen

b. Alternating screen

c. Rotary screen

d. Vibrating screen

Figure 4.15 Low cost, small scale screening operations (After Hauber and Guillard,1990)

5 MATERIAL SELECTION

5.1 Introduction

This chapter deals with the selection and use of naturally occurring materials for pavement construction and earthwork embankments, including in situ sub-grade, imported capping layer and drainage materials. It does not include discussion of cut-slope design and construction other than as they directly influence the placement of earthworks materials. The types of construction materials discussed are defined in Figure 5.1 and listed below:

- Common fill
- In situ sub-grade
- Capping layer (imported sub-grade/select fill)
- Filter and drainage aggregate
- Sub-base
- Base
- Surfacing aggregate

Particular attention is given to the distinct character of the soils and weathered rocks from tropical regions. The interaction of these materials with the controlling climatic and weathering environments can produce distinct behaviour patterns not encountered in temperate regions.

In many cases the problem is not so much how to identify the best material for a certain usage, but to decide what is the boundary between acceptable and non-acceptable and how best to use what is acceptable. The boundaries of acceptability are not fixed, but shift in response to technological innovation (Smith and Collis, 1993)

The performance of a construction material can be considered as being influenced by a combination of internal factors, such as engineering characteristics, and external factors such as climate, construction methods and maintenance. Selection of materials cannot, therefore, be based solely on material character.

5.2 Selection and Assessment Criteria

5.2.1 Common Fill

The construction of a road is along an alignment; to designed gradients and curvature, commonly involves the adjustment of natural ground levels by cut and fills operations. In-situ rock and soil is removed to form cuttings and the excavated material, if it is acceptable, is used as material for subsequent embankments. The excavated material may not be enough for these purposes and material from the borrow areas may be required to provide the quantity of fill required. The lower pavement layers such as the capping layer are laid on the formed embankment upper surface.

The volumetric need for fill materials on a road project is largely governed by the requirements of the vertical alignment, either to cross undulating terrain or to raise the road pavement above potential flooding or other water ingress. Secondary requirements may include such factors as the need for cut-fill balance or reduction of environmental impact.

Because of the large volumes frequently required, fill materials are generally won from the nearest acceptable sources and in cases where alternatives are scarce there is an emphasis on adaptation of available material rather than source selection.

The basic quality and performance selection requirements for fill material can be considered as; placed stability; resistance to erosion, degradability and workability. Fill material acceptability is generally less rigidly controlled with respect to geotechnical parameters than pavement materials. Selection factors are summarised in Table 5.1. Differing projects are likely to have variable importance attached to the influencing factors.

In developed countries, predominantly in temperate climatic zones, fill specifications are commonly based on relationships involving moisture content, plasticity and undrained strength, although more recently the use of the moisture condition value (MCV) has become more widespread within the UK. The transfer of unmodified specifications for fill acceptability from temperate to tropical environments is not recommended and has been shown in many cases to be inappropriate (Wallace, 1973; Belloni, 1988 Moor and Styles, 1988)

Apart from the geotechnical difficulties, the selection of fill materials in developing countries has, in most instances, to be pragmatically based. Common fill has to be selected from available materials within an economic haul distance. Their selection should take into account the relative importance of specific project requirements and the technical and economic implications in relation to the available material sources. Potential fill materials that may be marginal in terms of standard suitability criteria but which are acceptable in economic and fundamental engineering terms need not be rejected out of hand. In general, the fundamental requirements of a fill material are that:

- It is placeable with available construction technology
- It is possible to achieve the target design strengths
- It is durable in terms of project requirements.

5.2.2 In Situ Sub-Grade

The sub-grade can be defined in terms of location as the upper 600mm of the road foundation. It is required to be able to resist repeated stressing from traffic loads without rupture and without large recoverable and irrecoverable strain throughout the design life of the road. In addition, this layer may have to support larger stresses during construction. Table 5.2 summarises the general requirements for a sub-grade material.

The character of in situ sub-grade material is determined by the geological and weathering conditions of the soil-rock profiles underlying the road. This means that sub-grade materials are likely to vary along the route, Figure 5.2. The suitability of the sub-grade material is a function of internal factors such as soil-rock type and its interaction with external factors such as climate and the local moisture regime, (Croney and Bulman, 1975)

ORN 31 summarises sub-grade requirements in terms of CBR strength classes and the relationship between prevailing moisture content and material plasticity.

5.2.3 Imported Capping Layer

Capping layer materials, sometimes referred to as imported sub-grade, are commonly required as a cover for weak or unsuitable in situ sub-grade or on top of a common fill embankments.

5.2.4 Filter-Drainage Material

Filter materials have crucial roles, in assisting in the prevention or in controlling the ingress of water and in the reduction of porewater pressures, within both the earthworks and the pavement. Filter materials can account for a significant proportion of the construction material costs, particularly in wetter regions where road designs need to cater for the dispersion of large volumes of water, both as external drains and as internal layers within wet-fill embankments.

The general requirements for filter material are a highly permeable mix comprising a durable aggregate that is resistant to chemical alteration, Table 5.3.

5.2.5 Unbound Granular Pavement Materials

Unbound granular material (UBGM) employed either as sub-base or base pavement layers has to perform a number of functions:

- Provide a working platform for construction
- Provide a structural layer for load spreading and protection of underlying layers
- Provide a layer with resistance to rutting
- Act as a drainage layer.

There may be conflict between these requirements, for example between a tightly graded material to take load and resist rutting and a more open textured material for good drainage. The main engineering properties that are required are summarised in Table 5.4. Unbound granular materials may be in the form of :

- Crushed stone aggregate (CSA)
- Natural sands and gravels (either "as-dug" or processed)
- Dry bound macadam (DBM)/wet bound macadam (WBM)

The internal factors governing the engineering performance of an unbound gravel aggregate are: the engineering behaviour and geometric properties of its constituent particles, its mass grading and the plasticity of its fines. ORN 31 defines the acceptability limits of these properties in its Tables 6.1 to 6.7

5.2.6 Bitumen Bound Granular Pavement and Surfacing Aggregate

The general requirements for aggregate to be used as a bitumen bound granular material (BBGM) aggregate are that it will be durable, strong and should also show good adhesion with bituminous binders. If aggregate is to be used in a surfacing layer, it should also be resistant both to the to the polishing and abrasion action of traffic. The main qualities for BBGM aggregate are summarised in Table 5.5 and defined in ORN 31 as follows:

- Resistance to slow crushing
- Resistance to rapid loading or impact
- Resistance to stripping, or to have the ability to maintain adhesion with the binder)
- Durability, or the resistance to in service weatherability.
- Hardness or resistance to abrasion or attrition (surfacing material only)
- Resistance to polishing (surfacing material only)

Adhesion failure implies a breakdown of the bonding forces between a stone aggregate and its coating of bituminous binder, leading to physical separation. Mechanical failure by fretting and subsequent raveling of the surface is one possible, but invariable, consequence of adhesion failure.

Basic rocks are considered to have better adhesion properties than acidic rocks. The comparatively poor performance of acid rocks may not only be related to the high silica content but to the formation of sodium, potassium and aluminium hydroxides. This is considered more likely in felspathic minerals, (Hughes at 1960). Experience has indicated, for example, that coarse granite with large feldspar inclusions is likely experience bitumen adhesion difficulties.

Apart from the petrological nature of the material its cleanliness or freedom from dust is also a factor. Limits of less that 1% dust <75 microns are difficult to obtain by screening alone and washing of the aggregate may be required

The resistance to abrasion is related to the petrological properties of the material: the proportion of hard minerals; the proportion and orientation of cleaved minerals; grain size; the nature of the interparticle bonding or cementation and the proportion of stable minerals resistant to weathering.

Resistance to polishing is considered a function of material fabric, texture and mineralogy. Rocks which contain minerals of differing hardness and which show a degree of friability tend to give high polishing resistance. Rocks that exhibit a moderate degree of decomposition give higher PSV results then fresh unweathered rocks. There is, therefore, an inverse relationship between polishing resistance and abrasion resistance.

5.3 Selection and Assessment Procedures

5.3.1 General Frameworks

Guidance on the selection process for key road construction materials is presented in the following figures

- Figure 5.3: Earthwork material
- Figure 5.4 Sub-Grade material
- Figure 5.5 UBGM Sub-Base aggregate
- Figure 5.6 UBGM and BBGM Base aggregate
- Figure 5.7 Surfacing aggregate

These figures together with associated tables, present a logical framework for the selection and utilisation of available construction material reserves. The framework is built around a number of key steps:

- Assess the design requirements
- Evaluate the options available
- Utilise the available material in as cost-effective a means as possible

Important aspects of the selection process are the identification of potential problems, the assessment of the impacts that these might have, and the selection of the appropriate solutions. In this regard the recognition of the external influences of the road environment and the importance of historical performance information are highlighted within the framework.

The selection framework identifies three principle scenarios where, for engineering reasons, selection of a material is not immediately recommended:

- The material has been used successfully but within a different road environment
- The material meets the selection criteria, but has not previously been used successfully
- The material fails the selection criteria

In all the above cases, provided that there is no immediately available and acceptable alternative, it is recommended that a further evaluation be undertaken of the environment, geotechnical and engineering failure criteria. This should assess, whether the identified impacts are either negligible, can be overcome, or are such as to cause the material to be rejected or downgraded.

Table 5.6 presents a summary of the road environment factors that are likely to influence the use of material in road construction embankment.

5.4 Road Environment and Geotechnical Impacts

5.4.1 Earthwork Embankment Materials

Changes in the environment can significantly affect the suitability of an embankment material in terms of its excavatability, placeabilty, compaction or final performance.

The selection of a material without a previous record of successful performance as an earthwork material can involve significant risk, particularly within tropical areas. The geotechnical character and hence the potential engineering behaviour of earthwork materials within tropical regions is dominated by its in situ weathering condition at the material and mass scales. In the majority of cases material is likely to be won from a parent material that been weathered in situ into a soil-rock profile with marked vertical variability. Chemically dominated weathering can produce geotechnical behaviour at the material scale that may be very different from temperate zone materials for which the majority of design specifications and working practices for fill were built on. The majority of intrinsic material problems are associated with mineralogy and fabric. Table 5.7 summarises some of their geotechnical implications for embankment materials.

5.4.2 In Situ Sub-Grade

The road environment factors that are likely to influence the performance of an in situ sub-grade are included within Table 5.6. In the majority of cases in situ sub-grade material is likely to have been tropically weathered and as with embankment fills the majority of intrinsic material problems are associated with mineralogy and fabric, Table 5.7.

5.4.3 Unbound Granular Pavement Aggregate

The road environment factors that are likely to influence the performance of an UBGM are included within Table 5.6. Table 5. 8 summarises geotechnical considerations.

Failure of aggregate within unbound pavement layers has been frequently associated with the breakdown of rock already influenced to at least some degree by chemical weathering. It is now recognised that it is the secondary minerals formed from the alteration of primary minerals that are the principal causes of distress. Weinert (1968), for example, indicated a 30% threshold for alteration products, particularly within basic igneous rocks.

The crushing processes may induce or enhance fractures within an aggregate that allow ingress of water to enhance later weathering. Screening and washing processes can add to the degradation process by introducing wetting, drying and abrasion cycles. Stockpiling can allow wetting - drying cycles to impact on the material and induce an accelerated degradation.

5.5 Options for Material Utilisation

5.5.1 Common Fill

Options for the mitigation of negative impacts on fill materials are summarise on Table 5.9. The appropriate adoption of one or more of these procedures could allow the use of material that would otherwise have been rejected. Some of the options themselves have implications and consequence for fill performance that need to be evaluated before being adopted for a particular material. The following notes expand upon aspects of the impact and their mitigation options.

- The principal design options for the use of poorer grade fill materials are to flatten embankment side-slopes; incorporate internal filter layers (for wet material), and to adopt a zoned embankment design to contain weaker material.
- There may be limits to the rate at which a soil fill may be placed without incurring undrained stability problems, hence the use of internal filter layers.
- Flattening of side-slopes may result in problems with respect to erosion and land-take
- Highly plastic clays are potentially subject to volume changes and it would normally be prudent to use a capping layer of less plastic material below the pavement layers.
- Some residual soils materials have a higher intrinsic permeability than is usual for conventional sedimentary clays and even clayey fills may have significant vertical drainage characteristics. The use of oedometer tests on as-compacted fill is recommended in order to give realistic Cv values.
- One of the major limitations to the utilisation of marginal wet fill is its ability to be run-on by construction plant. Table 5.10 presents some reported guidelines on plant usage.

- Wet fill that cannot be run-on by pneumatic tyred plant may still be used, provided appropriate procedures are selected, for example, by excavation with tracked shovel, utilising trucks running on haul roads of stronger fill and spreading by low-contact dozers.

- Potential changes in fabric and texture can occur under compaction. Over-working during compaction may in some materials lead to deterioration of density-strength characteristics. /

- Experience from tropical environments indicates that between 2% and 5% per day of moisture can be lost during material loading, hauling and spreading during dry days

- When using a residual soil-rock profile it is very likely that as fill excavation proceeds downwards the character of the material will change significantly and compaction requirements will change accordingly.

- In a seasonally or diurnally wet climate there are significant implications with respect to having to use flexible earth-moving and placement procedures that can deal effectively with alternating wet and dry conditions.

- In residual soil profiles the likelihood of significant variation in particle density has implications for the calibration of monitoring programmes by placement density methods.

- In highly plastic clays the use of smooth-drum rollers carries with it a danger of the fill polishing with the result of low strength shear planes being formed.

- Poor quality rock-fill, particularly argillaceous material, has the potential for in service deterioration and settlement unless appropriate densification procedures are adopted during construction.

5.5.2 In Situ Sub-Grade

Options to deal with sub-grade that falls below acceptable design criteria are summarised on Table 5.11. The appropriate adoption of one or more of these procedures could allow the use of material that would otherwise have been rejected. The following notes expand upon aspects of the impact and their mitigation options.

- The design thickness and the strength of the sub-grade should be assesses for the material at its weakest, i.e. at a moisture content of the soil at its wettest

- The process of improvement of embankment materials due to consolidation is not as effective within the sub-grade owing to the low stress levels within it.

- There is an increased risk of deterioration owing to an in service increase of moisture content It is therefore essential that sub-grades are protected from water by design measures such as drainage and waterproofing the pavement and it's shoulders

- Where the remoulded strength is likely to be less than 70 kPa it is prudent to exclude wheeled plant from final cut-surfaces. Alternatively it would be possible to over-excavate and place a capping layer of higher strength material.

- In its undisturbed in situ condition a sub-grade material may have a relict fabric or a mineral chemistry that is susceptible to collapse, dispersion or erosion. Full-scale trials might be necessary to identify suspect areas that may have to be subjected to pre-loading and/or pre-soaking.

- Some in situ sub-grades may be prone to swell due to combinations of mineralogy, stress relief and water ingress. Dig-out and replacement with a capping layer and/or the use of geotextile water exclusion measures may be used to counteract this problem.

- Chemical stabilisation of weak, degradable or swelling sub-grade material is an option that is discussed more fully in Chapter 8.

5.5.3 Unbound Granular Pavement Aggregate

Options to deal with UBGM that does not immediately meet design criteria are summarised on Table 5.12 .The adoption of one or more of these procedures could allow the use of material that would otherwise have been rejected or downgraded. The following notes expand upon aspects of the impact and their mitigation options.

- It is possible to use low crushing strength aggregates or those with flaky or elongated shape provided that suitably dense grading are designed, Lees & Zakaria (1987).

- Degradation of aggregates during construction can have beneficial densification effects. In order for this degradation to be useful it must occur during the construction process as - degradation under traffic tends to produce rutting.

- Dense grading can however become saturated more easily and thus enabling the setting-up of transient pore pressures under traffic loading, Brown & Selig, (1991).

- As in all cases where lower quality aggregate are being considered a careful programme of field trials should be carried out.

- Chemical stabilisation of out-of-specification sub-base or base materials is an option that is discussed more fully in Chapter 8.

Key Engineering Factor	Material Requirement	Investigation Requirements	Site Controlling Factors
Stability	The material when placed and suitably compacted must be capable of standing at the appropriate designed slope angles, both in the short and long term.	Shear strength - unconsolidated undrained and consolidated drained at the relevant moisture-density relationship.	Placement and compaction control. Foundation conditions.
Erosion Resistance	The material when placed in the embankment, particularly at or close to slope faces, must be capable of resisting erosion by exposure to rainfall, both directly and indirectly by surface run-off. Internal erosion must be resisted by using non-dispersive soil. The long term effects of alternate wetting and drying (slaking) must also be resisted.	Grading and plasticity; fabric and mineralogy assessment. Possible use of soil dispersion and erosion tests	Compaction control at the embankment edges. Drainage and surface protection of earthworks both during and after construction.
Resistance to Degradation	Rock fill or soil-rock fills must be capable of resisting in-service degradation which could result in internal erosion, settlement or collapse.	Fabric and mineralogy assessment. Slake durability testing	Internal filters. Appropriate compaction control.
Haul Distance	Potential material must be within physically and economically feasible haulage distance.	Mass-haul analysis in conjunction with cut-fill earthwork balance. This may dictate options for fill sources in economic terms.	Haul road conditions. Available plant
Placeability	The material must be capable of being economically placed and compacted in the embankment by the selected project earth moving equipment. In doing this the material must be also capable of being run-on by the selected project plant	Unconsolidated undrained shear strength in the totally destructured condition (to model construction plant movement).	Compactability - in relation to available project compacting plant. Traffickability requirements economic use of construction plant.
Environmental Impact	Material reserves must be capable of being won and hauled within governing environmental impact regulations. Constraints may impose limitations on the amount, size and location fill borrow pits and on the transportation of material.	Environmental impact study..	Location of reserves. Working and restoration practices.

Table 5.1 General Embankment Fill Requirements

Key Engineering Factor	Material Requirements	Investigation Requirements	Site Controlling Factors
Strength	Aggregate particles need to be load resistant to any loads imposed during construction and the design life of the pavement.	Aggregate impact and strength testing.	Material excavation and processing. Site quaility control.
Mechanical Stability	The aggregate as a placed layer must have a mass mechanical interlocking stability sufficient to resist loads imposed during construction and the design life of the pavement.	Particle size distribution. Particle shape. CBR at appropriate density and moisture condition.	Material excavation and processing. Transportation (segregation). Construction traffic Site quaility control.
Durability	Aggregate particles need to be resistant mineralogical change and to physical breakdown due to any wetting and drying cycles imposed during construction or pavement design life	Aggregate durability test. Petrographic examination.	Material processing and stockpiling.
Haul Distance	Reserves must be within physically and economically feasible haulage distance.	Mass-haul analysis. Examine potential for alternative use of sub-standard sub-base reserves.	Haul road condition. Plant availability.
Placeability	Material must be capable of being placed and compacted by the selected project plant.	Trafficability. Unconsolidated-undrained shear strength in totally destructured condition.	Site compaction methodology. Site quality control.
Environmental Impact	Material reserves must be capable of being won and hauled within any governing environmental impact regulations.	Environmental impact study.	Location of reserves. Site working and restoration practices. Environmental management.

Table 5.2 General Sub-Grade and Imported Capping Layer Material Requirements

Key Engineering Factor	Material Requirement	Investigation Requirements	Site Controlling Factors
Permeability	The fundamental filter property, primarily a function of material grading. Generally desirable for filter aggregates to be equi-dimensional as this aids flow distribution and facilitates packing. It is also considered better to use material with rounded to sub-rounded rather then angular particles	Particle size distribution. A wide range of sizes may be specified and used but an important requirement is <10% fines and no-plasticity. Particle shape examination.	Site quality control on segregation. Clogging of filter material by construction waste.
Strength	Aggregate particles need to be load resistant to abrasion and any loads imposed by the road design	Aggregate strength/abrasion tests. Porosity, which can indirectly effect strength and durability.	Protection of filter material after placement.
Resistance to Degradation	Aggregate particles need to be resistant to breakdown due to wetting and drying and weathering during construction and for the life of the project.	Aggregate durability tests. Petrographic examination	Protection during construction
Resistance to Erosion	The as-placed material must be resistant to internal and external ersoion.	Erosion and dispersion tests.	Appropriate filter design or utilisation of geotextiles.
Chemical Stability	Aggregate should generally be inert and resistant to alteration by groundwater. Weak surface coatings such as clay, iron oxide, calcium carbonate, gypsum etc are on the whole undesirable	Chemical tests of aggreagte and groundwater. Petrographic and mineralogical examination	Contamination of groundwater or surface water by substances that may react with filter material.

Table 5.3 General Filter Material Requirements

Key Engineering Factor	Material Requirements	Investigation Requirements	Site Controlling Factors
Strength	Aggregate particles need to be load resistant to any loads imposed during construction and the design life of the pavement.	Aggregate impact and strength testing.	Material excavation and processing. Site quality control.
Mechanical Stability	The aggregate as a placed layer must have a mass mechanical interlocking stability sufficient to resist loads imposed during construction and the design life of the pavement.	Particle size distribution. Particle shape. Mass strength (CBR) at appropriate density and moisture condition.	Material excavation and processing. Transportation (segregation). Construction traffic Site quaility control
Durability	Aggregate particles need to be resistant mineralogical change and to physical breakdown due to any wetting and drying cycles imposed during construction or pavement design life.	Aggregate durability test. Petrographic examination.	Material processing and stockpiling.
Haul Distance	Reserves must be within physically and economically feasible haulage distance.	Mass-haul analysis. Examine potential for alternative use of sub-standard sub-base reserves.	Haul road condition. Plant availability
Placeability	Material must be capable of being placed and compacted by the selected project plant.	Trafficability. Particle size distribution.	Site compaction methodology. Site quality control
Environmental Impact	Material reserves must be capable of being won and hauled within governing environmental impact regulations.	Environmental impact study.	Location of reserves. Site working and restoration practices. Environmental management.

Table 5.4 General UBG Sub-Base and Base Requirements

Key Engineering Factor t	Material Requirement	Investigation Requirements	Site Controlling Factors
Strength	Aggregate particles need to be load resistant to any loads and abrasion imposed during construction and the design life of the pavement.	Aggregate impact and strength testing. Particle shape.	Material excavation and processing. Site quality control.
Durability	Aggregate particles need to be resistant mineralogical change and to physical breakdown due to any wetting and drying cycles and abrasion imposed during construction or pavement design life	Aggregate durability tests. Aggregate abrasion tests. Petrographic examination.	Material processing and stockpiling
Skid Resistance (Surface aggregate only)	Aggregate particles must be resistant to polishing.	Polishing tests. Petrographic examination.	Material processing and stockpiling. Construction impacts.
Adhesiveness	Aggregate must be capable of adhesion to bitumen and sustaining that adhesion for its design life.	Bitumen stripping tests. Petrographic examination	Construction methodology. Climate.
Haul Distance	Reserves must be within physically and economically feasible haulage distance	Mass-haul analysis. Examine potential for alternative use of sub-standard sub-base reserves.	Haul road condition. Plant availability
Environmental Impact	Material reserves must be capable of being won and hauled within governing environmental impact regulations.	Environmental impact study	Location of reserves. Site working and restoration practices. Environmental management.

Table 5.5 General Bitumen Bound and Surfacing Aggregate Material Requirements

Road Environment Factors	Details to be Evaluated	Impact of Environment on Material Selection
Climate	Wet climates : Seasonal and diurnal variations, rainfall intensity, evapotranspiration	All materials need to be resistant to wet working conditions and selected on soaked characteristrtics and resistant to dispersion. Inhibited drying of **soil fill** in particular may require selective use of material for haul roads. Exposed materials need to be resistant to erosion. Soaked **sub-grade** with potential for volume change (swell or collapse). High run-off and groundwater flow needs to be catered for by **filter** materials. Potential for weathering or in service deterioration of **UBG** and **BBG** aggregate
	Dry climates. Rainfall occurrences (storm return periods) temperature variation.	Materials in general should be as free from potential dust as possible. Wetting-up of dry materials for compaction purposes may induce shrinkage cracking in materials with highly active clays. **Sub-grad**e and **UBG** materials may be subject to changes in moisture condition in service due to hydrogenlsis in some climatic environments, hence selection on dry acceptability criteria may be misleading. **Surface aggregate** performance can be adversely influenced by dust contamination.
Hydrology	Surface water run-off and drainage patterns.	Impacts on **fill** in terms of drainage requirements. In extreme cases where flooding is a possibility there may be a requirement for free-draining material or even rock-fill and for slope face protection. **Filter** materials must be of high quality.
	Seasonal groundwater changes	High water table may give rise to poor working conditions and weak foundations requiring flatter embankment slopes and greater amounts of **fill** material. Significant amounts of **filter** materials may be required for drainage blankets. **Sub-grade** materials may be liable to soaking due to capillary rise.
Topography	High relative relief and steep terrain patterns relative to route alignment.	Higher strength **fill** materials may be required for higher embankments. Side-slopes may also need to be steeper to meet geometric constraints. Internal **drainage** layer materials may be required for higher embankments. In situ **sub-grade** conditions likely to be subject to rapid change and hence a requirement for **capping layer** materials. Terrain geometry may severely restrict separate haul road development. **Sub-grade** and **sub-base** materials must be selected with construction traffic impacts in mind.
	Low relative relief and flat terrain patterns relative to route alignment.	Likely to less constraint on embankment side slopes and hence possible use of weaker **fill** materials More uniform **sub-grade** conditions. Greater scope for separate haul roads and less construction impacts on **sub-grade and sub-base** materials.

Table 5.6 Road Environment Factors Impacting on Road Construction Materials

Road Environment Factors	Details to be Evaluated	Impact of Environment on Material Selection
Excavation and Processing	Excavation and transport methods	Impacts may alter the physical character and moisture condition of materials. Residual and fabric-sensitive **soil fill** character needs to be assessed for suitability in its altered state. **Filter** materials and **UBG** and **BBG** materials may segregate during transport.
	Crushing and sizing operations.	**Granular** materials outwith specified grading can be improved by sizing operations. Although at additional extra cost. Crushing operations may have significant impact on shape of **granular aggregate.** Inherently poorly shaped material may be improved by well conducted crushing operations
	Stockpiling	Stockpiling may alter physical character and moisture condition of materials, through segregation and exposure to rain and sun. **Argillaceous** materials such as mudstones and shales may slake considerably if stockpiled. **UBG** and **BBG** materials, particularly already slightly weathered basic igneous materials may degrade during stockpiling.
Engineering Context	Road section in embankment	**Capping layer** materials may be required in addition to common **fill.** Significant internal and external **drainage/filter** materials also a likely requirement. All embankment and **UBG** layers may have to withstand construction traffic impacts.
	Road section in cut	Possible requirement to incorporate excavated materials form cut into adjacent embankments. In situ **Sub-grade** conditions may vary from unweathered rock to soil. Potential stress release or moisture induced swell in base of excavated cuts may necessitate special measures including imported **capping laye**r
	Road section at grade	Possible need for shallow embankment materials to raise pavement layers above surface water run-off or capillary rise zone.
	Road section in mixed cut and embankment	Significant requirement for drainage materials to counter erosion and seepage at cut-fill interface. Likely requirements for good quality fill material in this potentially high hazard zone. Variable **sub-grade** conditions in cross section. . Possible requirement to incorporate excavated cut material into fills.
Construction Method	Compaction plant and procedures.	Compaction plant, compaction procedures and selected material must be compatible. Materials sensitive to remoulding (eg structured fill materials) can be made unsuitable by inappropriate plant operations. I
	Haul road and temporary works layouts	Can have significant impacts on performance of materials. Embankment and pavement layer materials likely to be subjected to these impacts need to be closely monitored for deterioration.
	Quality control procedures	Required at borrow pits and quarries to ensure selection criteria are adhered to.
Road Maintenance Programme	Good maintenance guaranteed likely to continue through design life of road	An environment where non-standard and cheaper materials may be selected and used within definable limits.
	Effective maintenance unlikely.	Likely to require a higher factor of safety built into standards of material selected

Table 5.6 Road Environment Factors Impacting on Road Construction Materials (Cont'd)

Material Characteristic	Key Aspects	Geotechnical Consequence	Potential Negative Impacts on Earthworks
Mineralogy	Tropical soil profiles (eg Tropical Red Soils) can include minerals with non-standard characteristics such as halloysite or allophane.	Moisture retention character is such that care is needed in the interpretation of laboratory material performance in relation to field behaviour. Allophane in particular will also aggregate to silt or fine sand sized material on drying-out. Densification characteristics may not be directly transferable form the laboratory to the field.	Inappropriate suitability specifications and acceptability standards. Unattainable field density/compaction.
	Tropical soil profiles (eg black cotton soils) may contain significant amounts of swelling clay minerals such as smectite.	Materials with an unpredicted high swell potential can have severe placeability and performance difficulties	High swell potential Rutting
	Soils with a high Exchangeable Sodium Percentage (ESP) - partculuarly argillaceous materials - in conjunction with reactive groundwater fluids	Can lead to a soil with a high dispersion potential, ie erodable even in non-flowing water conditions.	Serious long-term stability (piping) and erosion problems
Fabric	Tropically highly weathered in situ materials commonly retain a relict fabric inherited from their original parent material. A highly weathered soil from an igneous parent may retain an apparently crystalline fabric whose particles have, however, been replaced by secondary clay minerals.	The relict fabric can very easily break down and remould from a visible particle fabric to a wet clay during excavation, placement and compaction.	Wetter fill than anticipated. Poor haul roads; poor production rates. Short term instability of fills.
	Completely weathered residual soil materials can develop a new-formed fissure-block fabric that may be held together by weak bonding.	Bonding is easily broken down during excavation, placement and compaction of fills and during service life of cut faces..	Wetter fill than anticipated Slaking and erosion of fill and cut-slope faces.
	Soil-rock weathering profiles in suitable stable geological environments can develop pedogenic layers (eg calcrete, laterite)	Cemented concretions and layers can occur within soil horizons.	Potential excavation problems unless anticipated. Oversize materials placed on road.

Table 5.7 Tropical Zone Geotechnical Characteristics Influencing Earthwork Embankment Materials

Material Characteristics	Key Aspects	Geotechnical Implications	Potential Negative Impacts on Pavement Aggregate
Mineralogy	Inclusion of significant amounts of mafic minerals (eg basic igneous rocks).	Susceptability to tropical weathering and formation of secondary minerals	Poor durability. In service deterioration. Plastic fines.
	Inclusion of significant amounts of platey minerals (eg regional metamorphic rocks)	Anistotropic strength characteristics. Release of mica during processing.	Poor aggregate shape. Weakened aggregate. Compaction problems
Fabric	Foliation or lamination.	Anistotropic strength characteristics.	Poor aggregate shape.
	Increased void ratio (eg as a consequence of weathering).	Higher water absorption. Lower strength.	Low aggregate strength.
	Strained crystal fabric (eg metamorphic quartz).	Stress release fractures at the micro-scale.	Increased fines.
	Porphyritic crystal fabric (eg some granites).	Irregular grading on break-down of rock during processing.	Gap graded fine aggregate

Table 5.8 Intrinsic Geotechnical Factors Influencing Unbound Pavement Materials

Material Characteristics	Key Aspects	Geotechnical Implications	Potential Negative Impacts on Pavement Aggregate
Mineralogy	Inclusion of significant amounts of mafic minerals (eg basic igneous rocks).	Susceptability to tropical weathering and formation of secondary minerals.	Poor durability. In service deterioration Plastic fines
	Inclusion of significant amounts of platey minerals (eg regional metamorphic rocks)	Anistotropic strength characteristics. Release of mica during processing.	Poor aggregate shape. Weakened aggregate
	High percentage of large feldspar minerals (eg coarse granites)	Exposed free smooth crystal surfaces. High Na ion potential.	Poor bitumen adhesion
Fabric	Foliation or lamination	Anistotropic strength characteristics.	Poor aggregate shape. Poor bitumen adhesion.
	Increased void ratio (consequence of weathering)	Higher water absorption. Lower strength.	Low aggregate strength. High bitumen absorption.
	Strained crystal fabric (e.g. metamorphic quartz)	Stress release fractures at the micro-scale.	Increased fines.
	Porphyritic crystal fabric (e.g. some granites)	Irregular grading on break-down of rock during processing.	Gap graded fine aggregate

Table 5.9 Intrinsic Geotechnical Factors Influencing Bitumen Bound Pavement Materials

Trafficability	Quick-Undrained Strength:- Completely Remoulded (kPa)
Can be run on by low contact-pressure dozer	<15
Can be run on by standard dozer	15 to 30
Just passable by wheeled plant with very deep ruts: unsuitable for haul roads.	30 to 45
Passable by wheeled plant with severe rutting; poor haul roads possible	45 to 60
Reasonable running surface, some rutting	60 to 100
Good running surface	>100

Table 5.10 Trafficability Related to Shear Strength

Problems	Potential General Solutions																				
	Modify Sideslopes	Include drainage layers	Include sub-surface drainage	Use water sprayers	Drainage in temporary works	Local re-alignment	Key-in earthwork fill	Modify plant utilisation	Use method specifications	***Modify laboratory procedures***	***Undertake trial compactions***	Low contact pressure plant	Special haul road measures	Water exclusion measure	Pre-soak and collapse	***Road trials***	Seek specialist advice	Temporay fill protection	***Use of fill in non critical areas***	Immediate protection of sidelopes	Add capping layer
Wet working conditions	1	1	1		1	0	0	1	0	0	0	1	1	2	0	0	0	2	0	2	1
Excessively dry working conditions	0			2	0	0	0	1	0	0	0	0	1		0	0	0	0	0	0	0
High water table	1	1	2		1	0	0	0	0	0	0	0	0	1	0	0	0	0	1	0	1
Weak foundations	2	1	2		0	1	1	0	0	0	0	2	1	0	2	1	1	0	1	0	0
Steep cross-section	2	0	2		0	2	2	0	0	0	0	0	1	1	1	0	0	0	2	1	0
Land-take constraints	2	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Construction time constraints	0			0		0	0	1	1	0	1	0	0	1				0	0	0	
Poor production rates	0	0	0	1	1	0	0	2	1	0	1	1	0	1	0	1	0	0	0	0	0
Unattainable specification	0	0	0	1	0	0	0	1	2	2	2	1	0	0	0	2	2	0	1	0	2
Unattainable field density/compaction	0	0	0	1	0	0	0	1	2	2	2	1	0	0	0	2	2	0	1	0	0
Excessive rutting	0	1	0		0	0	0	1	1	0	1	2	1	1	1	1	0	0	2	0	2
Swell potential	0	2	1		0	1	0	0	0	0	0	0	1	2	1	2	1	1	2	0	2
Poor haul road condition	0	0	0	1	2	0	0	2	0	0	0	0	2	1	0	0	0	0	0	0	1
Very wet fill	1	2	1		1	0	0	1	0	0	1	1	1	1	0	1	1	0	1	0	1
Collapsing soil	0	0	0	0	0	0	0	0	0	1	0	0	0	0	2	2	2	0	0	0	0
Dispersive soil	0	0	0		0	0	0	0	0	1	0	0	0	0	0	2	2	2	2	2	0

Notes 0 Not relevant 1 Some possible benefit 2 A likely solution

Bold Italics - Likely to have consequent effects that will require assessment Detrimental effects

Table 5.11 General Options for Mitigation of Earthwork Material Problems

Problems	Include drainage	Water exclusion measures	Use Water spayers	Local re-alignment	Modify plant usage	Use method specification	Pre-soak and collapse	Temporay works protection	Separate haul roads	***Modify laboratory procedures***	***Road trials***	Chemical stabilisation	Seek specialist advice
	Potential General Solutions												
Wet working conditions	2	2		1	2	0		2	1	0	0	0	0
Excessively dry working conditions	0		2	0	2	1	1	0	1	2	1	0	1
High water table	1	2		0	1	0	0	0	0	0	0	0	0
Weak foundations	0	1		2	1	0	0	0	0	0	0	1	0
Time constraints				0	1	0			0	0			0
Unattainable specification	0	0	1	0	2	2	0	0	0	1	2	2	2
Below specification sub-grade	0	0	0	2	1	0	0	0	0	2	2	2	2
Dispersive soil	0	1	0	1	2	0		0	0	1	1	1	2
Swelling soil	1	2	0	1	1	1		1	0	1	1	2	2
Erodable soil	0	1		1	1	0		0	1	1	0	1	1
Collapsable soil	0	1	1	1	2	0	2	0	0	2	2	0	1

Notes 0 Not relevant 1 Some possible benefit
2 A likely solution Detrimental effects
Bold Italics Likely to have consequent effects that will require assessment

Table 5.12 General Options for Mitigation of In Situ Sub-Grade Problems

Problems	Potential General Solutions												
	Modify design (e.g. sealed shoulders)	Review stockpiling procedures	Modify screening and washing procedures	Modify crushing procedures	Modify plant usage	Use method specification	Chemical stabilisation	Review specification (e.g. grading)	***Specialist petrological testing***	***Compaction trials***	***Modify laboratory procedures***	***Road trials***	Seek specialist advice
Wet environment	2	1	0	0	1	0	0	0	0	0	0	0	0
Variable "at site" quality	0	2	1	1	0	0	1	0	0	0	0	0	0
In service deterioration	1	2	2	2	0	0	1	0	1	0	0	0	0
Unattainable specification	0	0	0	0	1	2	2	2	0	2	2	1	1
Compaction problems	0	0	0	0	2	2	1	2	0	2	1	1	1
Non plastic fines	1	0	0	0	0	0	0	1	0	2	0	2	1
High plastic fines	1	1	2	0	0	0	2	1	2	2	1	1	0
Poor shape	0	0	1	2	0	0	0	2	0	1	0	1	1
Low strength (CBR)	1	1	1	2	0	0	2	2	1	0	1	2	0
Poor grading	1	1	2	1	0	0	0	2	0	2	1	1	0
Oversize inclusions	0	1	2	2	2	0	0	0	0	0	0	1	0

Notes
0 Not relevant
1 Some possible benefit
2 A likely solution
[grey box] Detrimental effects

Bold Italics Likely to have consequent effects that will require assessment

Table 5.13 General Options for Mitigation of UBGM Pavement Problems

Problems	Modify design	Review stockpiling procedures	Modify screening and washing procedures	Modify crushing procedures	Modify plant usage	Chemical additives	Review specification (e.g. grading)	***Specialist petrological testing***	***Modify laboratory procedures***	***Road trials***	Seek specialist advice
	Potential General Solutions										
Wet environment	2	1	0	0	1	1	0	0	0	0	0
Variable "at site" quality	0	2	1	1	0	1	0	0	0	0	0
In service deterioration	1	2	2	2	0	1	0	1	0	0	0
Dust/Fines	0	2	2	1	1	0	0	0	0	1	1
High plastic fines	1	1	2	0	0	2	1	2	1	1	0
Poor shape	1	0	1	2	0	0	2	0	0	1	1
Low strength	1	1	1	2	0	2	2	1	1	2	0
Poor grading	1	1	2	1	0	0	2	0	1	1	0
Poor adhesion	0	0	0	1	0	2	0	2	1	2	2

Notes

0 Not relevant

1 Some possible benefit

2 A likely solution

(grey box) Detrimental effects

Bold Italics Likely to have consequent effects that will require assessment

Table 5.14 General Options for Mitigation of Surfacing Aggregate Problems

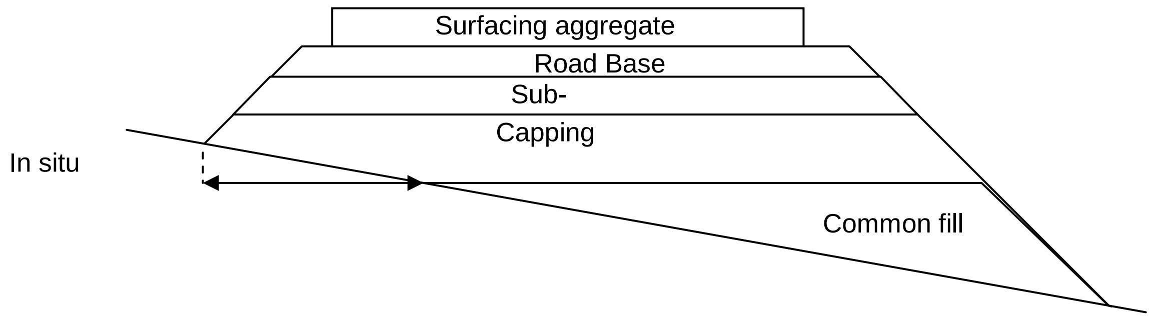
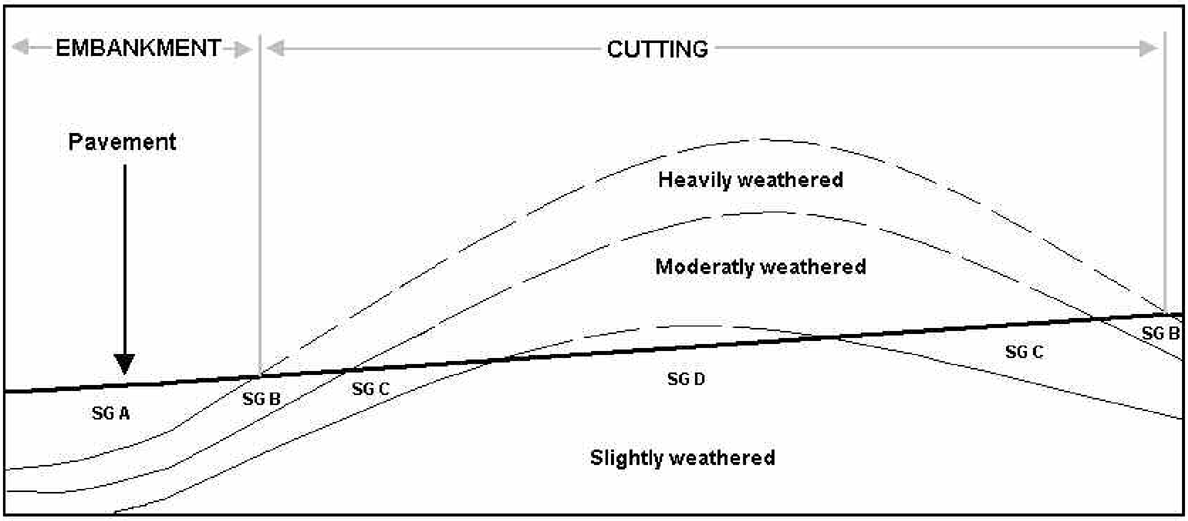


Figure 5.1 Definition of Road Construction Materials



Notes:

SG A: Sub-Grade on embankment
SG B: Sub-Grade on highly weathered, friable soil
SG C: Sub-Grade on moderately weathered very weak rock
SG D: Sub-Grade on slightly weathered strong rock

Because of permeability difference the boundary between highly weathered material and moderately weathered materials frequently becomes saturated and can cause significant problems to the sub-grade in that area.

Figure 5.2 Typical Variation in Sub-Grade Conditions in Tropical Hilly Terrain (Long Section through a cutting)

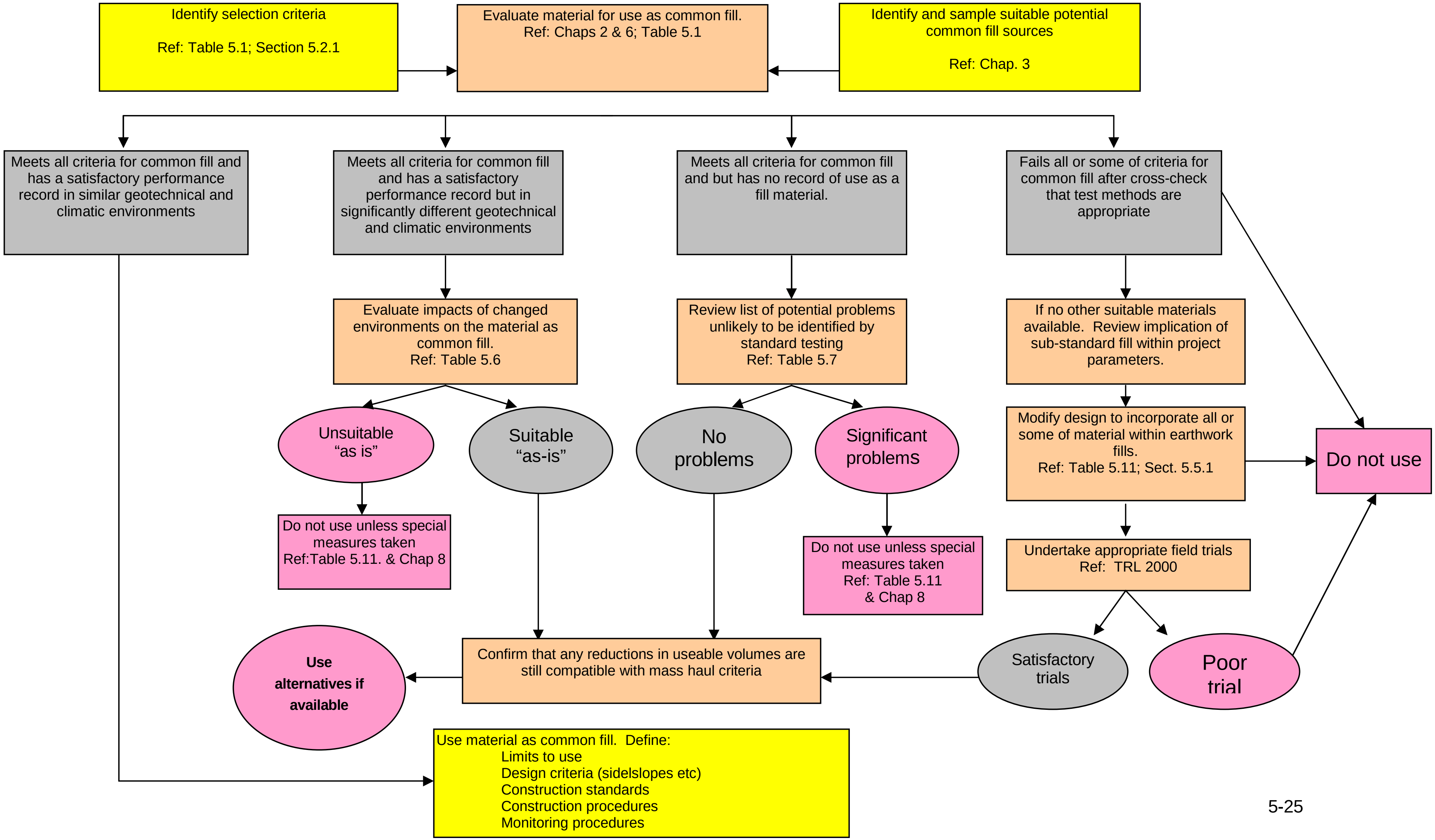


FIGURE 5.3 EARTHWORK FILL SELECTION FRAMEWORK

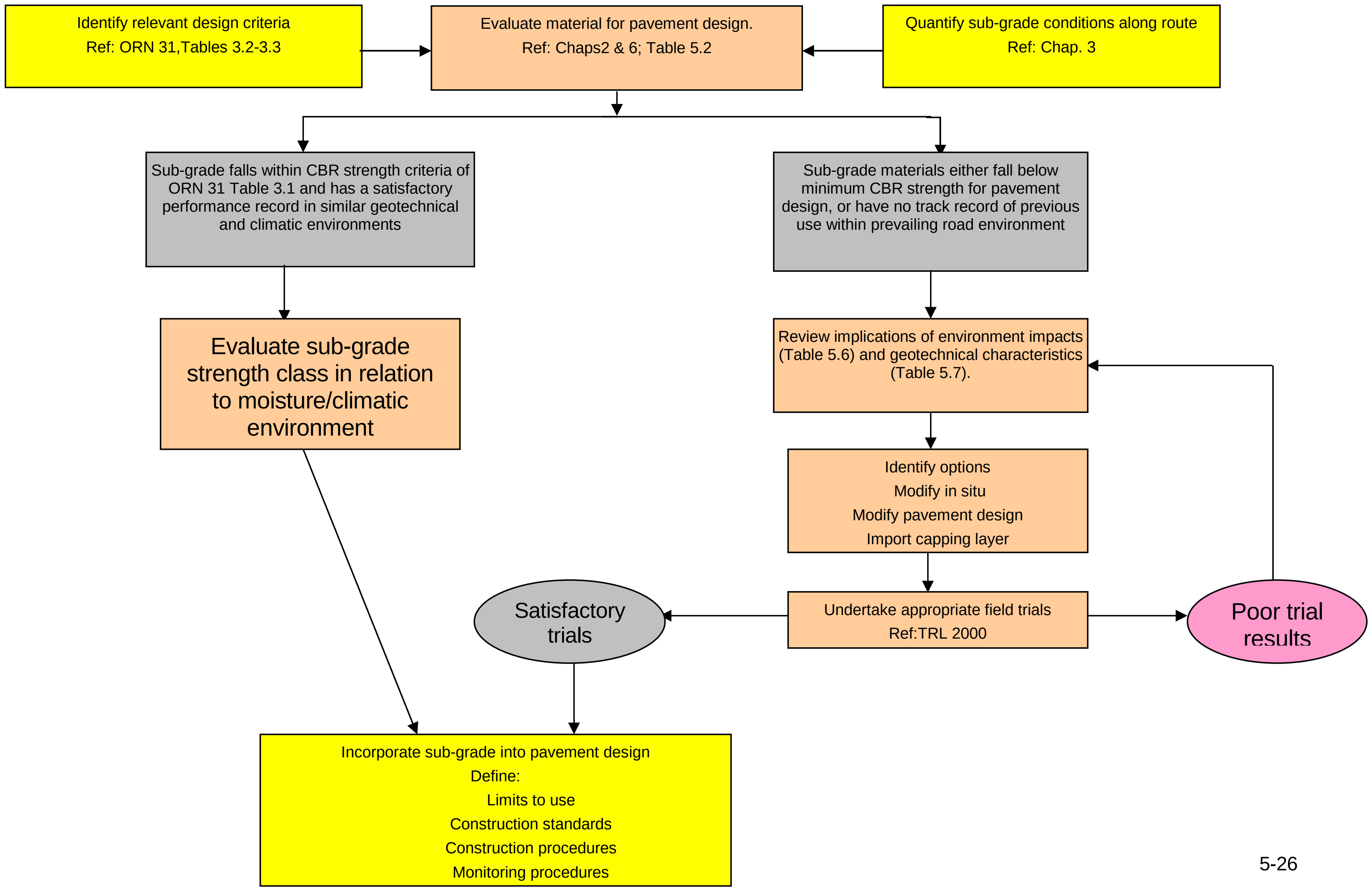


FIGURE 5.4 SUB-GRADE ASSESSMENT FRAMEWORK

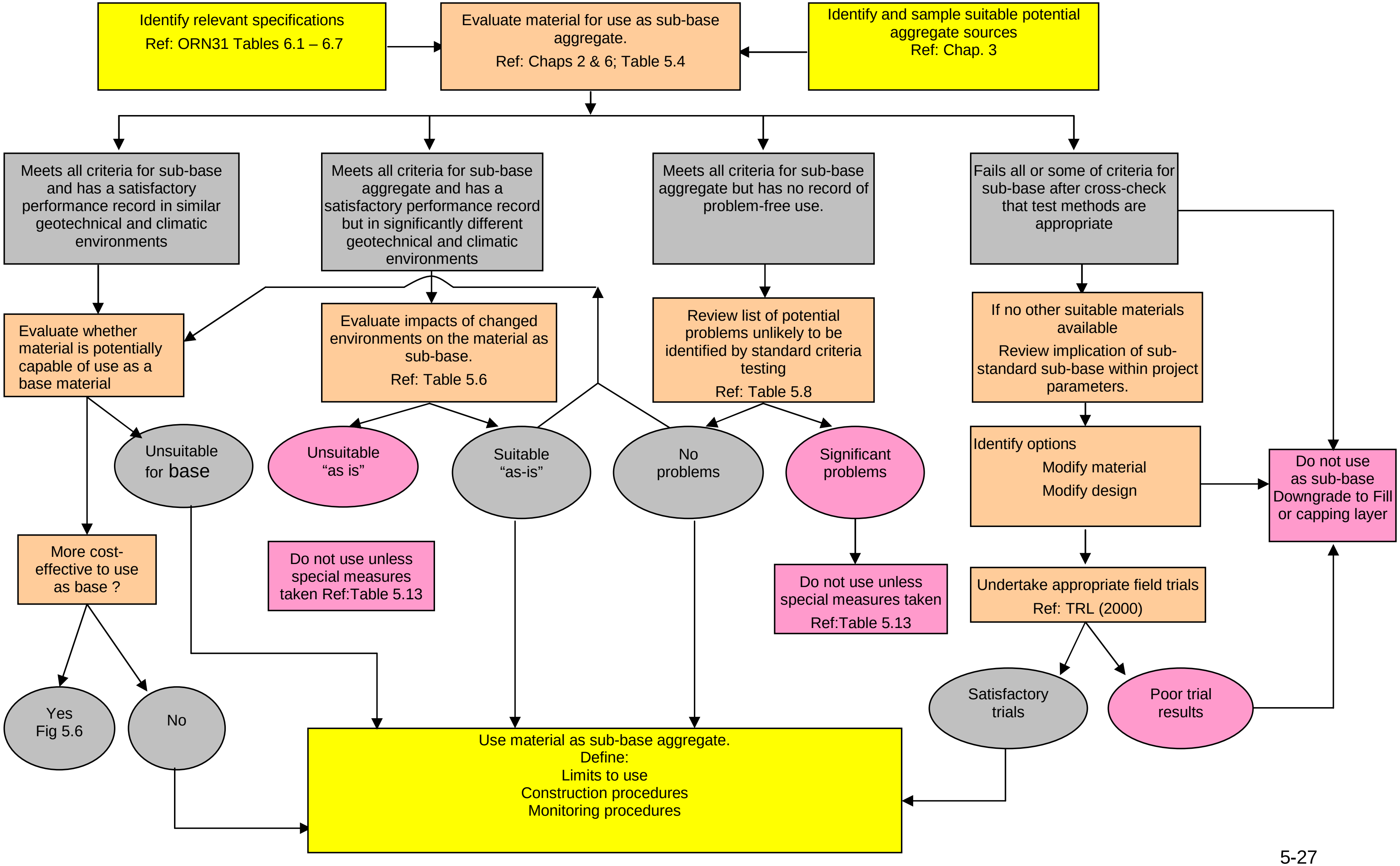


FIGURE 55 UBGM SUB-BASE AGGREGATE SELECTION FRAMEWORK

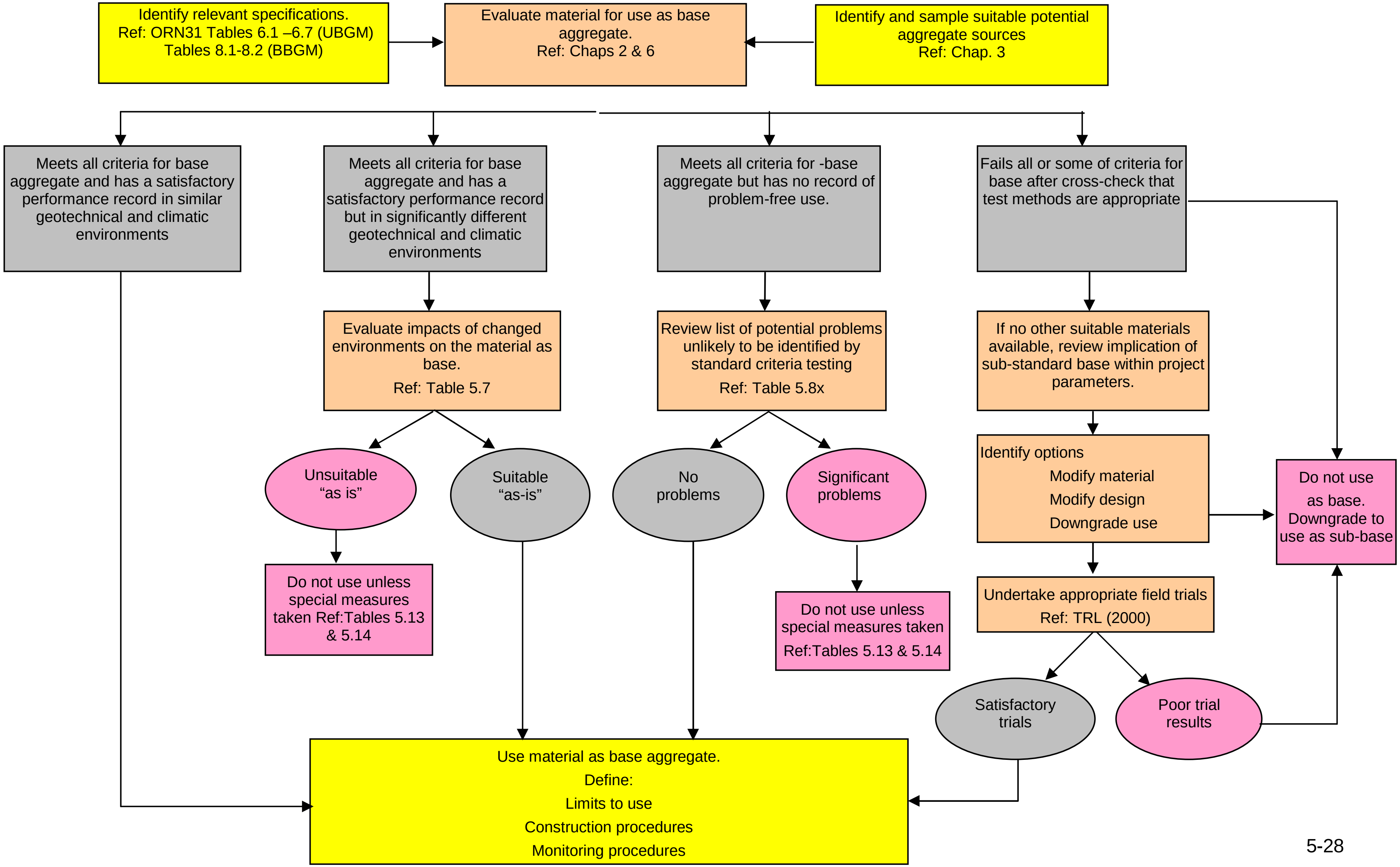


FIGURE 5.6 UBGM AND BBGM BASE AGGREGATE SELECTION FRAMEWORK

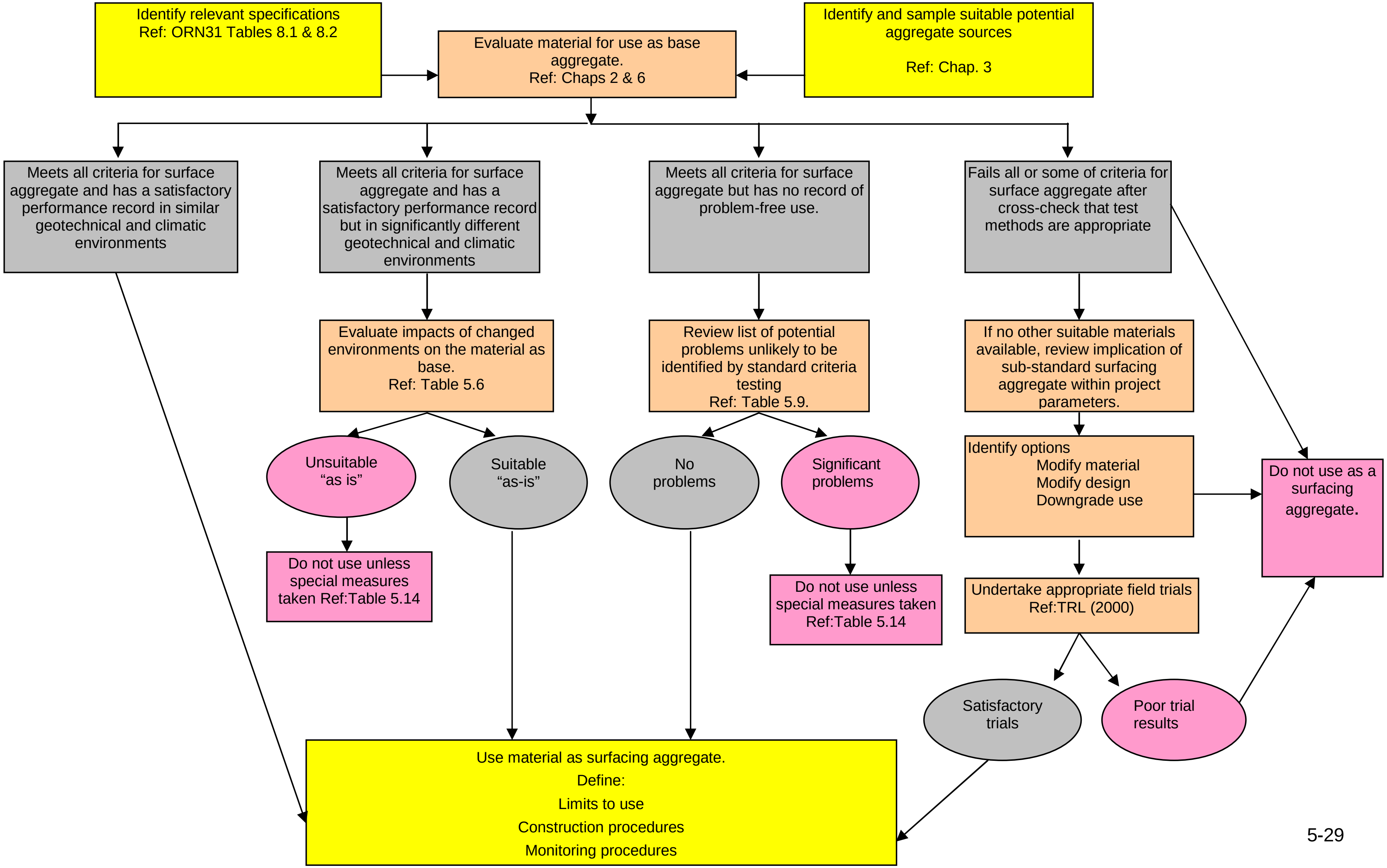


FIGURE 5.7 BITUMEN BOUND SURFACE AGGREGATE SELECTION FRAMEWORK

6 ENGINEERING MATERIALS TESTING

6.1 Introduction

Testing of materials is central to cost-effective road design, construction and performance and as such will be integral to any road project. To be effective, materials testing programmes should take into account not only the selection of appropriate tests but also such factors such the capacity of the laboratory and staff to undertake the tests and quality manage the data produced. Any laboratory can produce numbers; it is the mark of good laboratory to translate these data into believable and useful project information.

This chapter outlines and key aspects in the design and undertaking of material test programmes, both laboratory and in situ, and discusses in detail potential problems associated with specific test procedure in the tropical and sub-tropical environment. Particular emphasis is placed on the selection of appropriate tests and the need for effective quality management throughout the whole testing, reporting and analysis process.

6.2 Materials Testing Programmes

6.2.1 General Principles

Materials testing programmes vary greatly in size and scope depending on the type of the road project and associated works. Materials testing should not be commissioned on an arbitrary or ad hoc basis but should be part of a rationally designed programme. Clear objectives should be identified and test procedures and test programmes need to be designed with these in mind.

Within an overall aim of assuring that selected materials and designs are capable of carrying out their function, testing is undertaken for a number of reasons.

- Identification of potential material resources
- Proving quality and quantity of actual material reserves
- Monitoring quality of as-won or processed materials
- Construction quality assurance
- In service monitoring

These objectives are linked to the required progression of geotechnical certainty as a road scheme moves from investigation, through design and construction to performance. As discussed in Chapter 3 this involves the key aspects of material quality, material variability and quantities available, Figure 3.1.

The relationships between in situ conditions and those experienced by the sampled and tested material need to be clearly borne in mind when designing the and developing the test regime.

6.2.2 Test Identification and Selection Guidelines

Within the general objectives stated above, materials testing programmes may be considered a function of a number of key factors: material nature; utilisation condition; environmental impact; project impact; design guidelines and economic constraints, Table 6.1. Each of the above factors needs to taken into account when selecting an appropriate suite of test procedures appropriate to the phase of the road project.

ORN 31 provides guidance on selection of road construction materials based on a number of standard material test procedures. Table 6.2 summarises the recommended tests in relation to material utilisation and also suggests further tests that should be considered in specific situations.

It is common practice to divide materials test procedures into a number of general categories that reflect the nature of the test. These general divisions apply to the range of materials from soil to rock and are:

Physical: Tests associated with defining inherent physical properties or conditions.

Simulation: Tests associated with portraying some form of geotechnical or engineering character either directly or by implication.

Chemical: Tests aimed at identifying the occurrence of key chemical compounds

Petrographic: Those tests or assessments associated with analysing or describing fabric or mineralogy.

Tables 6.3 to 6.6 list construction materials tests under the above headings.

An understanding of the properties being measured by the individual tests is important in the selection of appropriate procedures, Table 6.7. Aggregate simulation testing, in particular, is for the most part is based very largely on empirical testing procedures rather than modelling expected service behaviours. This issue is particularly important in tropical and sub-tropical environments where assumptions and empirical correlations derived from testing temperate climate materials may not apply.

In the majority of cases no single test procedure will satisfy specification requirements and a battery of test procedures will be needed. An appropriate test programme specification will include a logical selection and sequence of procedures that is function of rock quality, the environment and the road design.

Final as built quality is very dependant on the processes of selection, winning, hauling, spreading and compaction, and an attempt to predict their impacts can be made through pre-treatment programmes prior to testing. For example, by subjecting samples due for particle size analysis, to a compaction cycle prior to sieving. Aggregate impact testing (eg AIV), abrasion, soaking, drying or slake durability pretreatments could also be used in appropriate circumstance on samples prior to a main test.

6.2.3 Application of Testing Standards

The majority of materials tests in developing countries are governed by strict procedures that, in the main, have been originally derived from British (BS), American (ASTM) or French (AFNOR) Standards. In most cases they have been incorporated into national standards, sometimes, however, with local amendments. There are significant problems associated with developing general guidelines for construction materials where multi-standard systems operate, particularly so in the overall tropical and developing country setting. These problems may be seen as falling within two overlapping areas of concern. Firstly there are problems concerned with differing test procedures and, secondly, and more fundamentally, there are difficulties associated with extrapolating test procedures and methods of analysis from a temperate engineering environment to one where tropical climates and road environments dominate. Table 6.8 summarises some common problems and their implications.

In the longer term, the development and adoption of European Standards (EN Series) will pose additional problems with respect to developing country specifications linked to British or other European National Standards as these become redundant, Table 6.9.

British and other standards such as ASTM or AFNOR lay down standards of good practice that are in the main based on "normal" experience with temperate zone sedimentary soils. That is material laid down by the action of water or ice. When dealing with tropical residually weathered materials special procedures are often necessary to obtain reliable, relevant and consistent results. This applies particularly to the handling and treatment of samples before testing (Head 1992, Geol.Soc, 1990)

It is now widely recognised that tropically weathered material behaviour can be noticeably different from temperate sedimentary soils, from which the standard principles of soil mechanics were derived, Terzaghi

(1958), de Mello (1971), Gidgasu (1988). The approach to the laboratory investigation of tropical materials in terms of the range of tests employed, their detailed procedures and their interpretation should derive principally from the following:-

- Chemically bonded materials (eg affects assessment of strength)
- Mineralogical complexity. (eg influences volume change)
- Fragile relict fabric and texture. (eg leads to particle break-down)
- Very heterogeneous soil-rock masses. (eg influences statistical reliability of data)
- Variable climate. (eg moisture susceptibility)

A testing programme needs to recognise the state of weathering so that it can incorporate both soil and rock testing standard procedures in an integrated and overlapping schedule of tests. Materials falling within the hard soil to weak rock category are likely to cause particular difficulty with respect to sample disturbance and/or choice of test procedure. The climatic environment may influence the selection of procedures, or their modification, in terms of moisture condition during test; ie soaked, unsoaked, or some intermediate moisture condition.

6.2.4 Programme Planning

After adequate and representative sampling, investigations should begin with a proper characterisation of the material type. The early identification of detrimental properties and deleterious inclusions is important to the material investigation and an appreciation of the geological background is of great advantage. The importance is stressed of relating the design of the test programme to the material being tested, and conducting tests aimed at identifying inherent engineering character. It is frequently an advantage to ascertain the mineral composition before mechanical tests are performed (Wylde, 1980). Table 6.10 summarises material types that demand particular attention to inherent geological characteristics that influence engineering character.

A phased programme should to be considered in which an initial testing stage is used to evaluate procedures and their validity as regards both material and environment, Figure 6.1 The bulk of the testing programme and the contract specification should be designed only after the initial characterisation of the material has been undertaken, Figure 6.2. In this way potential detrimental features will have been identified and their implications absorbed into the design and necessary adjustments made to the standard test procedures. Where possible test programme design should encompass an analysis of the engineering impacts of the expected road environment in relation to the behaviour of the materials used in its construction, Figure 6.3.

In addition to issues relating to the material type and its proposed utilisation, a well designed testing programme, needs to take a number of other key issues into consideration, namely:

- Focus of application of results (design, contract preparation, resource approval, construction control)
- Economic and time-related constraints
- The ability of laboratories to undertake specific tests
- The requirement for specialist laboratories or equipment
- If a number of laboratories are involved there may be a need for parallel control testing
- The requirement for a preliminary testing programme
- Standards to be adopted
- The need for sub-sampling
- Testing sequences; some procedures may be dependant on material properties

6.3 Test Programme Management

6.3.1 General

The quality of testing programme depends upon the procedures in place to assure that tests are conducted in correctly, in an appropriate and controlled environment and utilise suitable equipment that is mechanically sound and calibrated correctly. The condition of test equipment and the competence levels of the laboratory staff are crucial in this regard. Procedures that assure the effective quality management of the resulting test data are also a necessary requirement. Figure 6.4 presents a laboratory management flow chart that indicates typical checks that need to be in place to ensure quality data is produced.

6.3.2 Testing Equipment

Laboratory and in situ testing equipment should be as specified in standard or modified standard procedures and must be suitably calibrated. Appropriate manuals and procedure standards must be available, and these should be up-to-date originals and not photocopies.

Calibration is the process applied to check the measurements provided by an apparatus or device in order to verify its accuracy. This may done by checking against either a similar instrument with a traceable calibration, or directly utilising standard test materials or a calibration apparatus.

All measuring equipment must be properly calibrated after purchase and then re-calibrated at regular intervals. Calibration should also be undertaken after repair, dismantling or when inaccuracy is suspected. Calibration is of particular importance when locally manufactured equipment or spare parts are employed. A calibration history should be retained for all testing equipment.

Laboratory balances and drying ovens are central to the majority of laboratory tests and their correct usage and maintenance is essential.

Balances appropriate to the amounts of material being weighed should be used. They should also be calibrated at regular intervals using suitably certified weights, preferably at periods of not more than 6 months.

Laboratory drying ovens provide a convenient means of drying materials and in the case of some tropical soils will need to capable drying at different temperatures. They are normally controlled by means of thermostat, which should be calibrated by using a calibrated and certified thermometer to measure temperature at the complete range of settings. A good oven will show temperature fluctuation over period of time in one place within the oven of +/- 0.5^{O} C and a maximum variation throughout the drying space of < 4^{O} C. Ovens require re-calibration if moved and should be used for no other purpose than for drying test material.

6.3.3 The Laboratory Environment

The climatic environment within a testing laboratory needs to be monitored and regular measurements should be made. It may be required for some test procedures, particularly within the tropics, to have specific areas set aside for tight climatic control by air-conditioning. Lack of laboratory climatic control can lead, for example, to significant changes in moisture condition to samples both prior to, and during, test procedures. The following readings are considered desirable:

> <u>Temperature</u>: By means of a max-min thermometer; 3 readings/day - start of day (including overnight max-min); mid-day and end of day.
>
> <u>Atmospheric pressure</u>: By means of a Fortin barometer capable of reading to 0.05mm Hg. Readings twice/day (am and pm)
>
> <u>Relative humidity</u>: By means of a calibrated wet and dry bulb thermometer.

6.3.4 Personnel

Materials testing engineers and technicians are required to be adequately trained and competent in the test procedures for which they are responsible. In larger laboratories there is likely to be a wide range tests undertaken and an ongoing record of staff competence and training is recommended. Table 6.11 lists levels of basic knowledge required of testing staff that should be assessed.

6.3.5 Data Management

The correct management of data produced by a laboratory is an essential element of its quality assurance. A key element of this process is the issue of data reliability. Test results are a function of controllable influences (test procedures) and non-controllable influences (the nature of the materials) and it is absolutely essential that the latter be clearly identifiable and not confused with the former. This is of particular importance when dealing with materials that may have unpredictable behaviour patterns. Table 6.12 outlines a staged approach to the accreditation of laboratory data.

In general terms laboratory management should form part of an overall approach in which information recovered from the desk and field studies is closely integrated with the selection and interpretation of laboratory tests. The use of descriptive information in laboratory testing procedures is also strongly recommended. The observed nature of a material can be of particular use in making decisions on the applicability of particular tests.

The use of standard test procedures and reporting forms is normal laboratory practice and their use should be strictly adhered to. In cases where non-standard tests are being performed, for example mineralogical examination, then new forms should be drawn up and used. Quality assurance procedures should be identified and adhered to. From experience, issues that that can cause difficulty include:

> Sample misplacement: Responsibility for samples, from when they were taken in the field until they are finally disposed of, needs to clearly identified, Figure 6.5 indicates this in terms of sample tracking.
>
> Repeatability and reproducibility: Tests on duplicate samples should be used as control checks, particularly if several laboratories are being used.
>
> 'Black-box' computer programmes: Some of the more sophisticated soils procedures, e.g. triaxial or oedometer tests may have computer control for testing by software that analyses and reports results. The validity of these programmes requires checking, particularly with respect to any assumptions that are made regarding sample characteristics.
>
> Non-Standard procedures: If any non-standard, or modified, procedures are being used this needs to be clearly stated on any test procedure and data record sheets

6.4 Laboratory Physical Condition Tests

6.4.1 General

Physical tests on soil, rock and aggregate materials are summarised in Table 6.3. Key points regarding these tests are discussed in the following sections.

6.4.2 Moisture

Key Definitions:

- *Moisture content; the mass of water which can be removed from a sample by drying, expressed as a percentage of the dry mass.*

- *Water absorption: the increase in mass of a sample of aggregate due to the penetration of water into the water accessible voids into an oven-dried sample.*

The conventional definition of moisture content is based on the loss of weight when the soil is dried to a constant mass at a temperature of between 105 and 110^{O}C. In some topical soils, in addition to the "free" water that is available to influence engineering behaviour there may be additional water contained within the clay mineral structure that is released at these temperatures, Table 6.13. The release of "structural" water varies with mineral types and in some cases results in highly significant differences in moisture content between conventional testing temperatures and engineering working temperatures.

Many authors have highlighted the sensitivity of tropical soils index test procedures, particularly the effects of drying and drying temperature eg; Terzaghi (1958), Newill (1961) and Wesley (1973). Even partial drying at moderate temperature may change the structure and physical behaviour of some materials. Some of these changes are mineralogical and are irreversible. These changes can affect the plasticity, shrinkage, particle size and particle density. It follows that simulation tests such as compaction, compressibility and shear strength can also be affected. Sample may be subject to a number of moisture conditions during pretreatment for testing and in moisture sensitive materials a knowledge of these conditions is important for analysis. The basic moisture conditions are usually as follows:

- As received: ie at natural moisture content (NMC)
- Soaked; immersed in water for at least 24 hours
- Air dried; dried to a constant mass under normal site temperature
- Oven dried: dried in an oven to a constant mass at 105-110^{O}C (OD 110), or at some other lower temperature eg 50^{O}c (OD50)

Comparative testing of drying temperatures on duplicate samples is therefore important in the initial stages of a tropical soil test programme, Figure 6.6. If drying tests are conducted at temperatures lower then 110^{O}C then care should be taken to ensure that a constant mass has been achieved. In some cases this may take a matter of days rather then the conventional 24 hours.

6.4.3 Atterberg Limits

KEY DEFINITIONS

- *Liquid limit W_L: The moisture content at which a soil passes from the plastic to the liquid state, as determined by the liquid limit test.*

- *Plastic limit Wp: The moisture content at which a soil passes from the plastic to the solid state, and becomes too dry to be in a plastic condition, as determined by the plastic limit test.*

- *Shrinkage limit Ws: The moisture content at which a soil on being dried ceases to shrink.*

Liquid and plastic limits are key index parameters, greatly influenced by mineralogy, carried out on fully remoulded soil. For some materials a lengthy period of working is required to achieve this. The fabric of some relict-structured or bonded soils breaks down during the mixing period and the amount of manipulation to which a soil is subjected will influence the test result. The sensitivity to mixing requires verification usually by initially using a range of mixing times prior to testing, eg 5,10, 30 and 45 minutes. In some circumstances it is useful to link the amount of disturbance undertaken for this test to the actual "in service" disturbance.

The Liquid Limit test, as currently recommended by BS, utilises the cone penetrometer (BS1377:part 2: 1990: 4.3) and some differences (up to 2 points) are reported between this procedure and the method using the Casagrande apparatus (ASTM D4318). This can be important in characterising low plasticity soils. The procedure of removing soil particles >425mm and recalculating plasticity on the basis of the removed mass can present difficulties and the use of the large scale cone penetration liquid limit test (Vaughan et al, 1988) may be more appropriate, provided adequate correlations have been established, Table 6.14.

Using the location of a soil on a conventional A-line chart to indicate geotechnical parameters may be misleading for tropical soils, particularly for materials where the fabrics of the in service, the remoulded and the undisturbed conditions are very different. Figure 6.7 illustrates the influence of test procedure on A-Line position for some typical tropical soils

The shrinkage limit test was initially intended for undisturbed samples although remoulded material can be used. Linear shrinkage is a simpler test on remoulded materials, giving a linear rather than volumetric shrinkage. The established relationship between linear shrinkage and plasticity for sedimentary soils (Ip = 2.13 Ls) may not hold true for some tropical soils. It is also important to differentiate between materials that shrink irreversibly (eg halloysite) and those that expand again on re-wetting (eg smectite clays).

6.4.4 Particle Size and Shape

KEY DEFINITION

- *Particle size distribution (PSD): The percentages, by mass, of various grain sizes present in a material as determined by sieving and/or sedimentation.*

Aggregate and soil particle size distributions (PSDs) are investigated by two separate and quite different procedures are used to cover the range of materials from coarse and fine particle size. Sieving is used for gravel and sand particles that can be separated into different size ranges with sieves of standard aperture openings. For the smaller silt and clay size particles a sedimentation procedure is used (either the pipette method or the hydrometer methods). Results are normally presented as a particle size distribution curve.

Some differences occur between BS and ASTM sieve sizes, derived principally from the use of metric and imperial measurement systems. Although both wet and dry sieving are catered for in the standard procedures, dry sieving is only recommended for materials known to be free of fine particle sizes.

For clean ands and gravels (containing no silt or clay) dry sieving can be used. Coarse soils containing silt or clay require the use of the wet method, in which the material must first be washed to remove the fine particles (passing 63um). Predominantly fine soils must first undergo a pretreatment dispersion process and then washed.

The complex relationship in tropically weathered soils between particle size, material fabric, and particle nature can cause problems in the interpretation of particle size distributions (PSDs). The correlation between laboratory PSD and in situ or "in service" PSD size needs to be understood. Materials can have the visual appearance of a sand, yet a laboratory wet PSD of silt or clay can result due to the breakdown of relict parent material clasts.

Drying of fine grained soil should be avoided. Detailed procedures for particle size tests should be assessed in the light both of the nature of the material involved (fabric and mineralogy) and the objective of the proposed test. Figure 6.8 illustrates the effects of differing preparation procedures on PSD. If sedimentation testing is being carried out a proper dispersion of the fine particles is needed. Alkaline sodium hexametaphosphate solution has been found to be suitable for a wide range of soils but in some cases a stronger concentration than the normal 2:1 may be required. An alternative dispersant such as trisodium phosphate may be more effective. Special pretreatments may be required in particular cases, for example hydrogen peroxide in organic materials and hydrochloric acid when carbonate cementation is indicated. Comparative trials may be required at an early stage. In all cases the nature and concentration of dispersants should be reported with the results.

Particle shape is generally defined by British Standards in terms of a flakiness index (If) and an elongation index (Ie) with reference to standard shape gauges. Where:

- If = Mass of particles whose least dimension is <0.6 mean dimension/ Total mass
- Ie = Mass of particles whose long dimension is >1.8 mean dimension/ Total mass

Additional shape factors that may be used include angularity and sphericty. The former can be arrived at by indirect methods in terms of the angularity number (AN) which is reported as a comparison with a standard well-rounded river gravel of air 33% voids (BS 812). The Average Least Dimension (ALD) is a shape parameter used in assessing surface dressing aggregate and is defined as the least perpendicular distance between two parallel plates through which a particle will pass.

6.4.5 Density

KEY DEFINITIONS

- *Particle density: The ratio of the oven-dried mass of a sample of material to the volume it occupies in water including both internal sealed voids and water accessible voids.*

- *Absolute particle density: The particle density of the mineral constituents present in the soil, measured by pulverizing the soil such that all impermeable voids in the coarser grains are exposed.*

- *Apparent particle density: The ratio of the mass of a sample in its natural state to the volume it occupies in water including any internal sealed voids.*

- *Bulk Density: The mass of material (including solid particles and any contained water) per unit volume, including the voids between the particles.*

- *Dry Density: The mass of material after drying to constant mass at 105^{O}C contained unit volume of undried material*

Aggregate particle density, sometimes referred to as “bulk particle” density or “relative” density has a number of variations (BS 812):

- *Oven Dried (OD): The ratio of the oven-dried mass of a sample of aggregate to the volume it occupies in water including both internal sealed voids and water accessible voids, with the aggregate dried to a constant mass at 105^{O}C.*

- *Saturated and surface dried aggregate (SSD) is defined as aggregate in which all the accessible pores of the aggregate fully saturated with water but where the outer surface is dry and has no adhering water.*

- *Wet Surface Dried (WSD): Similar to SSD but with the permeable voids not entirely filled with water.*

The above range of definitions can give rise to significant confusion. The term 'particle density' (Ps) has replaced the previously used 'specific gravity' (Gs) in current British practice. Particle density has the same numerical value as the former specific gravity although it has the units Mg/m3 rather than being dimensionless. Some materials that have large inherent void content, pumice or cinder gravel for example may show an "apparent" low particle density although the absolute particle density is high. Due regard should be taken of the moisture drying temperature problems discussed above.

It is important to recognise the differences between the various defined particle densities. In particular the particle density of aggregate particles and the particle density of a fine soil material composed of exactly the same minerals may vary significantly due to any internal voids contained in the aggregate particle. The latter will be closer to an "absolute" particle density than the former.

The absolute particle density gives the highest particle density that can be obtained for a given material. The apparent particle density may be equal to, but usually slightly smaller than, the absolute particle density. The aggregate particle density parameters are generally employed in bitumen bound or concrete aggregate mix design calculations and in decreasing order of magnitude are:

Bulk particle density (SSD)
Bulk particle density (WSD)
Bulk particle density (OD)

Tropical soil-rock profiles may have highly variable particle densities and hence the parameter should be measured whenever it is required in the calculation of other parameters, such as air voids, rather than using assumed values, Table 6.15. In the PSD tests the proportions are determined by weighing the volume occupied by the dried material retained between each sieve sizes, this involves the assumption that the particle density is constant over the range of sizes, which is not always the case.

In addition to being a requirement for geotechnical analysis, the in situ bulk density (P) and the related dry density (Pd) can prove to be useful index tests, and can be used as rapid indicators of relative strength and fill suitability within previously defined limits. Bulk density may be arrived at by a variety of procedures summarised in Table 6.16.

6.4.6 Derived Indices

A number of common soil indices are derived from relationships between moisture content, Atterberg limits and particle size, such as void ratio, porosity, activity, liquidity index. These can be useful for characterising general engineering and geotechnical behaviour, however, empirical relationships and classifications established for temperate materials may need to be re-established for tropical materials.

A further range of derived indices based primarily on grading and plasticity are used to characterise unbound granular materials and soils. These are listed and defined in Table 6.17

6.5 Laboratory Simulation Tests

6.5.1 General

Tests that essentially measure properties of materials in conditions designed to portray some form of in service condition, either directly or by implication, are termed simulation tests. Procedures that fall within this category for soil, rock and aggregate materials are summarised in Table 6.4. Key points regarding these tests are discussed in the following sections.

6.5.2 Volume Change

Detrimental volume change in compacted road materials or within in situ soils arise through a number of processes:

- Swell or shrinkage due to the inter-action of clay minerals with moisture
- Collapse of an over stressed soil or aggregate fabric
- Consolidation of a road layer due to the expulsion of water under load.

The potential for volume change is enhanced both by the occurrence of "swelling" clays and a sensitive material fabric that can lead to collapse on moisture increase or loading. Although indications for such behaviour may be given by other index tests or by fabric examination, it is recommended that direct testing methods be employed where such behaviour is suspected.

Swelling tests fall into three general procedures; radially confined swelling pressure tests, radially confined swell amount tests and free swell tests. The first two procedures utilise the standard oedometer equipment and hence necessitate a sample capable of being extruded or cut into the required shape.

It is important to distinguish between the swelling pressure test in which the pressure to prevent swelling is measured and the swelling test in which swelling is allowed to take place and is measured. For soil materials both tests involve the use of the odoemeter.

The swelling pressure test measures the load required to give zero swell on flooding, while the swell-amount test measures the swell either under minimal load or projected working load. ASTM (D4829) suggests the use of a swell index (EI) based on the change in sample height based on a 50% saturation.

Reviews of swell testing have indicated that assessment of swell for road design should be based on laboratory procedures that best simulate the expected sequence of loading and wetting that is expected in situ, Schreiner & Gourley (1993). It is recommended that the majority of testing programmes aimed at reproducing project conditions should select the swell-amount test as being the most appropriate, Holtz & Gibbs (1956). This test can form part of the double oedometer procedure, Knight and Jennings (1957a) when it is used in conjunction with standard consolidation procedures involving identical non-flooded samples for comparison purposes.

A swell test on indurated rock-like materials is described by ISRM (1981) and involves soaking a rectangular prism of material in water and measuring free swell by micrometer gauges mounted against three orthogonal faces. This test can only be conducted on materials that are not susceptible to significant slaking.

The collapse test also utilises standard oedometer equipment and entails the flooding of the sample under a constant load. The flooding load may either be related to anticipated project conditions or may be a standard index as suggested by Jennings and Knight (1957b). They proposed a standard load of 200 kPa as part of their collapse potential index (CPI), Figure 6.9.

Consolidation is the process of squeezing out of water from saturated soils, as distinct from compaction, which essentially involves the reduction of air voids in unsaturated soils. Testing is usually employed to estimate the extent to which soils will settle under embankment loads. Commonly undertaken using oedometer apparatus; it can also be part of a triaxial test procedure. Results will aid in decisions as to the rate of construction and the need for internal or external drainage to reduce pore pressures as material is placed and compacted in embankments .

6.5.3 Compaction

KEY DEFINITIONS

- *Compaction: The process of packing material particles more closely together by mechanical means to increase the dry density of the soil.*
- *Optimum moisture content (OMC): The moisture content of a material at which a specified amount of compactive energy will produce the maximum dry density.*
- *Maximum dry density (MDD): The maximum dry density of a material obtained using a specified amount of compactive energy.*

Compaction is concerned with relationships between moisture content, applied effort and density. Compaction is undertaken on the road to enhance the mass density and hence the strength, rigidity and durability of placed materials. In the laboratory compaction testing is undertaken to predict moisture-density responses of a material to applied effort and to provide a reference with which to control on-site compaction during construction. Laboratory compaction procedures provide only an approximate simulation of field stress regimes under varying compaction plant. It is necessary to bear this in mind when relating laboratory results to field density specifications.

Laboratory compaction tests are undertaken to determine the combined influences of compactive effort and moisture content on the dry density of soils. Compactive effort can be varied depending on the selected procedure and Table 6.18 summarises the principal laboratory compaction procedures in relation to their applied effort. Typically this test produces an inverted U curve indicating an optimum moisture content for maximum compaction utilising a particular compactive effort. For meaningful interpretation it is necessary to plot air voids lines (typically 0%, 5% and 10%). It is not possible to interprate the relevance of compaction curves without reference to these air voids lines, Figure 6.10.

On the "wet" side of the optimum moisture content (OMC) the presence of an increasing volume of water physically prevents further compaction of soil. On the "dry" side of the OMC the diminishing presence of water reduces its lubricating affect. With some soils the state of compaction increases again as the moisture content moves towards zero, insufficient water being present to mobilise suction forces. This can find useful application in some arid climates where water not available for field compaction and a dry compaction technique can be used (Ellis, 1970).

There are number of factors that need to be taken into account when selecting, or modifying a compaction programme for tropical materials, namely

- Soil drying can irreversibly aggregate some soil fabrics
- Oven drying to 105Oc may give misleading moisture content results
- Particle densities may be highly variable (influence on the air voids line)
- Fragile soil particles can degrade on compaction
- Tropically weathered soil profiles are likely to be highly variable

All the above can have a significant effect on the connection between laboratory moisture-density relationships and those achieved or asked for in the field. It has been shown for example that in lateritic and residual red soil materials oven drying to 105^{O}C will give higher MDDs and lower OMCs than materials tested at natural moisture content, Newill (1961); Wesley (1973). In some cases it is appropriate, therefore, to counter the effects of drying by using fresh soil for each compaction point and drying-back soil from its natural condition rather than wetting-up dried soil.

Variation in particle density, either between size fractions within a sample or between adjacent samples in a profile, can cause problems in selecting representative air voids lines.

The Moisture Condition Value (MCV) test is basically a compaction test in which compactive effort is increased incrementally until a state of full compaction is reached. Compactive effort, in terms of the number of blows necessary to compact a sample of soil determines the MCV of that soil, Figure 6.11. The MCV test is not included within ORN 31 and has only been fully evaluated for temperate soils. Its use with tropical and sub-tropical materials can only be recommended following rigorous parallel testing with established compaction, CBR and strength test procedures to establish material specific correlations.

6.5.4 Density –CBR Relationships

The California Bearing Ratio (CBR) is an empirical test employed in road engineering as an index of compacted material strength and ridgity, corresponding to a defined level of compaction. It is not based on any theoretical concepts and is reported as a dimensionless percentage of a standard intended to represent the value that would be obtained from compacted crushed stone.

Figure 6.12 illustrates the range of CBR test procedures that can be employed. Both BS and ASSTM tests are similar although there are important differences with respects to the sample moulds and compaction procedures used. The ASTM procedure emphasises soaking as an essential feature, while BS allows but does not insist on soaking. The four-day soaking period is somewhat arbitrary, particularly with clayey soils of low permeability where there may be insufficient time for full saturation of the sample. It is more useful to undertake CBR tests on working or estimated in situ densities and equilibrium moisture conditions. ORN 31 offers sub-grade design CBR evaluation at moisture contents other than saturation. Experience suggests that CBR test is of poor reproducibility, particularly with granular soils (Millard, 1990)

In addition a form of swell test is commonly undertaken during the soaked CBR procedure. This should be completed to zero swell increase.

The test can be useful to investigate variation of bearing strength in relation to changing moisture content or changing density. For example, when investigating the characteristics of soils not previously used in road construction it is recommended that a full examination of the CBR-density-moisture content-compactive effort relationships be undertaken. The construction of iso-CBR plots is a useful means of investigating these relationships, Figure 6.13. ASTM requires the plotting of CBR values for three separate densities and other standards recommend the use of five point procedures. This practice allows for realistic interpolation of as compacted field density-CBR relationships.

6.5.5 Soil and Rock Material Strength

Unconfined compressive strength (UCS) testing may be used for strength testing the more robust tropically weathered materials; ie from hard soils to strong rocks. Good intact core samples recovered using high quality drilling or core cutting techniques are particularly useful. Care must be taken in selected a representative orientation of sample, in preserving the in situ moisture condition and forming parallel axial faces.

Triaxial compression test procedures that now play such a large role in geotechnical testing programmes have largely been derived for use on traditional sedimentary soils in temperate climates. Their application to partially saturated and fabric-influenced materials in climatic environments that impose rapid changes in moisture condition can cause difficulties both in establishing relevant test procedures and in the modelling of site conditions. The standard policy of imposing saturation on under-saturated materials appears difficult to justify on the grounds of modelling site conditions

The various types of triaxial testing that are commonly employed in the field of road construction are as follows:

- Undrained unconsolidated (UU): To asses short term fill strength and side-slope stability. Also used for traffickability and control of fill quality.
- Consolidated undrained (CU) with pore pressure measurement: To assess long term strength and side-slope stability of embankmnets and cut-slopes.
- Consolidated drained (CD): Assessment of "during construction" strength and side-slope stability and long terms assessment of cut-slopes.

Multistage triaxial testing is not recommended for tropical materials especially in those with an unstable fabric liable to collapse, for brittle soils and for those that show strain-softening characteristics. Special procedures are likely to be required for high void ratio or bonded materials, eg low confining pressure, slow loading rate (Geol. Soc. 1990).

Many tropical soil-rock profiles are known to be in an under-saturated condition. As a result their in situ geotechnical performance is likely to be influenced by variation of soil suction (negative pore water pressure) in response to rainfall infiltration.

Measurment of suction can be undertaken indirectly in the laboratory by means of the filter paper method, Chandler and Gutierrez (1986); Chandler et al (1992). This method involves placing Whatman's No. 42 filter paper in sealed contact with the soil for a period of 7 days and measuring the amount of moisture taken up by the paper (Wfp). Matrix suctions may be arrived by the following empirical relationships

$$\text{Suction (kPa)} = 10^{(4.84-0.0622Wfp)} \quad ; \text{for } Wfp < 47\%$$

$$\text{Suction (kPa)} = 10^{(6.05-2.48\log Wfp)} \quad ; \text{for } Wfp > 47\%$$

The procedure involves using undisturbed hand trimmed tubes samples extruded in the laboratory with filter paper sandwiched and sealed between two cut sections. The direct effects of suction may be examined by use of the double strength testing in natural and in soaked conditions. An alternative approach is to measure suction directly in situ (cf Section 6.9.3)

In some cases the testing of remoulded slurry materials in quick undrained conditions can yield valuable information with respect to field properties; for example, in determining the minimum moisture conditions required for fill placement and traffickability (Burland 1990, Vaughan 1993). Laboratory shear vane testing can be usefully used on samples compacted at a range of densities to obtain information on shear strength-density relationships

The point load test utilises a simple portable a hydraulic device for breaking rock cores or irregular lumps, and is commonly related to the UCS test. Correlations vary with rock type and should be checked before adoption. It is of particular use in situations where good, representative, core samples are available, Broch and Franklin (1972).

The Schmidt rebound hammer, originally designed for the non-destructive testing of concrete cubes, can be employed as a rough guide to strength either on in situ exposures or on rock boulders. It can be significantly influenced by the occurrence of fractures in the rock and by operator procedure. Great care should be exercised in interpretation of Schmidt hammer data, Kolaiti & Papadopoulus (1993)

6.5.6 Aggregate Strength and Durability

There is a range of test procedures available for assessing aggregate strength and durability. The strength tests involve evaluating the resistance of selected aggregate to either impact or load, whilst durability procedures involve assessing the performance of aggregate when subjected to some form of artificially imposed degradation or weathering. Some test procedures, such as Los Angeles Abrasion, encompass elements of both strength and durability testing.

Durability can be defined as the ability of a construction material to maintain its mechanical and physio-chemical integrity within the road environment service life. Durability assessment should therefore have elements of behaviour modelling and prediction and should not rely on one-off strength evaluations.

Examples of aggregate strength test are the Aggregate Impact Value (AIV), the Aggregate Crushing Value (ACV) and its related 10% Fine Aggregate Crushing Test (10%FACT). The main disadvantage of these tests, in their unmodified form, is that they use only a single size (9.5mm - 14mm) fraction.

The AIV test measures the ability of aggregate particles to resist 15 blows of standard weight dropped through a standard height by measuring the amount of fines (passing 2.36mm) produced. Apart from procedural non-compliance the test result can be influenced by factors such as particle shape, moisture condition, base plate seating and the cushioning effects of the produced fines. Modified AIV test procedures have been proposed by Hosking & Tubey (1969), which can be employed to:

i) Measure the intermediate breakdown between 10mm and 2.36mm.
ii) Limit the number of blows and then extrapolate to the full 15 blows
iii) Test both in an unsoaked and soaked condition.

The ACV evaluates the resistance of aggregate particles to a continuous load of 400 kN over period of 10 minutes by measuring fines produced as in the AIV test. In order to reduce cushioning effects the 10%FACT is more generally used in which the load to achieve 10% breakdown is measured. As with the AIV, both soaked and dry samples can be tested. In some aggregate specifications the ratio of soaked to dry values is stipulated (NITTR, 1990)

The use of ethylene glycol soaked aggregate can be used in the above tests as rapid means of evaluating suspect materials such as basic igneous rocks. Ethylene glycol AIV tests that are five percentage units above an AIV (wet) test result are considered indicative of a problem material (Sampson, 1980)

Sodium and magnesium soundness tests measure resistance to mechanical degradation through cycles of crystallisation and rehydration. They have, however, come under criticism for lack of reproducibility in comparisons between different laboratories. A detailed review by Sheftick et al (1989) indicated that the test was valid, but that it is susceptible to poor laboratory management control, in particular with respect to temperature control and the type of sodium sulphate used.

The Los Angeles Abrasion (LAA) test is sometimes linked in specifications to AIV and ACV tests. However, the test mechanism is more one of mechanical degradation rather then particle strength and it should be considered more reasonably as a durability indicator. The Durability Mill test, or Texas Ball Mill test, has the advantage over other strength-degradation tests in that a larger range of grading is tested, less bulk sample is required and the degraded products of the test are retained for identification and assessment. The NITRR procedure includes testing the plasticity of the degraded fraction and recording the results as part of the durability index

The slake durability index test (ISRM, 1981), in addition to being a useful performance indicator can perform a significant role in indexing materials in the rock to hard soil range. The procedure requires competent lumps of material for testing. The combination of slake index with plasticity has been suggested as a useful means of presenting results for argillaceous materials.

Three principal tests are involved in the determination of soil-like material durability or erodability: the Pinhole Test; the Crumb Test; and Dispersion Test. The first two tests rely largely on the qualitative observation of behaviour allied to empirical correlation with known behaviour. The dispersion test depends on the comparison of treated and untreated hydrometer particle size results again allied to correlation with known behaviour. Directly applicable data may be more reliable as recovered from the close observation of field exposures.

6.5.7 Surfacing Aggregate Tests

In addition to strength and durability tests a number of special tests are used for assessing the suitability of a material as a surfacing aggregate, as summarised in ORN 3 (TRL, 2000).

In the Accelerating Polishing Test; which involves 50 representative chippings being subjected to the polishing action of a revolving pneumatic tyre fed with abrasive powder, is considered complicated and tedious and needing specialist operators. The state of polish is measured with a portable pendulum skid resistance tester and the results reported as Polished Stone Value (PSV - 0.3 = high polish; 0.8= gritty rock).

The aggregate abrasion value (AAV) test is used to provide an estimate of surface wear for surface aggregate. Mounted aggregate particles are abraded in a controlled fashion and the loss of weight as a percentage of the original weight is reported as the AAV. The test is a function of aggregate strength and hardness and can be related to AIV (Hawkes and Hosking, 1972)

Although it has similarities with the LAA test the Micro-Deval test, as developed in France (NF P 18-572) is also used for surface aggregate assessment. The test involves samples of aggregate being rotated along with steel balls in a drum and the loss measured in terms of material passing a 1.25 mm sieve. Aggregate can be either soaked (MDE) or dry (MDS).

There are also observational tests directly relevant to bitumen adhesion, for example ASTM D1664 in which the bitumen coating is observed after 16 to 18 hours of soaking in water. ORN 3 provides further detail of aggregate-bitumen testing.

6.6 Chemical Testing

Whilst the detailed chemical composition of materials may be of limited interest for road engineers the presence of some constituents can be of great significance. These include: organic matter; sulphates; chlorides; and carbonates. Table 6.6 summarises commonly used chemical tests.

The biggest source of error in chemical testing is in the selection of the sample. Usually a very small sample of dried soil is required and it is essential that this sample is truly representative of the original sample.

Results of chemical tests on soils should be regarded as an indication of the order of magnitude of constituents for classification purposes rather than as precise percentages. British Standard test procedures provide sufficient accuracy for most temperate soils but for some tropical soils there is the possibility that the presence of other constituents could have an influence on results.

6.7 Petrographic Evaluation

6.7.1 General

The geological nature of an aggregate is fundamental to its engineering character and behaviour as a road construction material (cf Chapter 2). Therefore, an examination of the mineralogical composition, its fabric and texture can yield valuable information on its likely performance, Whilst the occurrence of deleterious minerals may indicate a potential problem, their size, shape and spatial arrangement can give a direct lead as to the seriousness of the problem. Combined mineralogical textural and fabric examination procedures, such as those as described by Cole and Sandy (1980), are strongly recommended as part of any comprehensive aggregate assessment.

Table 6.6 summarises common petrographic procedures. The following sections highlight some key aspects of their application.

6.7.2 Optical Microscopy

A standard binocular zoom microscope may be effectively used for the examination and classification of soil or rock fabric and mineralogy. Some care may be required in the preparation of suitable samples and also in the correlation of results to site conditions, due to the small size of sample involved. Instruments may be fitted with camera attachments so that photographs of fabric and mineralogy may be taken for later examination or illustration.

Examination of rock or impregnated soil thin sections by means of a petrological microscope is a long-established geological procedure. In the context of construction material usage there should be an emphasis the on the assessment of secondary or unsound mineral content and its spatial distribution (fabric). A number of petrographic indices have been established for the purpose of giving guidance on the likely performance of materials based on mineralogy and fabric, (Lumb, 1962; Irfan & Dearman, 1978: Cole and Sandy 1980)

Thin sectioning has limitations in weathered and friable materials. Specialist impregnation techniques may be needed to strengthen materials prior to slide manufacture (Humphries 1992). Identification of fine clay minerals is very difficult and the use of thin sections alone for the mineralogical examination of mudstones and shales is not recommended.

6.7.3 Scanning electron microscopy (SEM)

The SEM is being increasingly used as a means of examining micro-fabric and mineralogy.. The cost of equipment and the care required in sample preparation are major drawbacks. The very small size of sample examined needs to borne in mind when using this method and it is necessary to have a clear idea of the nature of the material and the information required when operating the SEM. The electron microprobe, which is essentially an X-ray fluorescence spectrometer attached to the SEM equipment, allows a chemical analysis to be made to assist chemical or mineralogical identification.

6.7.4 X-ray diffraction (XRD)

XRD has been widely used in the identification of mineralogy. There are established standard approaches for sample preparation and interpretation (Wilson 1987; Brindley and Brown 1980). Variation in test procedure, for example by using glycolation or high temperature drying, is essential in the identification of some clay minerals. The XRD procedure produces a trace with peaks generally indicative of the presence of minerals; it is not a truly quantitative procedure and requires a correlation with other mineralogical or chemical evidence before definitive mineral percentages can be determined.

6.7.5 Methylene Blue Testing

Methylene blue testing should be a standard procedure when performing petrographic examination. It is not always necessary to know the nature of the deleterious clay minerals, only that they are present. The MBA test does this.

In the Methylene Blue Value (MBV) test about 2g of finely ground aggregate, or soil (<0.063mm), is made into a suspension in distilled water. Methylene Blue (MB) solution is titrated into this suspension until successive 'spot' tests on filter paper indicate a definite blue halo. The volume of MB solution used to cause this colour change is noted down and used to calculate the MBV value in g/100g from the following:

$$MBV = \{(X/Y).PMB\}/(A/100)\ \%]$$

Where:

X=Wt of dried MB crystals in g
Y=Volume of diluted MB solution, in ml
P=Volume of MB solution added, in ml
A=wt. of dry soil, in g

Suggested values for MBV with respect to aggregate durability are as follows (Higgs 1996):

- <0.7 Acceptable
- 0.7 -1.0 Marginal
- >1.0 Unsound

Methylene blue can also be used to stain thin sections and point-counting procedures used to get a percentage volume of stained (clay) minerals. An MB mineral value of 20% is considered to be a cut-off between problem and non-problem aggregates (Verhoef et al, 1999). It should be noted that the MB tests take no account of texture or fabric and that this may result in anomalous results unless these factors are taken into account by incorporating some form of textural factor, as used by Cole and Sandy (1980).

6.8 In situ tests

6.8.1 CBR Testing

The acquisition of in situ CBR values is commonly achieved by indirect correlation, rather than by the direct use of in situ CBR equipment. With the possible exception of heavy clays, correlation has proved difficult between results from the in situ CBR equipment and those obtained in the laboratory. The differences in confining effect between the field situation and the laboratory mould are reported as being a prime reason this problem (Millard, 1993)

The TRL DCP procedure involves an 8 kg hammer dropping through a height of 575 mm and a 60° cone with a maximum diameter of 20mm. The results are plotted on a standard field sheet and then processed utilising established relationships between DCP readings and CBR (TRL 1990, Smith and Pratt 1983). Agreement between these relationships s generally good although differences become apparent at low values of CBR in fine grained materials. In cases such as these, where precise values are required, it is advisable to calibrate the CBR for the material being evaluated. (TRL 1999).

The DCP has drawbacks when testing involves penetration of hard granular layers, which may require coring through before continuing with the DCP profile. Care should be taken with respect to any side friction build-up if the rods are in contact with the ground. Continuous in hard material will accelerate wear on the cone and it is recommended that this should be replaced when its diameter is reduced by 10%

A number of other indirect investigation methods have been employed to ascertain in situ CBR. These include; Clegg impact hammer; standard penetration testing (SPT N values); and undrained shear strength. Regression equations derived for the procedures (eg Linveh, 1988) are generally very material-specific and need to be approached with caution.

6.8.2 Density Testing

The usual method of measuring compaction in the field is to determine the in-situ dry density of the soil/material. It is important that the measurements are carried out in such a way that the quality of compaction is determined as accurately as possible: it is imperative, therefore that the complete depth of a layer being compacted is included in the sampler. Dry density normally decreases towards the bottom of the layer, where the compaction stresses are lowest and the density in lower regions of the compacted layer may be critical to the satisfactory performance of the compacted fills.

Methods of determining in-situ dry density are summarised in Table 6.19. These standard methods of measuring field density of soils are not very precise and require considerable skill, particularly the sand replacement test, which is recommended for most granular materials and soils. Their reproducibility is also very poor. Nuclear density and moisture content methods offer a more rapid and much less tedious approach, although the equipment requires careful handling, safety precautions and regular calibration.

6.8.3 Suction Measurement

A suction probe procedure is now available for the monitoring of suction in sub-grade and embankment fills, O'Connell & Gourley (1994). This involves the use of a simple, robust and reliable technique incorporating filter paper in an inner assembly that is sealed against an outer assembly which can be driven into the ground using a procedure based on the dynamic cone penetrometer. The whole assembly is permanently sealed into the materials to be monitored.

Factor	Description	Implication
Material nature	Materials occur in a wide range of natural conditions, from a variety of hard unweathered rock types, through weathered rock to unconsolidated or pedogenic soils.	In addition to the gross general difference between the of testing soil or of rock, inherent characteristics of material sources, such as mineralogy and fabric can be such as to demand special attention.
Utilisation condition	The state of disturbance, of the as-used material as compared to its in situ and test conditions	Some material types, particularly those with a dominant fabric can exhibit very different engineering behaviours depending on whether they are in an undisturbed, disturbed, or totally remoulded condition.
Environmental impacts	The geological, geomorphological, geotechnical climatic and hydrological environments influencing the road.	The engineering behaviour and likely whole road-life performance of some material types is influenced by environment to the extent that some test procedures may require modification
Project impacts	Depending on location within the road profile materials can be subjected to varying influences such as static load; dynamic load; attrition; erosion; chemical alteration and hydrological changes.	The results from any test procedure must be clearly understood in terms of what they mean. Many tests are merely indications of expected behaviour based on certain correlations and assumptions.
Design guidelines	ORN 31 provides tables of material properties to be ascertained by a variety of standard tests.	Test procedures as indicated in ORN 31 need to be in strictly accordance with those procedures used to define the specification limits
Economic constraints	Test programmes will usually be constrained within some form of budget.	There may be need for experienced engineering judgment to balance the "ideal" test programme with that which is possible within the available budget

Table 6.1 Key Factors Influencing Test Programme Design

Material Usage	Standard Specification Compliance Testing in ORN 31	Additional Testing Recommended in ORN 31 for Specific Situations/Materials	Further Testing Recommended
Common Fill (Soil)	Heavy compaction MCV		Moisture content Atterberg limits Triaxial UU [traffickability] Triaxial CU [settlement; permeability] Consolidation [fill settlement]
Common Fill (Rock)	Non Recommended		Point load Unconfined compression strength Durability Mineralogy
Sub-Grade	Moisture content Plasticity Light compaction Heavy compaction CBR (soaked & unsoaked)		PSD Dispersion [pin hole etc] Collapse Swell
Drainage-Filter	Atterberg Limits CBR (soaked)		PSD Chemical testing Durability-strength
Capping Layer	Heavy compaction CBR (soaked)		Moisture content Atterberg Limits PSD
UBG Sub-base/ Roadbase	Atterberg Limits PSD CBR (soaked) Heavy compaction 10%FACT AIV (modified) Particle shape	***Basic Igneous Aggregate*** Durability (Mg & Na Sulphate) Methylene Blue Value Texas Ball Mill Petrographic analysis	Full petrographic examination
Bitumen Bound Base and Surfacing	Sedimentation Flakiness index ACV AIV LAA AAV PSV Mg/Na Soundness Water Absorption Immersion Tray Test	10%FACT for weaker aggregates For fine aggregates: Sand equivalent Plasticity	Full petrographic examination
Cement & Lime Stabilisation	Plasticity PSD ICL ICC Heavy compaction Strength (UCS) CBR (soaked)	Gravel ICC (NITRR, 1984)	

Table 6.2 Selection of Laboratory Tests based on Material Usage

Physical Condition Tests	Standard Procedures		Advantages of Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	BS	ASTM			
Moisture Content	1377: 2:,3.1 812:109	D2216	Simple and widely accepted test.	Misleadingly high moisture contents in halloysitic and allophane rich soils.	Drying at differing temperatures. Sand bath option available as a quick option for granular materials.
Water Absorption	812:109	C127 & C128	Simple test with correlations established with bitumen bound material design	Variability in multi-clast type deposits	
Liquid Limit (WL)	1377:2,:4.3-6	D4318	Well established soil index and classification test	Influence of >425µm particles; moisture condition and mixing time. Correlations between procedures require caution.	Undertaken at differing moisture states. Drying at differing temperatures. ASTM D421 is an air dry option. Use of large cone in coarser soils, Vaughan et al (1988)
Plastic Limit (Wp)	1377:2,:5.3	D4318	Well established soil index test. Plasticity index (Ip = W_L-Wp) used as a key defining parameter in many specifications	Influence of >425µm particles; moisture condition and mixing time. Poor reproducibility and repeatability	Undertaken at differing moisture states. Drying at differing temperatures
Shrinkage Limit (Ws)	1377:2, 6.3	D427 & D4943	Yields index information on volume change potential	Initially intended for undisturbed samples although remoulded material can be used.	D427: Shrinkage factor. D4943 Wax method.
Linear Shrinkage (Ls)	1377:2,:6.5		Can give an estimate of Ip for soils where W_L and Ws are difficult to obtain .Better repeatability and reproducibility than plasticity test.	Established relationships between Ls . and Ip may not hold true for some tropical soils.	Drying at differing temperatures.

Table 6.3 Laboratory Physical Condition Tests

Physical Condition Tests	Standard Procedures		Advantages of Using Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	BS	ASTM			
Particle Size Distribution	1377:9:2.3-5 812:103,1	D422	Simple and widely accepted test incorporating both sieving and sedimentation. A fundamental soil classification tool.	Interpretation problems with aggregated particles or weak clasts. Requires particle density values. Different maximum sand size between BS (2mm) and ASTM (4mm).	Drying of soil should be avoided. Use computed weights for calculations. Alternative dispersion agents; trisodium phosphate; tetrasodium phosphate may be better from some tropical soils. Cohesive soils with gravel/cobble content require special attention.
Sand Equivalent Value		D2419	A rapid site-lab means of determining relative fines content	Dispersion problem in agglomerated minerals. Relative proportions only	
Aggregate Grading (Sieve) Aggregate Sedimentation	812:103,1 812:103,1	C136 & C117	Simple and widely accepted test for defining aggregate size distribution.	Differing usage of "coarse" and "Fine" between BS 812 and 1377. Wet sieve unless little or no fines.	Dry sieving methods in materials free from agglomerated particles only.
Flakiness Index (If) Elongation Index (Ie)	812:105, 1 812:105, 2	D4791	Standard gauge methods of ascertaining particle shape. Parameters incorporated into coarse aggregate specifications	BS Flakiness test (105.1) easier to undertake than Elongation Index 105.2). Use restricted to coarse aggregate only.	Additional shape test: Average Least Dimension (ALD): NTRR, 1986. D4791 produces estimates of flat, elongated or flat and elongated particles only
Angularity Number	812:105		Rapid indirect method of estimating gravel roundness based on relative voids content.	Can only be valid for strong aggregate particles, adequate enough to resist tamping blows without any degradation.	Roundness also by labour intensive observational methods..
Surface Condition	812		Means of indirectly assessing frictional and bitumen adhesive properties of c aggregate.	Subjective assessment only.	Detailed objective procedures using thin sections in Wright (1955)
Soil Particle Density	1377:2, 8.2	D854	Required for use in analysis of other parameters (e.g. psd, compaction)	Some soils influenced by drying temperature Potential confusion between density definitions.	Undertake at natural moisture content. Drying at differing temperatures. Slight differences in procedure between BS and ASTM.
Aggregate Particle Density (Bulk particle or Relative Density)	812:2	C127/128	Required in bitumen bound granular material design calculations	In aggregate the procedure will give an "apparent" rather than an "absolute" value. Not directly correlatable with soil particle density	Can be measured for a number of states: saturated surface dried (SSD);wets surface dried (WSD) or oven dried (OD)
Bulk Density	1377:2	C29 & C29M	(Table 6.15)	(Table 6.15)	Variety of alternative methods outlined in Table 6.15 . ASTM test for aggregate <150mm

Table 6.3 Laboratory Physical Condition Tests (Continued)

Simulation Tests	Standard Procedure		Advantages of Using Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	BS	ASTM			
Swell Pressure	1377:5,:4.3	D4546	Undertaken on undisturbed or recompacted material to determine pressure to minimise swell.	Only measures swelling pressure. Soil or fine aggregate only. To measure swell amount use BS 1377:5, 4.4	Swell amount test; BS 1377 5:4.3; ASTM D4546. ASTM: D4829 - use of swell index EI. Unconfined swell; ISRM (1981)
Collapse	1377:5,:4.5	D4546	Can give good indication of potential for fabric collapse.	Preferable to model site compaction-moisture conditions, eg flooding at project load. Disturbance problems in sensitive fabric materials.	Alternatively Collapse Potential Index (CPI) load at 200kPa; Jennings and Knight (1975).
Consolidation (oedometer)	1377:5, 3	D2435	Consolidation characteristics of as-compacted soil-fill or on undisturbed sub-grade samples	Disturbance problems in sensitive materials. Allows vertical drainage only, unrealistic in some structured materials.	For radial drainage consolidation and for undisturbed materials the use of the Rowe cell procedures is recommended (BS 1377
Pinhole Test	1377:5, 6.2	D4647	Laboratory assessment of soil dispersion.	Based on empirical evaluation of material performance, mainly in temperate materials.	D4221 – Double hydrometer test based on comparison of gradings before and after artificial dispersion. Needs PI>4.
Crumb Test	1377:5, 6.3		Laboratory assessment of soil dispersion.	Based on empirical evaluation of material performance, mainly in temperate materials	Physical observational tests associated with chemical tests for sodium cations in pore water.
Compaction	1377:4, 3.3-7	D698 & D1557	Simple test. Basis of control on site compaction of fill and pavement materials	Zero air voids a function of particle density- highly variable in tropical soils. Be aware of relationships between "laboratory" and 'engineering' moisture and between laboratory and site compactive effort.	See Table 6.16 for range of alternative procedures. Avoid drying of samples as much as possible and use fresh sample for each moisture point
Moisture Condition Value (MCV)	1377:5, 5.4-5		Assessment of material suitability. Easy to perform. Simple apparatus	Methodology not yet proven effective for fabric-sensitive residual soil materials.	Modifications for dry soils (Freer-Hewish 1984)
CBR	1377:4, 7	D1883	Quick and simple to perform. A convenient and widely established test for defining material suitability for road construction and subsequent quality control.	An empirical test only. Correlations with other parameters may be material-specific. Dependant on transient soil moisture-density-void ratio conditions. 3 test points are recommended	A range of conditions and procedures as shown in Fig. 6.14. ASTM allows for testing at a range of compactive efforts. A minimum of three (3) test points is recommended. MBR test for coarser aggregate in larger mould (Zakaria & Lees, 1998)

Table 6.4 Laboratory Simulation Tests

Simulation Tests	Standard Procedure		Advantages of Using Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	BS	ASTM			
Triaxial: UU (soil)	1377:7, 8-9	D2850	Unconsolidated UndrainedShort term fill analysis and cut-slope during construction	Unconsolidated Undrained. . Not strictly applicable for non-saturated conditions or for non-cohesive materials.	Single point tests at range of moisture contents for trafficability (Vaughan, 1995)
Triaxial: CU (soil)	1377:7	D4767	Consolidated Undrained (with pore pressure measurement).Enables long term effective stress analysis of cut slopes	Sophisticated test requiring careful supervision of experienced staff.	May need increased backpressures in some residual soils.
Triaxial: CD (soil)	1377:7		Consolidated Drained. Enables long-term effective stress analysis of fill slopes	Time consuming and sophisticated test requiring careful supervision of experienced staff	May need increased backpressures in some residual soils.
Unconfined Compressive Strength (Soil)	1377:7, 2	D2166	Quick and straightforward method of obtaining undrained shear strength.	Material needs to be intact, cohesive and at least stiff in consistancy.	
Vane Shear (Lab)	1377:7, 3	D4648	Very simple and rapid method for obtaining undrained shear strength. Can be used on materials in compaction or CBR moulds.	Only of use for soft saturated clays with no coarse particles. Small sample tested.	
Point Load Strength	[ISRM]		Simple test with portable equipment. Correlates with UCS	Sensitive to changes in moisture condition and surface crushing. Requires identical samples (10 min.). Correlations with compressive strength vary with different materials.	Tests at soaked, natural and dry moisture conditions
Unconfined Compressive Strength (rock)	[ISRM]	D2938	Straightforward test for measure strength of intact rock samples	Requires regular (core) shaped samples. Sensitive to changes in sample moisture condition, orientation and end-face preparation	Tests at soaked, natural and dry moisture conditions
Schmidt Hammer	[ISRM]		Very simple portable field test	Modified from concrete test. Correlations with strength require confirmation for each rock type.	Laboratory procedures specify "L" type hammer; alternative "N" hammer requires separate correlation.

Table 6.4 Laboratory Simulation Tests (Continued)

Simulation Tests	Standard Procedure		Advantages of Using Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	BS	ASTM			
Aggregate Impact Value (AIV)	812: 112		Simple test with inexpensive portable equipment giving a basic index parameter for aggregates	Flakiness, elongation can influence results as well as base-floor condition. Tests limited grading. Measures breakdown below 2.36mm only	Soaked/unsoaked tests. AIV(R) value measures breakdown from 10-2.36mm (M)AIV limits blows for weaker materials. Ethylene glycol soaking may be appropriate for some materials such as basic igneous roc
Aggregate Crushing Value (ACV)	812:110		Gives basic index parameter for aggregates commonly used in specifications.	Flakiness, elongation can influence results as well as base-floor condition. Tests limited grading. Measures breakdown below 2.36mm only. Requires compression test machine.	Soaked/unsoaked tests. ACV(R) value measures breakdown from 10-2.36mm
10% Fines Aggregate Crushing Tests	812: 111		Modification of ACV test, more generally used, particularly for weaker materials.	As for ACV	Soaked/unsoaked tests. Ethylene glycol soaking may be appropriate for some materials such as basic igneous rocks.
Sulphate Soundness	BS 812: 121	C88	Assesses aggregate durability as a response to repeated crystallization and rehydration stresses. Incorporated in many specifications	Time consuming. Poor repeatability and reproducibility unless great care taken over procedures.	Magnesium sulphate may be preferred to sodium sulphate because of greater penetrating power of the saturated solution.
Slake Durability	[ISRM]	D4644	Simple assessment of durability of rock-like material.	Not generally used a suitability parameter in specifications. Fragile materials require careful handling	Use with plasticity index for argillaceous materials. D3744: Durability Index- separate fine and coarse tests. Washington Degradation test (WSSHD 113A),
Los Angeles Abrasion (LAA)		C131/535	Standard combined impact and rolling abrasion test. Commonly used as a specification parameter	For aggregate <37.5mm. Tests a specified grading only. Measures breakdown in terms of material passing 1.68 mm sieve only	ASTMC535 for aggregate >19mm
Micro-Deval		[NF P 18-572]	Similar to LAA test but is also used to define surface aggregate suitability. Smaller equipment than LAA	Measures breakdown in terms of material passing 1.6 mm sieve only	Can be reported “dry” : MD:S or “wet” MD:E
Durability Index	[NITTR]			Whole grading. Durability index based on plasticity of degraded fines	Based on very similar Texas Ball test (Texas 110E). .

Table 6.4 Laboratory Simulation Tests (Continued

Simulation Tests	Standard Procedure		Advantages of Using Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	BS	ASTM			
Accelerating Polishing Test	812:114	E303	Means of assessing the tendency for aggregate to polish. Polished Stone Value (PSV) commonly incorporated into surfacing aggregate specifications.	Difficult and time-consuming test not normally carried out in standard laboratories. Selected aggregate pieces only.	D3319. PSV based on accelerated polishing machine. E660 based on small wheel circular track polishing machine.
Aggregate –Bitumen Adhesion		D1664	Tests for assessing adhesion of bitumen to aggregate in water	Observational test only. Takes account of stripping only and not prior coating difficulties.	D2849. Degree of particle coating. Adhesion may be indirectly assessed by mineralogical examination.
Aggregate Abrasion Value (AAV)	812: 113		Means of assessing surface wear in surfacing aggregates.	Selected aggregate pieces only.	Can be related to the AIV

Table 6.4 Laboratory Simulation Tests (Continued)

Chemical Tests	Standard Procedure		Advantages of Using Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	BS				
pH	1377:3, 9	E70	BS.Electrometric: Standard method, accurate to 0.1pH.	Requires regular cross-checking against buffer solutions	Use of Indicator papers - simple and quick, approximate values only Colourmetric method requires comparison with standard charts
Sulphate Content	1377:3, 5.2-5.5		Total sulphate in soils, including water- soluble calcium sulphate. Accurate if performed with care.	If measured sulphate content is >0.5% the water soluble sulphates should also be measured	Water soluble sulphate in soil and sulphates in water also by gravimetric (1377:3, 5.6) and ion exchange(1377:3, 5.5) methods.
Organic Content	1377:3, 3	C40	BS dichromate oxidation method. Accurate and suitable for all soils. Fairly rapid test.,	Presence of chlorides influences results, a correction can be applied.	Peroxide oxidation - used to eliminate organic matter for PSD testing
Carbonate Content	1377:3, 6.3	D4373	BS Rapid titration for carbonate content greater than 10%, has 1% accuracy.	Not suitable for carbonate content <10%. ASTM utilises gas pressure method	Gravimetric 1377 3:6.4. Used for hardened concrete. D4373 solubility in HCL. Calcimeter: simple, quick - approximate but adequate for most engineering purposes
Chloride Content	1377:3, 7.2-7.3	D1411	BS Silver nitrate method. Designed for concrete aggregate testing purposes	Titration process requires proper chemical facilities	Water Soluble: 1377 3:7.2; 812.117 BS Acid Soluble: 3:7.3. D1411, Calcium and magnesium chloride in graded aggregate.
Loss on Ignition (LOI)	1377:3, 4		Destroys all organic matter. Applicable for sandy soils containing little or no clay	High temperature may break down water of crystallisation in some minerals and give misleading results.	

Table 6.5 Soil and Aggregate Chemical Tests

Petrographic Procedure	Procedure Description	Procedure Application
Aggregate: Qualitative Visual Examination	Record general character of aggregate sample including grading, texture, shape and rock type	A quick and rapid assessment
Aggregate: Quantitative Visual Examination	Sieve into separate size fractions and examine each fraction in terms of grading, texture, shape, rock type and mineralogy. Utilise additional procedures set out below as appropriate	Detailed petrographic procedure for identification of weak and/or unsuitable materials and recognition of potentially deleterious minerals.
Methylene Blue Value	Based on absorption of methylene blue by clay minerals. Powdered rock or fine soil sample suspended in solution and then titrated with methylene blue.	Rapid method of indicating the presence of deleterious clay minerals Does not give any indication of mineral type. May need additional fabric assessment work for more reliable results.
Binocular Microscopy	The use of plane light binocular microscope requires little sample preparation. Small hand-held microscopes can be used in the field.	A quick and straightforward method for the examination of soil fabric and texture of hand specimens. Photographs can be easily taken to support descriptions
Thin Section Microscopy	The traditional geological method of examination of mineralogy and fabric of thin sub-samples of hand specimens under both plane and polarized light.	May be used for the examination of fabric and as a means of establishing mineral composition by point-count techniques. Difficult to make sections in friable materials. Possible to take photographs.
Scanning Electron Microscope (SEM)	Utilises a focused beam of electrons to scan a specially prepared sample. Some electrons back scattered others produce secondary electrons. Patterns can be captured on film for observation.	Needs careful operation to achieve meaningful results. Most useful in finer grained soils. The use of stereoscopic photographic pairs of photographs increases the effectiveness of interpretation.
X-Ray Diffraction (XRD)	Utilises the identifiable diffraction angle that X-rays make with differing minerals. Powdered samples may be oriented, non-oriented, dried, glycolated or heated to aid identification.	Widely used in the identification of tropical soil mineralogy. By itself this method is only semi-quantitative. Can not identify non-crystalline clay minerals
Thermal Analysis	Based upon whether the thermal reaction, which occurs as a clay mineral is heated is exothermic or endothermic and an interpretation of the resulting diagram.	These methods may be usefully employed in the study of clays and may provide useful mineralogical information in conjunction the methods listed above.

Table 6.6 Petrographic Assessment Procedures

Simulation Tests	Primary Properties Measured	Additional Influencing Properties
Volume Change	Potential for swell/collapse	Mineralogy, fabric
Compaction	Compactive effort-moisture content-density relationships	Particle grading density, shape, strength. Fabric.
CBR	Relative bearing strength at defined compactive states	Particle grading, shape. Saturation
MCV	Compactive effort moisture content relationships	Particle grading density, shape, strength. Fabric.
Triaxial	Compressive strength, consolidation.	Saturation, fabric, discontinuities
Point Load	Index compressive strength, anisotropy	Saturation, fabric (fissility)
Schmidt Hammer	Relative strength	Saturation. Fabric. Discontinuities
Soundness	Relative durability	Mineralogy. Fabric. Porosity
Slake Durability	Rock lump resistance to wetting/drying and abrasion	Mineralogy. Fabric. Porosity
AIV	Resistance of size fraction 10-14mm to direct dynamic impact	Saturation. Particle shape. Fabric
ACV/10%FACT	Resistance of size fraction 10-14mm to direct crushing pressure	Saturation. Particle shape. Fabric
LAA	Resistance to rolling and impact abrasion.	Saturation. Particle shape. Fabric
Texas Ball Mill	Particle resistance to wet rolling abrasion	Saturation. Particle shape. Fabric
PSV	Relative resistance to polishing	Mineralogy
Stripping Test	Adhesion of bitumen to aggregate faces.	Mineralogy

Table 6.7 Properties Measured by Laboratory Simulation Testing

Potential problem	Typical example
Correlation between similar tests from different standards that may have apparently slight but significant different aspects	The maximum sand size in ASTM/AASHTO psd tests is defined as 4mm as opposed to the British Standard 2mm
Correlation between similarly named tests with differing procedures	Comparison between liquid limit test performed with drop cone and those undertaken using the Cassegrande apparatus
Use of data from tests that are not covered by a uniformly accepted procedure.	Petrographic examination and mineralogical tests are generally not covered by widely accepted procedures.
Correlation between index test results and engineering behaviour based on historical precedent.	Relationships between SPT, shear strength and in situ CBR.
Test interpretation founded on assumptions of material behaviour, based on temperate soil or rock behaviour.	The common adoption of a laboratory drying temperature of 100-110^{O} C for moisture content determinations. Some tropically occurring minerals (e.g. halloysite and allophane) contain intra-crystalline water that is driven off below 100^{O} C
The use of local or regionally developed "modifications" to test procedures.	Central African Standards for compaction and CBR assessment

Table 6.8 Problems Posed By Conflicting Standards

..European Standard No.	Title
TESTS FOR GENERAL PROPERTIES OF AGGREGATES	
prEN 932-1	Methods for sampling
prEN 93 2-2	Methods for reducing laboratory samples
prEN 932-3	Procedure and terminology for simplified petrographic description
prEN 93 2-4	Quantitative and qualitative procedures for description and petrography
prEN 932-5	Common equipment and calibration
prEN 93 2-6	Definitions of repeatability and reproducibility
prEN 932-7	Conformity criteria for test results
TESTS FOR GEOMETRICAL PROPERTIES OF AGGREGATES	
prEN 933-1	Determination of particle size distribution - Sieving method
prEN 93 3-2	Determination of particle size distribution - Test sieves, nominal size of apertures
prEN 93 3-3	Determination of particle shape - Flakiness index
prEN 93 3-4	Determination of particle shape - Shape index
prEN 933-5	Determination of the percentage of crushed and broken surfaces in coarse aggregates
prEN 93 3-6	Determination of the flow coefficient in aggregates
prEN 93 3-7	Determination of shell content - Percentage of shells in coarse aggregates
prEN 933-8	Assessment of fines - Sand equivalent test
prEN 933-9	Assessment of fines - Methylene blue test
prEN 933-10	Assessment of fines - Grading of fillers (air jet sieving)
TESTS FOR MECHANICAL AND PHYSICAL PROPERTIES OF AGGREGATES	
prEN 1097-1	Determination of the resistance to wear (micro-Deval)
prEN 1097-2	Method for the determination of resistance to fragmentation
prEN 1097-3	Determination of loose bulk density and voids
prEN 1097-4	Determination of the voids of dry compacted filler
prEN 1097-5	Determination of water content by drying in a ventilated oven
prEN 1097-6	Determination of particle density and water absorption
prEN 1097-7	Determination of particle density of filler - Pyknometer method
prEN 1097-8	Determination of the polished stone value
prEN 1097-9	Determination of the resistance to wear by abrasion from studded tyres test -Nordic test
prEN 1097-10	Water suction height
TESTS FOR THERMAL AND WEATHERING PROPERTIES OF AGGREGATES	
prEN 1367-1	Determination of resistance to freezing and thawing
prEN 1367-2	Magnesium sulfate test
prEN 1367-3	Boiling test for “Sonneubrand basalt” and disintegration of steel slags
prEN 1367-4	Determination of drying shrinkage
prEN 1367-5	Determination of resistance to thermal shock

Table 6.9 European Standards for Aggregates

European Standard No.	Title
prEN 13285-1	Unbound mixtures specifications
(Pre-Enquiry Stage)	Cement bound mixtures - Specifications for soil cement and cement granular mixtures
93/3 963	Unbound and hydraulically bound mixtures - Test methods for reaction to frost (frost heave and freeze/thaw behaviour)
prEN 13286-1	Unbound and hydraulically bound mixtures - Part 1: Test methods for laboratory reference density and moisture content - introduction and general requirements
prEN 13286-2	Unbound and hydraulically bound mixtures - Part 2: Test methods for laboratory reference density and moisture content - Proctor compaction
prEN 13286-3	Unbound and hydraulically bound mixtures - Part 3: Test methods for laboratory reference density and moisture content - Vibrocompression with controlled parameters
prEN 13 286-4	Unbound and hydraulically bound mixtures - Part 4: Test methods for laboratory reference density and moisture content - Vibrating hammer
prEN 13286-5	Unbound and hydraulically bound mixtures - Part 5: Test methods for laboratory reference density and moisture content - Vibrating table
prEN 13286-6	Unbound and hydraulically bound mixtures - Part 6: Test methods for sampling and sample preparation
93/3 968	Unbound and hydraulically bound mixtures - Part 7: Cyclic load triaxial test for unbound mixtures
(Pre-Enquiry Stage)	Unbound and hydraulically bound mixtures - Part 40: Test method for determination of the direct tensile strength of hydraulically bound mixtures
93/4025	Hydraulically bound mixtures - Test methods for compressive strength
93/4026	Hydraulically bound mixtures - Test methods for the determination of indirect tensile strength
93/4031	Hydraulically bound mixtures - Test methods for modulus of elasticity
99/1224	Unbound and hydraulically bound mixtures - Test method for the determination of the bearing capacity - California Bearing Ratio (CBR), immediate bearing index and linear swelling
99/1220	Unbound and hydraulically bound mixtures - Part 44: Test method for the determination of binder activity - Alpha coefficient of vitrified blastfumace slag
93/122 1	Unbound and hydraulically bound mixtures - Test method for the workability period
99/1222	Unbound and hydraulically bound mixtures - Test method for the determination of compactibility - Moisture Condition Value (MCV)

Table 6.9 European Standards for Aggregates (Cont'd)

Material	Key Inherent Characteristic(s)	Tests Given Particular Importance
Basic Igneous	Development of secondary minerals due to weathering; leads to potentially poor durability.	Petrographic assessment.
Acid Igneous	Coarse crystalline fabric, especially feldspars leading to poor bitumen adhesion.	Stripping tests
Pyroclastic	May contain deleterious mineral. Possibility of high void content ash materials. Variability.	Petrographic assessment. Water absorption. Particle density.
Clastic Sedimentary	Mineral content and lower strength/durability of matrix as compared to particles. Variability in bedded sequences. Degradation/slaking of argillaceous materials.	Petrographic assessment. Slaking test. Abrasion.
Non-Clastic Sedimentary	Poor strength/durability in non-crystalline materials. Polishing of some limestone aggregate.	Aggregate abrasion, impact strength. PSV.
Metamorphic-Foliated	Anisotropic character. Foliated fabric. Platy minerals (muscovite, chlorite)	Aggregate shape. Petrographic assessment
Metamorphic -Non Foliated	Mineralogy in rocks derived from argillaceous sedimentary parent.	Slaking/durability tests
Sands & Gravels	Variability in character of clasts, particularly in deposits containing differing rock types.	Petrographic assessment. Shape
Residual Soils	Non-standard mineralogy, relict and voided fabric.	Non-standard moisture content/Atterberg limit procedures. Collapse-swelling tests
Duricrusts	Variability. Weak clast strength.	Degradability. Particle crushability. Particle density variation.

Table 6.10 Major Material Types: Key Inherent Characteristics

Areas of Competence	Definitions
Theoretical Knowledge	An adequate knowledge of the objectives of a test procedure and the properties it is aimed at defining.
Technical Awareness	The technical ability to carry out a specific test to the required standard without special supervision.
Data Reporting	The ability to report the results of test and where necessary undertake any required calculations and recognise gross errors.
Quality Awareness	An awareness of the accuracies required for a test and the safeguards required to ensure reliability and repeatability

Table 6.11 Materials Testing Staff: Competence Assessment

Reliability Level	Definition	Implication
1	Numbers as reported by the testing technician	Not to used without further verification
2	Numbers checked as being mathematically correct by the responsible technician.	For use by in-house engineering staff with caution. Not for contractual use.
3	Data checked as being geotechnically reasonable by a qualified engineer or laboratory manager.	For use by in-house engineering staff. For use contractually on an "indication" basis
4	Data cross-checked for internal site correlation and project scientific validity by a qualified engineer or laboratory manager	Data suitable for contractual use and project technical reporting.
5	Data cross-checked for site to site correlation and regional scientific validity by an experienced senior engineer.	Data suitable for regional extrapolation and international technical reporting.

Table 6.12 Definitions of Test Data Reliability

CATEGORY	DESCRIPTION	AVAILABILITY
Structural Water	Water held within the structure of component minerals	Generally not removable below 110^{o}C except for clays such as halloysite, allophane and gypsum.
Strongly Adsorbed Water	Held on particle surface by strong electrical attraction.	Not removed by drying at 110^{o}C.
Weakly Adsorbed Water	Held on particle surface by weak electrical attraction.	Can be removed by drying at 110^{o}C but not by air drying
Capillary Water ("Free" Water)	Held by surface tension	Removed by air-drying.
Gravitational Water ("Free Water)	Moveable water held in the material voids	Removable by drainage.

Table 6.13 Categories of Water Held in Soil

Material	Liquid Limit% Standard Cone	Liquid Limit % Large Cone	% Material Passing 425µ
Shale	95	80	93
Shale	90	85	100
Phyllite	76	69	75
Schist	80	60	80
Schist	52	45	95
Schist	39	33	50
Schist	81	76	100
Schist	76	67	85
Sandstone	66	47	65
M. Granite	35	29	33
M. Granite	35	30	31
M. Granite	53	41	46
F. Granite	59	43	43
F. Granite	72	49	48

Data from residual soil materials in Peninsular Malaysia

Table 6.14 Comparison of Some Liquid Limit Results for Small and Large Cones

Mineral	Particle Density Mg/m^3
Calcite	2.71
Feldspar- orthoclase	2.50 -2.60
Feldspar - plagioclase	2.61-2.75
Gibbsite	2.40
Haematite	4.90-5.30
Halloysite	2.20-2.55
Kaolinite	2.63
Magnetite	5.20
Quartz	2.65

Table 6.15 Common Mineral Particle Densities

Method	Reference	Comment
Measured dimensions. Hand trimmed from block or tube.	Part 2: 7.2 BS 1377 (1990)	Material has to be suitable for trimming; eg robust soil or weak rock.
Measured dimensions. Sample within tube.	Part 1: 8.4 BS 1377 (1990)	Used where extrusion may disturb sample; eg loose or weakly bonded soil material.
Water displacement (waxed sample)	Part 2: 7.4 BS 1377 (1990)	Simple test used for irregular shaped water sensitive samples.
Weighed in Water (waxed sample)	Part 2: 7.3 BS 1377 (1990)	As above, generally more accurate.
Weighed in water (non waxed sample)	ISRM (1981), Part 1	Used for irregular lumps of rock-like material not susceptible to swelling or slaking.

Table 6.16 Methods of Density Measurement

Index	Definition
Plasticity Modulus Plasticity Product Shrinkage Product	PI x % material passing 0.425mm sieve PI x % material passing 0.075mm sieve Bar Linear Shrinkage Limit x % material passing).425mm sieve
Grading Coefficient Uniformity Coefficient (U)	(% passing 26.5mm – %passing 2.00mm) x (% passing 4.74mm)/100 D_{60}/D_{10} The ratio of the 60% particle size to the 10% particle size

Table 6.17 Commonly Used Derived Grading Indices

Test Procedure	Mould Size	Rammer Mass (Kg)	Drop (mm)	Layers	Blows per Layer	Work Done (kJ/m3)
BS Light	1 Litre CBR mould	2.5 2.5	300 300	3 3	27 62	596 594
ASTM Standard	4" 6"	2.49 2.49	305 305	3 3	25 56	
BS Heavy	1 Litre CBR mould	4.5 4.5	450 450	5 5	27 62	2682 2672
ASTM Modified	4" 6"	4.54 4.54	457 457	5 5	25 56	
BS Vibrating Hammer	CBR mould	32-41 (downward force)		3	(1min.)	11739
West African	CBR mould	5.54	457	5	25	

Table 6.18 Compaction Test Procedures

Procedure	Standard Procedure		Comment on Use	Alternative/Additional Procedures
	BS	ASTM		
Sand Replacement Density	1377:9	D1556	Sand replacement (cylinder) the commonest method employed. Time consuming. Potential difficulties with unstable holes in granular materials	F-M soils hole dia. 100mm, C soils 200mm. Scoop method as BS alternative.
Core Cutter Density	1377: 9	D2937	Can only be used in cohesive soils free from coarse-grained materials.	Driven cylinder can also be used
Water Replacement Density	1377:9	D2167	Used effectively only in fine materials	BS procedure: hole lined with plastic sheet. ASTM procedure uses special calibrated balloon apparatus.
Nuclear Density	1377: 9	D2922(b)	Gamma radiation a function of material wet density. Source is inserted in small hole in ground. Requires careful calibration for each chemically variable soil type.	A separate method (ASTM D1037) may be employed to gauge the moisture content using same or similar equipment. D 2922 (a) uses backscatter method; over influenced by top 20-40mm of compacted layers.
CBR	BS 1377: 9		Generally not accepted as a means of monitoring construction specification compliance. Some correlation achieved with Lab CBR in conventional heavy clays only.	Indirect empirical methods such as DCP generally now used in preference. Correlation also possible with SPT blow counts.

Table 6.19 In Situ Material Control Tests

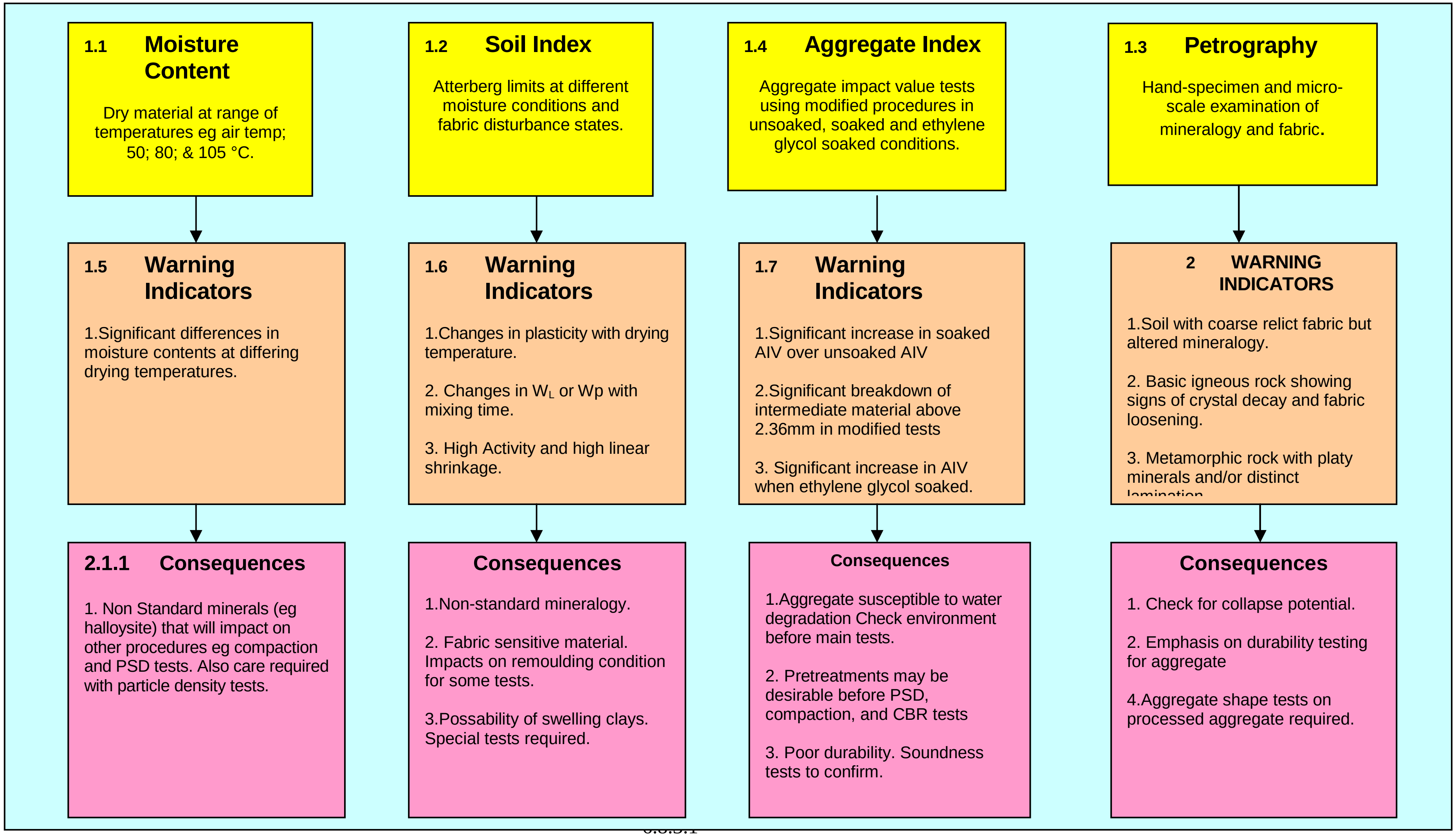


Figure 6.1 Preliminary Testing Programme and Consequence for Further Test Programmes

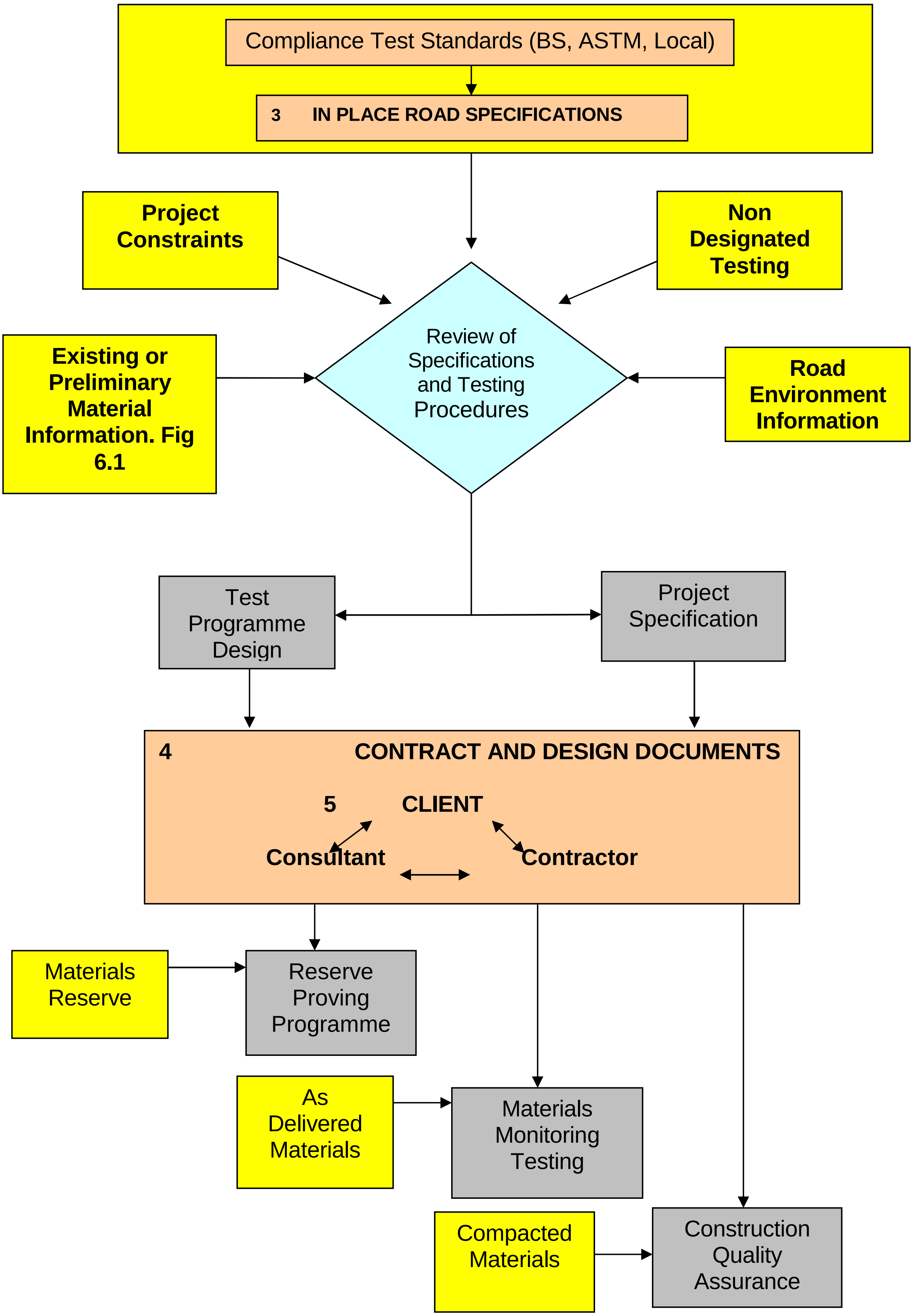
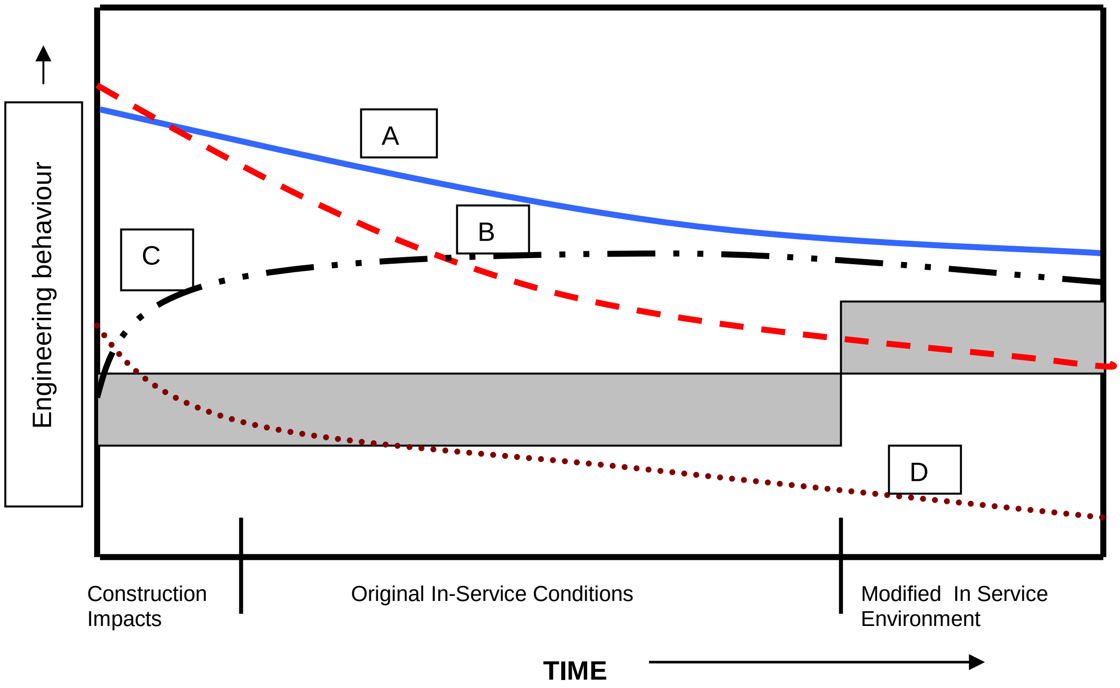


Figure 6.2 Typical Project Framework for a Materials Testing Programme



Notes

Engineering behaviour required by design = Wet Climate / Dry Climate

Material A: Degrades during whole-life of road but maintains adequate strength over in service stress

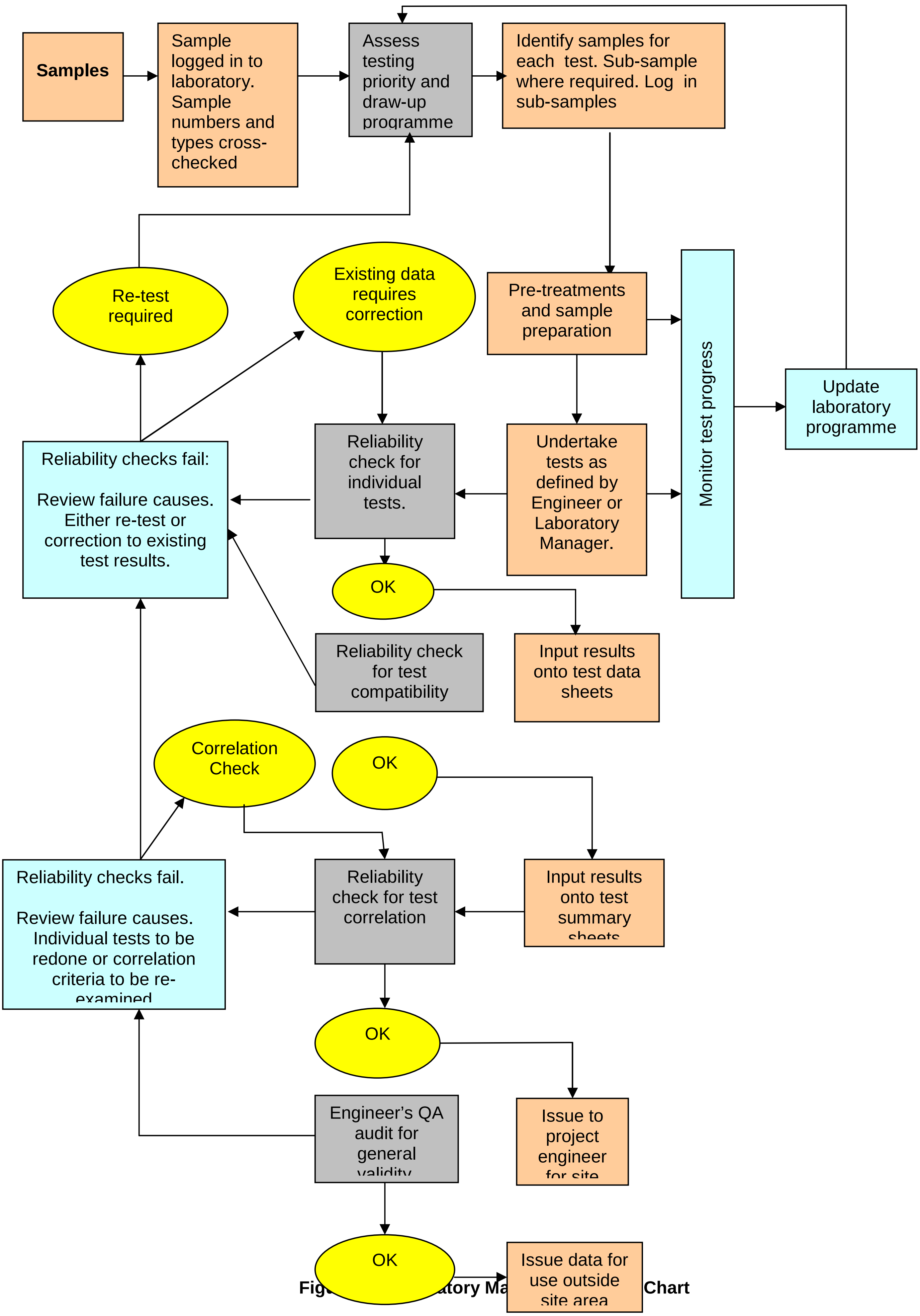
Material B Degrades but maintains sufficient strength for original design but not for modified environment

Material C Originally a marginal material but its degradation improves its engineering character and it then sufficient strength for the road working life

Material D Not acceptable or suitable

For a pavement aggregate material, problems with material B could be solved by overlaying or upgrading, for an earthwork material the problem is much more fundamental.

Figure 6.3 Schematic Representation of Whole-Road Life Material Durability Relationships



Fig… atory Ma… Chart

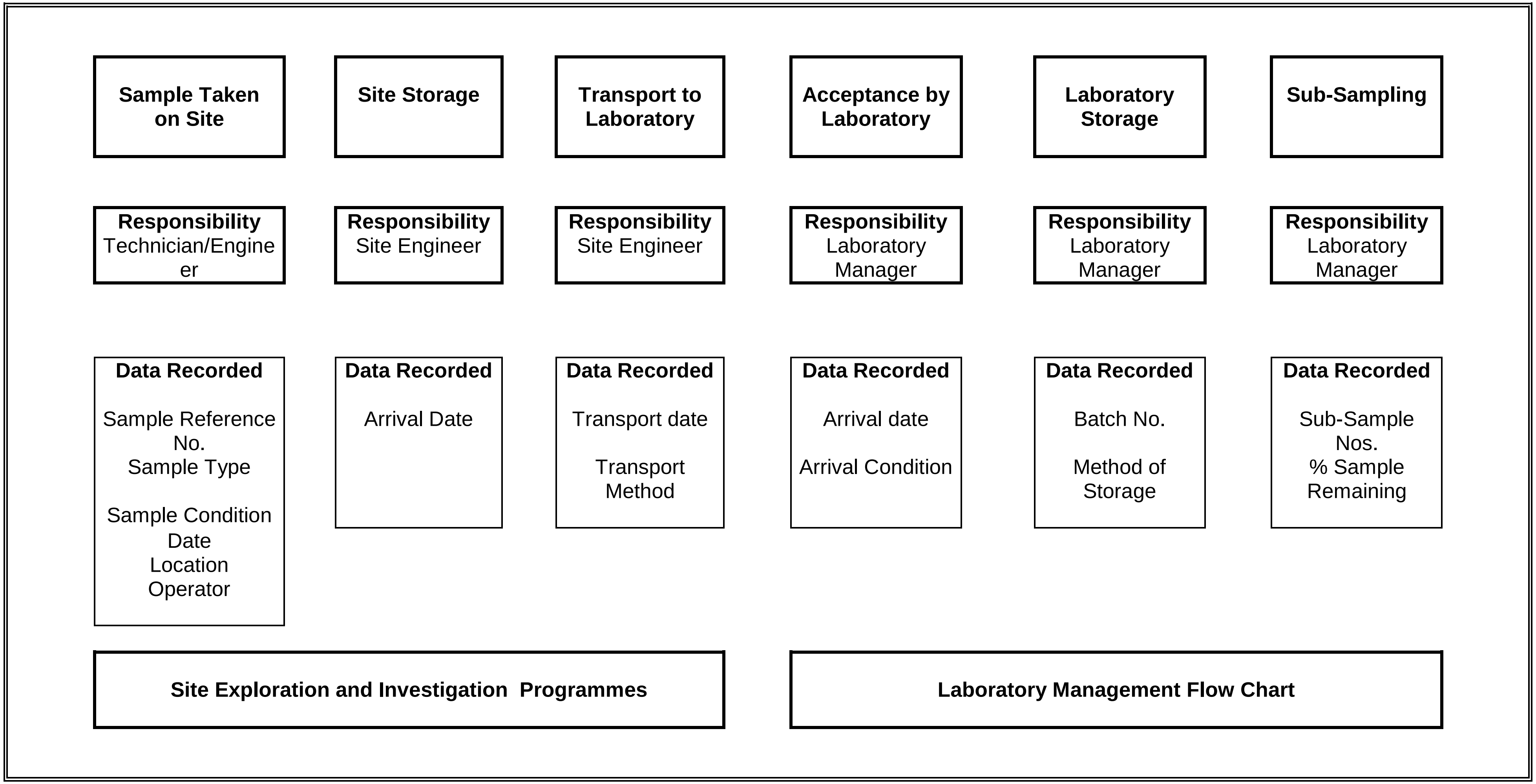


Figure 6.5 Stages in the Sample Tracking Procedure

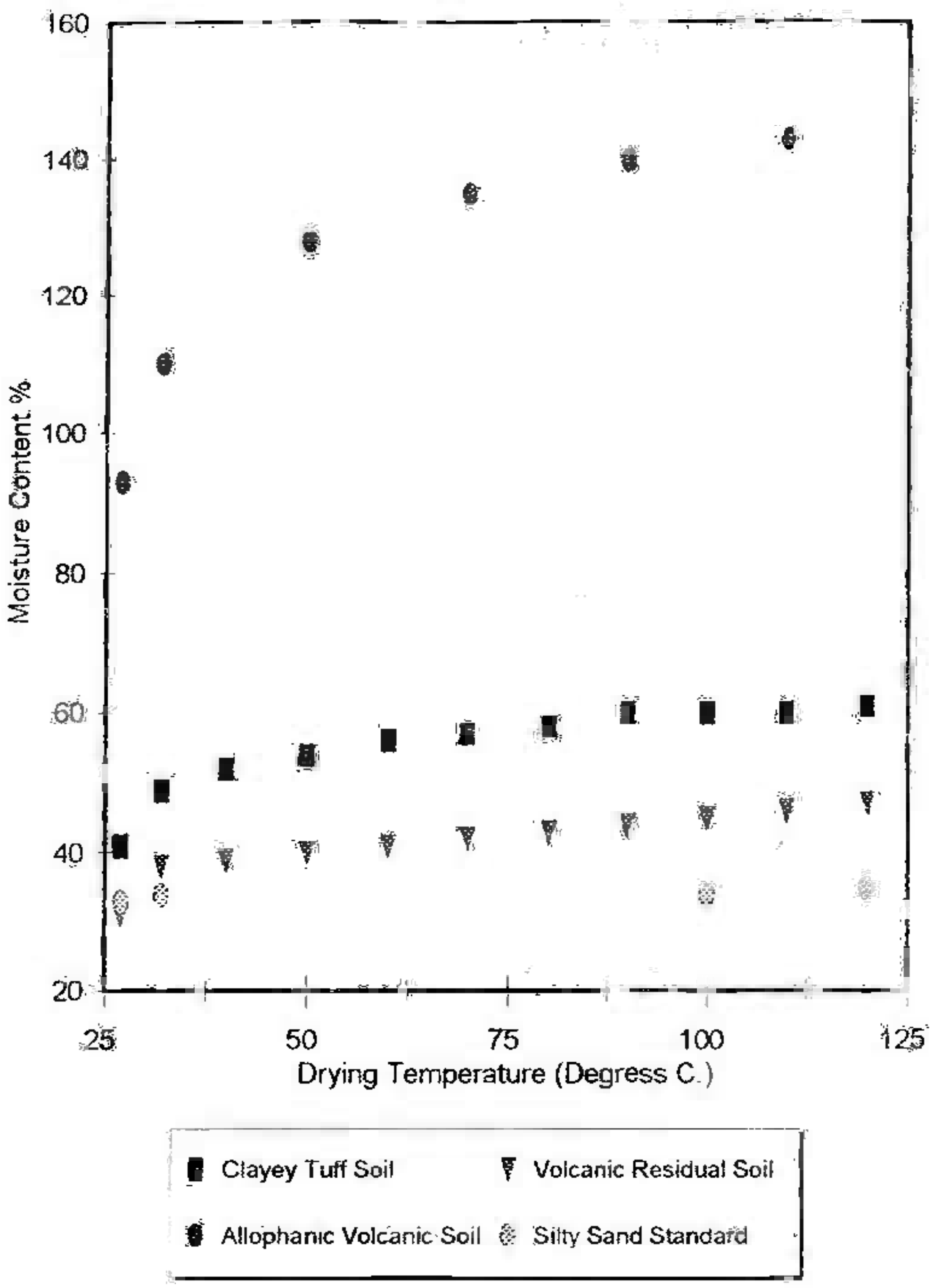


Figure 6.6 Typical Influence of Drying Temperature on Moisture Contents

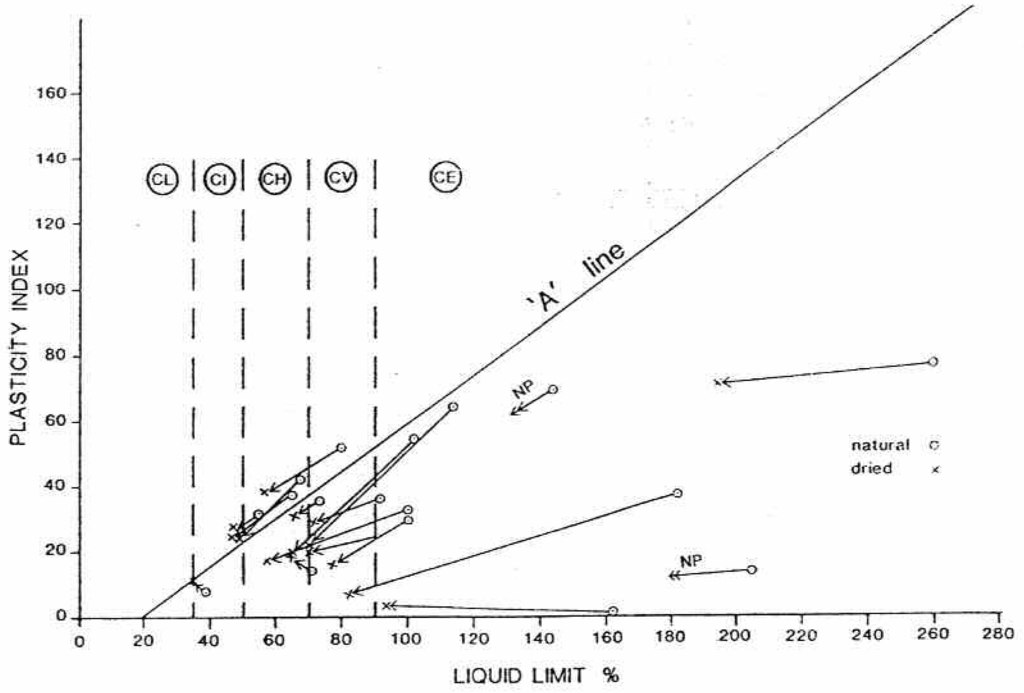


Figure 6.7 Effects of Drying on Plasticity for Tropical Residual Soils (Geol.Soc. 1997)

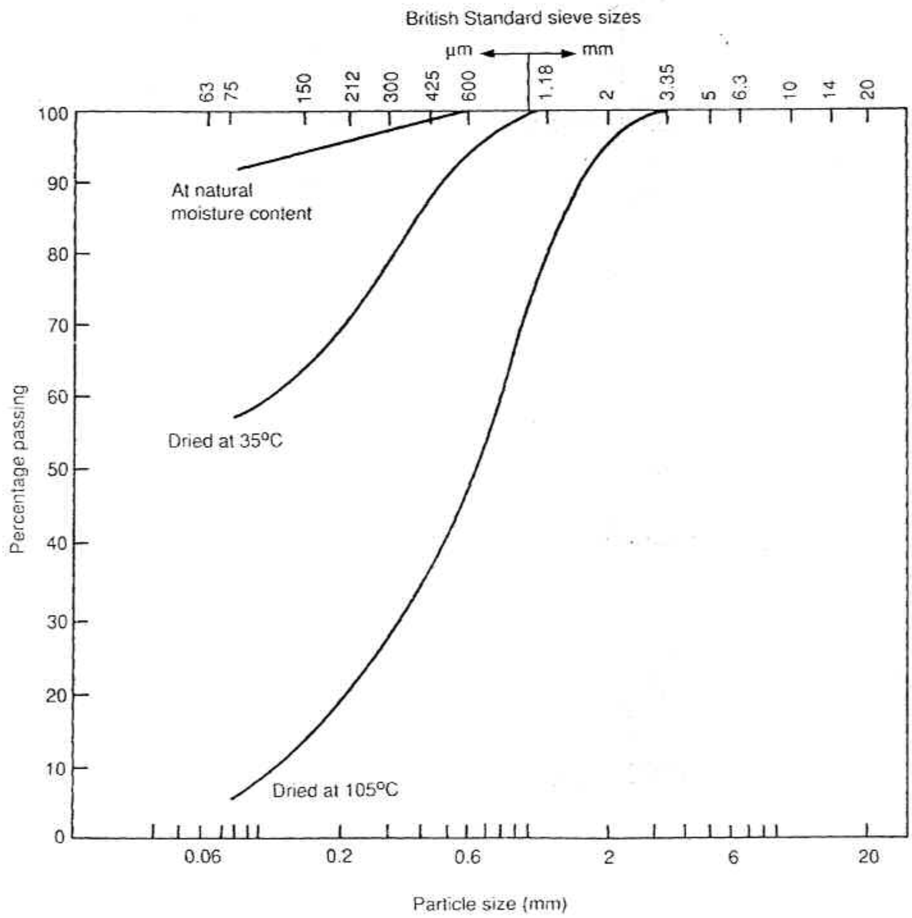


Figure 6.8 The Effect of Drying Temperature on PSD of a Weathered Volcanic Ash (Millard , 1990)

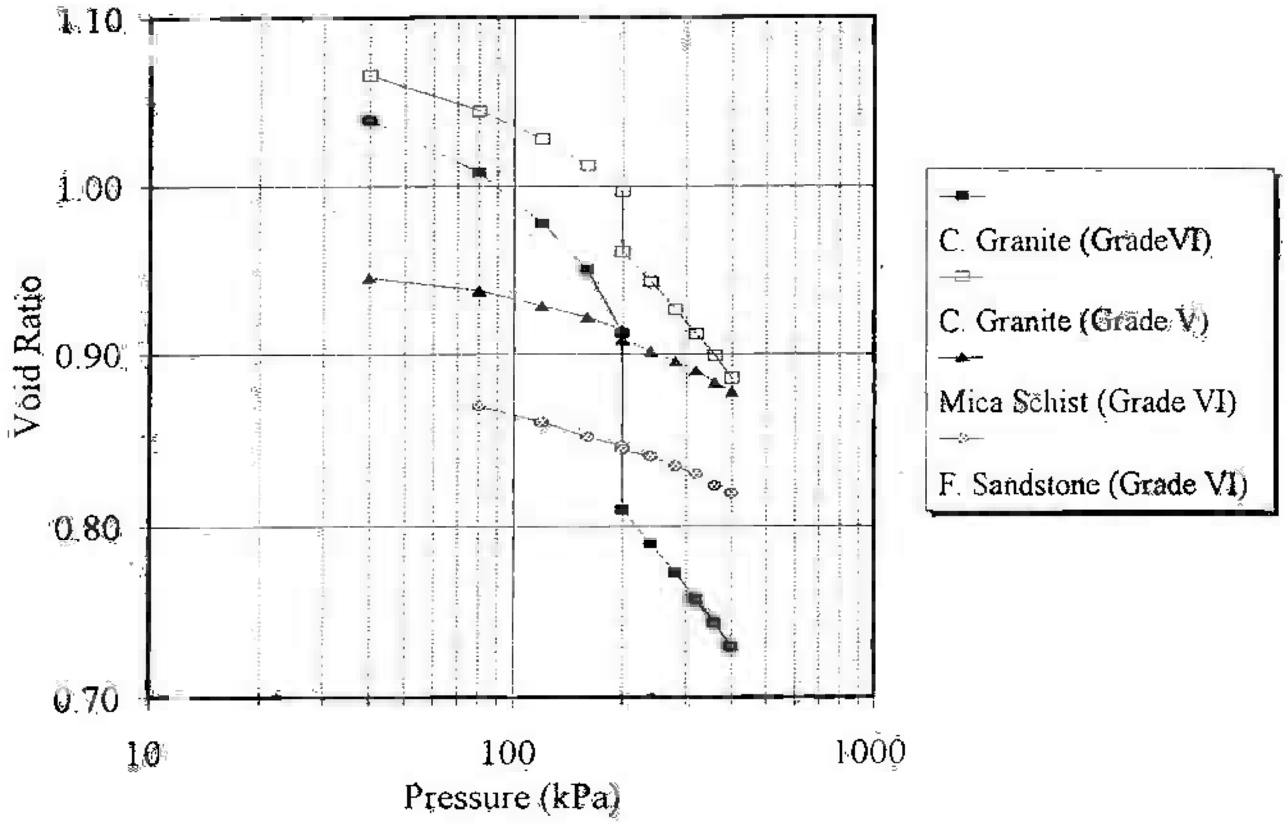
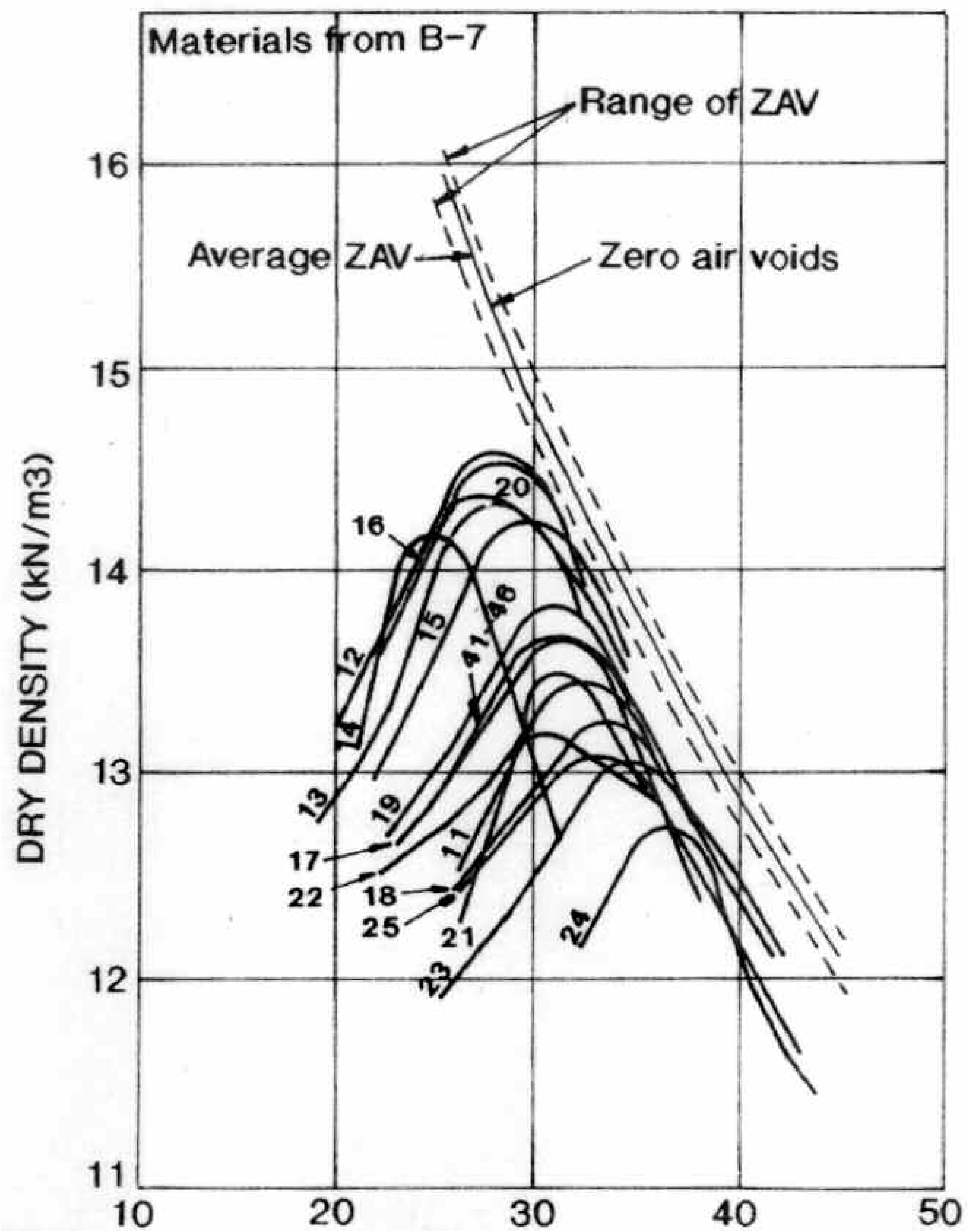


Figure 6.9 Typical Collapse Test Plots: Malaysian Residual Soils (Cook & McGown 1995)



6.10 Examples of Compaction Curves from a Single Borrow Pit Profile in Indonesia (Younger 1990)

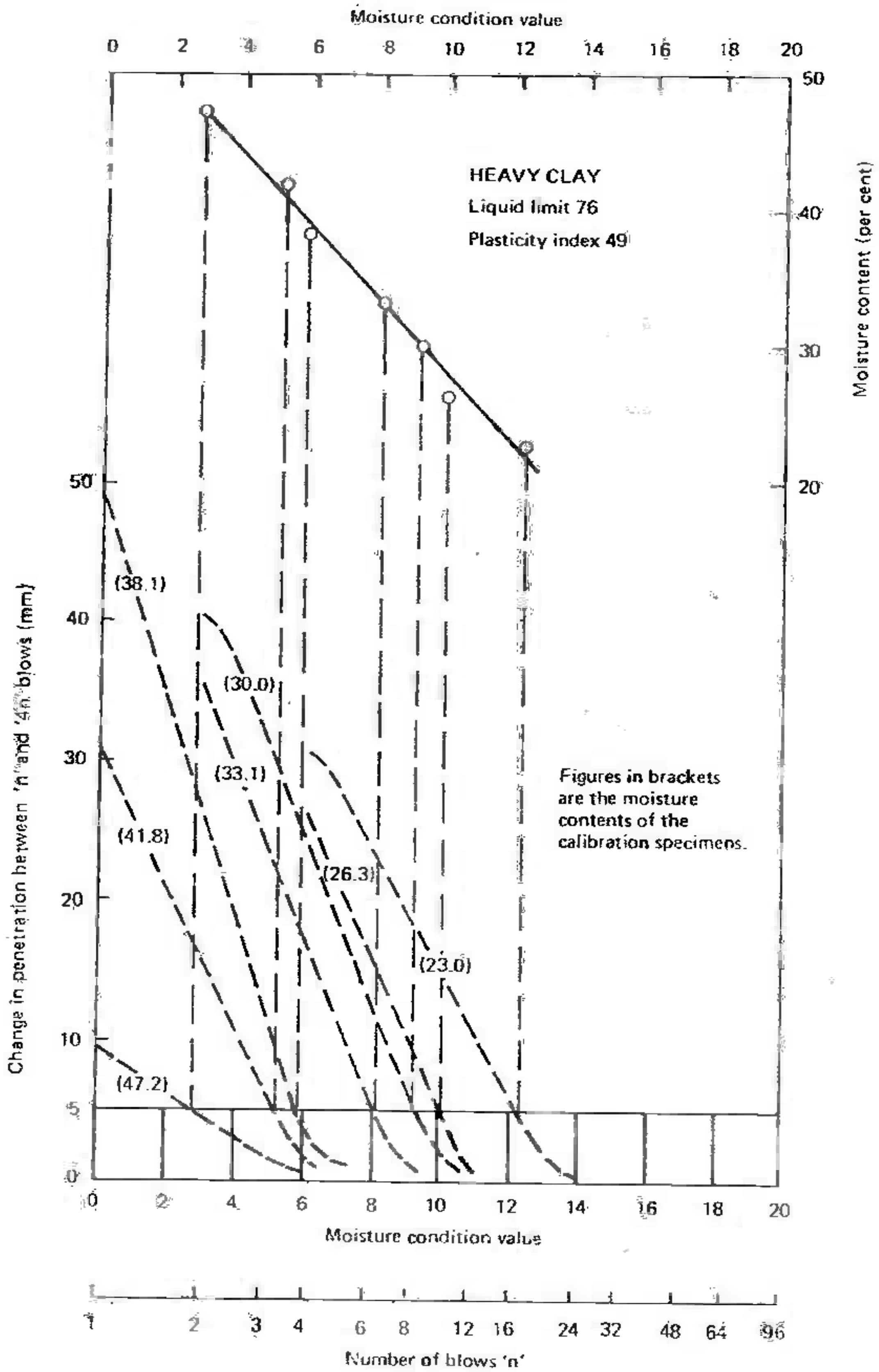


Figure 6.11 A Moisture Condition Value Calibration. (Parsons 1979)

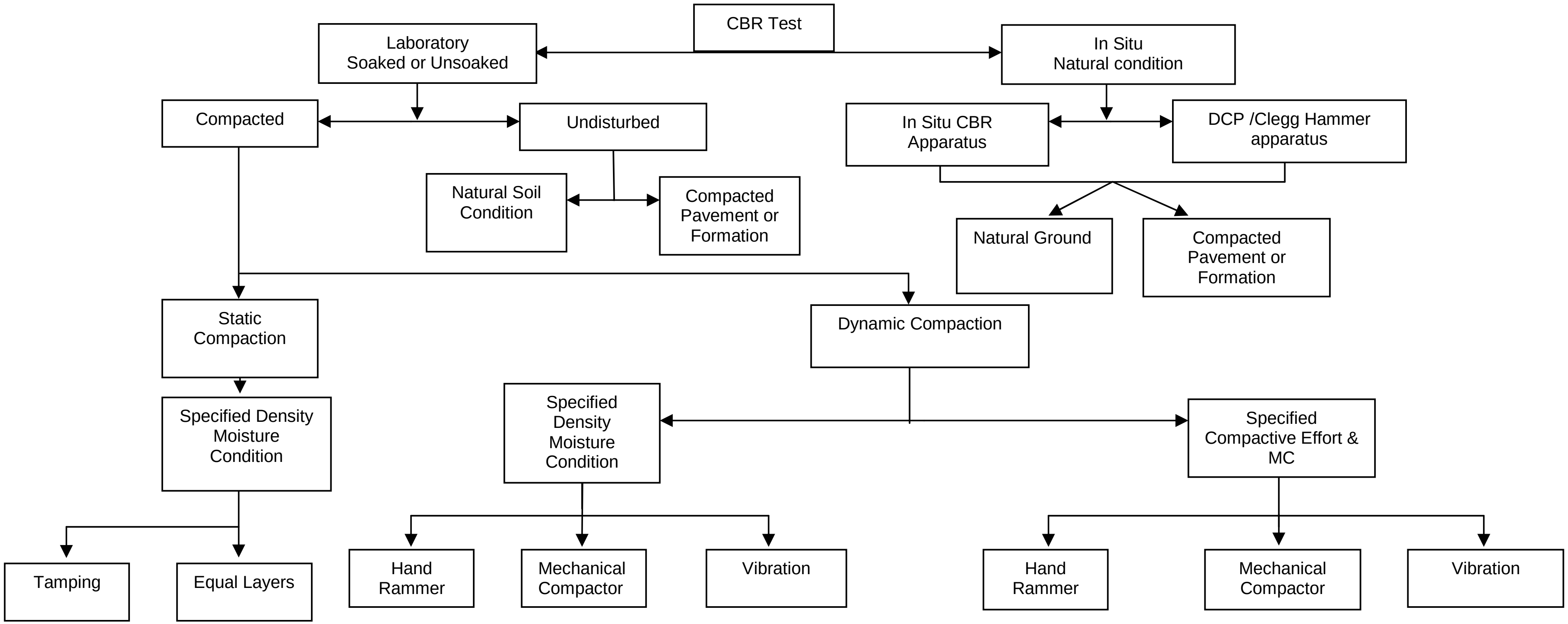


Figure 6.12 Variation in CBR Test Procedures

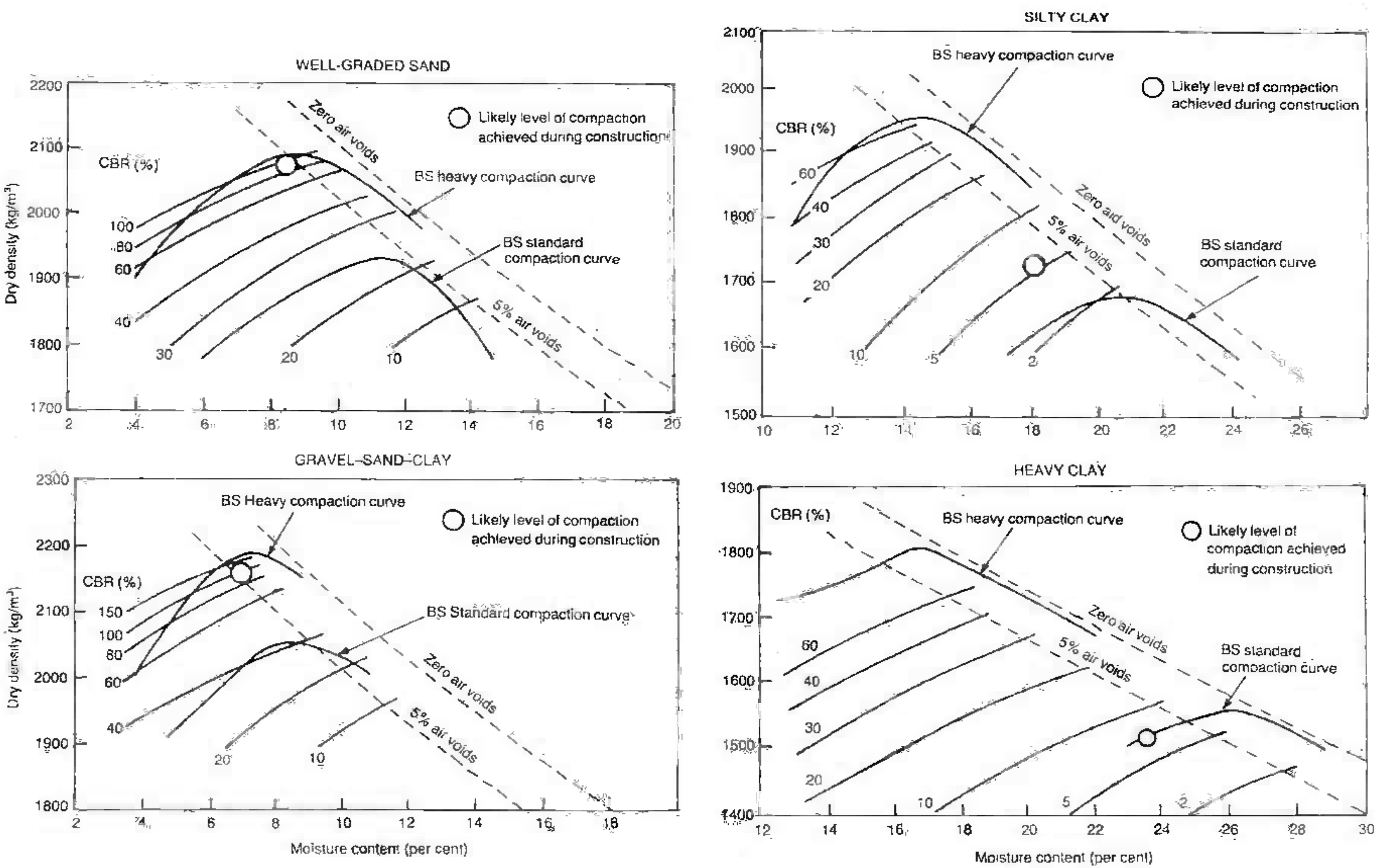


Figure 6.13 Typical Iso-CBR Plots. (Millard, 1990)

7 SAMPLING AND STATISTICS

7.1 Introduction

Sampling and testing of aggregates, and the precision of the test results have been described in detail in various books. Several of them (Smith and Collis, 1993 and Pike, 1990) and have been used as sources of information in producing this chapter.

7.2 Sampling

Statistically, a sample can be defined as an individual or group of individuals drawn from a large or infinite population. Therefore, it should be borne in mind that a sample is of a limited size and so will contain limited information. Also all sampling investigations are subject to experimental error. In order to keep this error to a minimum, careful consideration needs to be given to the design of the investigation.

The concept of a representative sample is frequently encountered in testing aggregates. In most cases this turns out to be nothing more than an average-looking sample which is deemed, without proper justification, to be representative of a quarry or a larger batch of aggregate, and suitable for testing to determine properties that are held to be typical of the original source.

Information from samples is only as representative of the material as the samples on which they are performed. If the variation is large then more and/or larger samples will be required. As larger samples are selected, the chances of obtaining a perfect sample increase steadily until the ultimate limit is reached when the whole of the original lot comprises the sample. Hence, the only way to be completely sure of obtaining a truly representative sample by a random method is to take the whole of the original lot. Clearly this is impossible for most practical purposes. Practical sampling entails taking a sample large enough to be representative within acceptable and known limits, but small enough for convenience in handling.

Individual samples may be taken from a range of locations:

1. From exploration cores/samples
2. From in situ exposures (near surface)
3. From quarry/pit faces
4. As dug material
5. After processing
6. From stockpile
7. During construction
8 After construction
9. After in-service effects

It is preferable for samples to be representative of the material in the condition for which information is required. Material sampled in situ form a pit or quarry, for example, may not be representative of its nature when placed within the road or in service. If it is not possible to get truly representative samples then it is necessary to be clear as to the assessment consequences of any condition mis-match.

An impartial assessment of likely sample condition in relation to in service material condition is essential. This assessment should not be influenced either by optimistic estimations of lack of disturbance..

Individual samples should be of sufficient size for the proposed test programmes. Table 7.1 indicates typical sample sizes for common test procedures. It should be noted that in the case of some tests, such as for moisture-density relationships, on tropical materials there may be a need for using fresh material at each stage of the test and a greater volume of material will be necessary.

7.3 Statistics

7.3.1 General

Naturally occurring road construction materials have in built variability stemming from their geological origins and subsequent near surface earth process such as weathering, erosion and transportation. The processes of sampling and testing such materials add additional variability.

Test data representability, the confidence that can be placed in individual results or groups of results and the inferences that can be drawn from comparisons of groups of results are important issues in the assessment of construction materials information. The following sections outline basic statistical concepts of use in this field, whilst Table 7.2 presents typical examples of their application.

7.3.2 Means and Variation

The most common value obtained from a set of observations is the average or *arithmetic mean*, usually denoted by $\overline{X}$. Calculated as follows:

$$\overline{X} = \sum_{i=1}^{n} \frac{x_i}{n}$$

where x_i represents the observed values and n is the number of observations.

In addition to the average, it is important to know the distribution of the observations; i.e. the amount by which each observation deviates from the average. The most important measure of dispersion is the *standard deviation*, usually denoted by σ calculated for a sample as follows:

$$\sigma = \sqrt{\frac{\sum\left(x_i - \overline{X}\right)^2}{n-1}}$$

The variance is simply the square of the standard deviation.

In some instances it may be necessary to test whether one distribution is more variable than another. In such cases it is necessary to compare the *relative* variability. A relative measure of variation is given by the *coefficient of variation*, (γ), i.e. percentage standard deviation, and is calculated as:

$$\gamma = \frac{100\sigma}{\overline{X}}$$

7.3.3 Normal distribution

The Normal distribution is commonly used to represent data sets using the parameters $\overline{X}$ and σ. In order to make all such distributions comparable with each other, these parameters are standardised. This is done as follows:

- The mean of each distribution is regarded as having the value zero.
- All deviations from this mean are measured in terms of the standard deviation of the distribution (not in terms of the original units).
- The total area under the curve is taken as unity; i.e. representing total probability

The Normal curve asymptotes to the x-axis, but for most practical purposes, it may be regarded as ending at three standard deviations on either side of the mean, as illustrated in Figure 7.1. As shown in Figure 7.1, the probability that an observation lies within 3σ of the mean is 99.7%, 95.5% probability within 2σ and 68% probability within 1σ of the mean.

Care should be taken when using statistical tables to look up probabilities for a Normal distribution. It is common for probabilities for only *one tail* of the distribution to be quoted. For example, the probability that an observation is *greater* than $\overline{X}$ + 1σ is 16%. This can be derived from Figure 7.1 as follows. There is obviously a 50% probability that an observation is below the mean. There is a further probability of 34% (½ of 68%) that an observation lies between the mean and +1σ giving a total probability of 84% that the magnitude of the observation is less than the value of the mean plus one standard deviation. This gives a probability of 16% (100% - 84%) that the observation is in *excess* of $\overline{X}$ +1σ.

7.4 Sample size

As stated earlier, the larger the sample the closer the sample mean should be to the population (grand) mean. As the sample size increases, its Normal characteristics should remain, but its standard deviation decreases as the square root of the number of observations (n).

Denoting the standard deviation for the averages of samples of n items by σ and the standard deviation for the distribution of the individual items by σ_n then

$$\sigma_n = \frac{\sigma}{\sqrt{n}}$$

The quantity σ_n is referred to as the *standard error of the mean*.

For large samples, it is reasonable to assume that the sample standard deviation σ_n is equal to the standard deviation of the population σ. Using the sample mean $\overline{x}$ and referring to Figure 7.1, one can say that the probability that the population mean lies in the range $\overline{x}$ +/- 1σ is 68% and in the range $\overline{x}$ +/- 2σ is 95.5%. In this manner confidence limits can be established.

7.5 Repeatability and Reproducibility

Important considerations in carrying out a programme of tests are the notions of repeatability and reproducibility (BS812, 1975).

Repeatability is a measure of the random error associated with a single operator obtaining successive results on identical material, with the same equipment and constant operating conditions. It is the difference between two single results, which would be expected to be exceeded in only one case in twenty, assuming normal and correct operation of the test. This is expressed as:

$$r = 1.96 \sqrt{2\sigma_1}$$

where σ_1 is the single operator standard deviation within a laboratory.

Reproducibility is a measure of the random error associated with test operators working in different laboratories, each obtaining single results on identical test materials when applying the same method. Again it is the difference between two single and independent results, which would be expected in only one case in twenty. This is expressed as:

$$R = 1.96\sqrt{2}\sqrt{\sigma_1^{\,2} + \sigma_2^{\,2}}$$

where r_2 is the standard deviation applicable to all causes of variability other than repeatability, when the results from different laboratories are compared.

Values of r and R are given in Table 7.3 (BS812, 1975), indicating the greatest differences expected between tests on separate samples.

7.6 Student's t test

In order to test the hypothesis that a sample whose mean value is $\bar{x}$ could have come from a population whose mean value is $\bar{X}$ and whose standard deviation is σ, it is necessary to calculate the ratio:

$$\frac{Error\ in\ Mean}{S\tan dard\ Error\ of\ Mean} = \frac{\left|\bar{X} - \bar{x}\right|}{\left(\sigma / \sqrt{n}\right)}$$

which is called *Student's t*.

A more common use of this technique is to test whether the difference between the mean values of two samples drawn from different sources is significant of a real difference between the parent sources.

For this test the 't' value is calculated as follows:

$$t = \frac{\left(\bar{x}_1 - \bar{x}_2\right)}{s\left(\sqrt{\dfrac{1}{n_1} + \dfrac{1}{n_2}}\right)}$$

Where $\bar{x}_1$ and $\bar{x}_2$ are the means of the two samples containing n_1 and n_2 items respectively and s^2 is the pooled estimate of the variance (ie. weighted mean of s_1^2 and s_2^2 – the variances of the two samples).

The value of s is therefore calculated as follows:

$$s = \sqrt{\frac{n_1\, s_1^{\,2} + n_2\, s_2^{\,2}}{n_1 + n_2 - 2}}$$

Statistical tables give 't' values for a probability level and the *number of degrees of freedom*. The number of degrees of freedom for Student's t in this case is ($n_1 + n_2 - 2$). If the magnitude (ignore possible negative sign) of the calculated 't' value is greater than the tabulated 't' value then a significant difference between the two means is probable at that probability level.

7.7 Regression Analysis

Regression analysis is used to decide whether a trend or relationship exists between two or more variables. The general form of the relationship can be written as:

$$y = a + b_1x_1 + b_2x_2 + b_3x_3 + \ldots\ldots\ldots$$

where y is the dependent variable and the x_i are the independent variables.

Generally, estimates or measurements of the independent variables are obtained to predict values of the dependent variable.

Regression analysis derives the 'best fit' relationship to the data using the *least squares* method; ie the values of the constant (a) and the coefficients (b_i) are chosen such that the sum of squares of the differences between the observed and predicted y values are a minimum.

The *goodness of fit* for the relationship is denoted by r^2. The r^2 value ranges between zero (extremely poor fit) and 1 (perfect fit). In other words, if the r^2 value is 0.792, this means that 79.2% of the variation in the dependent variable y is explained by the independent variables.

The degree of uncertainty in using the regression line for estimating y from the independent variables is indicated by the *standard error of the estimate*.

In using regression analysis, particular care should be taken of the following:

- Form of the independent variables. The effect of each independent variable on the dependent variable should be examined separately to determine the functional form that the independent variable should take.

- Significance of the independent variables. Most regression analyses packages will give the *standard error* of each of the coefficients in addition to the magnitude of the coefficients. To test the significance of a coefficient, the value of the coefficient is divided by its standard error. This gives the 't' value of the coefficient; ie. the level of significance. As a general rule, a 't' value in excess of 2.0 (depends on number of observations and required significance level) indicates that the independent variable is a significant explanatory variable.

- Percentage explained. The r^2 value will either remain the same or increase when a further independent variable is used in the analysis, irrespective of the significance of the variable. Therefore an increase in the r^2 value should not be used as the sole justification for adding an independent variable to the regression analysis.

- Extrapolation. The relationship derived using regression analysis is only valid over the ranges of the variables used in the analysis. The relationship should never be applied outside these ranges.

Test Procedure	Minimum Sample Required Fine	Medium	Coarse
Moisture Content	0.05kg	0.35kg	4.00kg
Liquid Limit (Cone /Casagrande))	0.50kg	1.00kg	2.00kg
Liquid Limit (one point Cone)	0.10kg	0.20kg	0.40kg
Plastic Limit	0.05kg	0.10kg	0.20kg
Shrinkage Limit	0.50kg	1.00kg	2.00kg
Linear Shrinkage	0.50kg	0.80kg	1.50kg
Particle Size (BS1377: Sieve)	0.15kg	2.50kg	17.00kg
Particle Size (BS1377: Hydrometer)	0.25kg		
Particle Density (Gas jar)	0.30kg	0.60kg	0.60kg
Compaction (BS Light, 1L mould)		25.0 kg	
Compaction (BS Light, CBR mould)		80.0 kg	
Compaction (BS Heavy, 1L mould)		25.0 kg	
Compaction (BS Heavy, CBR mould)		80.0 kg	
Compaction (Vibration)		80.0 kg	
MCV (Total)	6.00kg	6.00kg	12.00kg
MCV (Rapid)	3.00kg	3.00kg	6.00kg
Chemical Tests (pH Value, Sulphate,Organic Matter,Carbon & Chloride Content)	150g	600g	350g
Point Load Test	10 No. identical samples		
Schmidt Hammer Test (Lab.)	20 No. tests on each sample		
Aggregate Crushing Value (ACV)		2.00kg	
Aggregate Impact Value (AIV)		2.00kg	
Los Angeles Abrasion (LAA)		5.00-10.00kg	
Slake Durability	10 No. lumps @ 40-50g each		
AAV	24 No. Aggregate particles		
PSV	4 x 35-40 No. Aggregate patches		
Methylene Blue Value (MBV)		1.00g	
Thin Section Point Count	Intact sample approx. 50 x 20 x 20 mm		

Table 7.1 Individual Sample Sizes

Statistical Element	General Application	Example of Use in Construction Materials
Mean	Selecting a representative value for a sample population.	Identifying single test values from a series of results to represent a specific characteristics of a material layers within a borrow pit..
Standard Deviation	A means of defining the distribution of test results within a population of results.	Establishing the likely variability of a material's compaction characteristics in the field by looking at the mean and standard variations of the laboratory MDDs and OMCs.
Repeatability (r)	Assessing the expected variability of test results from a single operator.	Assessing the validity of defining specification limits to key parameters. For examples see Table 7.3.
Reproducability (R)	Assessing the expected variability of test results between laboratories.	Assessing the validity of defining specification limits to key parameters. For examples see Table 7.3.
Student's T Test	A means of assessing the difference in the mean of two samples	Establishing whether or not test results from two aggregate sources indicate similar materials or not.
Regression Analysis	Establishing relationships between two populations of results.	Establishing empirical relationships between index tests and material behaviour, for example between point load and UCS or between plasticity and swell potential.

Table 7.2 Examples of the Application of Statistical Concepts in Construction Materials Engineering.

Test	Repeatability (r)	Reproducibility (R)
Relative density		
i) most aggregates	0.02	0.04
ii) low density porous aggregates	up to 0.4	up to 0.08
Water absorption	5% of value recorded	10% of value recorded
Bulk density	10 kg/m^3	20 kg/m^3
Aggregate Impact Value	1.0	2.0
Aggregate Crushing Value	0.8	1.5
Ten Per Cent Fines Value	7 kN	14 kN
Aggregate Abrasion Value	1.5	3.0
Polished Stone Value	4.9	6.0

Table 7.3 Estimates of repeatability and reproducibility of some tests for aggregates

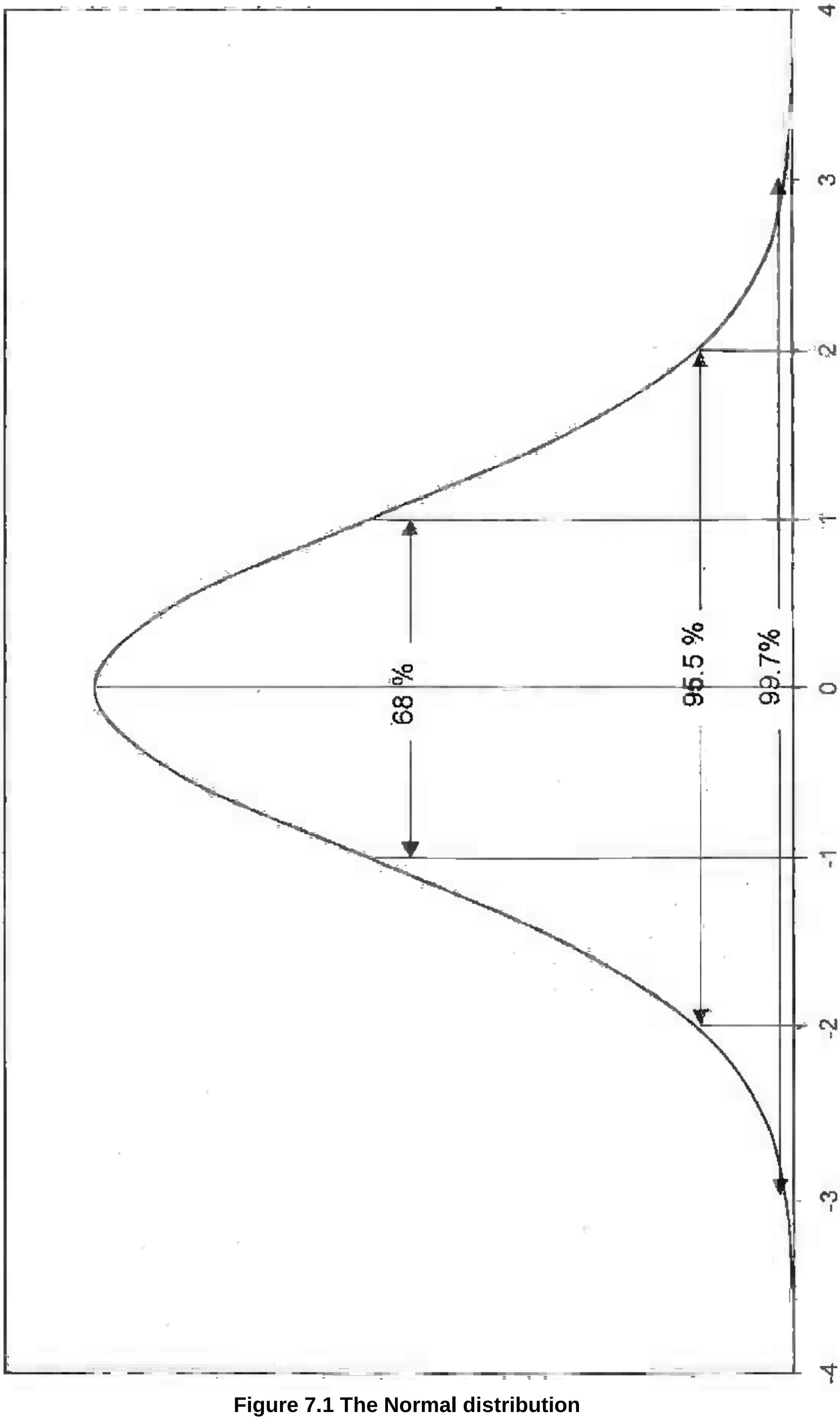


Figure 7.1 The Normal distribution

8 STABILISATION OF NATURAL MATERIALS FOR ROAD BUILDING

8.1 Introduction

This chapter gives guidance on the stabilisation of soils and gravels for road building. Stabilisation is carried out to achieve the following main objectives:

- To increase strength and bearing capacity.
- To control volume change when moisture content changes.
- To increase the resistance to erosion, weathering or traffic usage.
- To reduce the permeability of the stabilised soil.

Many natural materials can be stabilised to make them suitable for road pavements but this process is only economical when the cost of overcoming a deficiency in one material is less than the cost of importing another which is satisfactory without stabilisation.

The simplest method of increasing the strength is to stabilise it mechanically. Compaction is an inexpensive and effective method of providing a dense layer with improved load bearing capacity. In areas where good quality materials are not readily available it may be possible to blend two otherwise unsuitable materials to produce an acceptable product.

Chemical stabilisation involves the incorporation of relatively small percentages of lime, cement or pozzolans. These stabilisers are called hydraulic binders which ‘set’ in the presence of water. They can dramatically increase the strength of unbound materials making them suitable for use in the main load bearing layer of a road pavement, or they can be mixed with soils in small amounts which merely ‘modify’ the physical characteristics of the soil rather than to significantly strengthen it.

8.2 Mechanical Stabilisation

8.2.1 Compaction

Compaction is the simplest method of stabilisation. Well-graded soils can be compacted to high densities at the optimum moisture content that can be determined using standard compaction tests. BS 1924: Part 2: 1990 describes four such tests, each has its merits and the one that is most closely related to field conditions will depend on the type of material that is being compacted.

Figure 8.1 gives typical relationships between moisture content and dry density for a variety of soil types when subjected to the BS 2.5 kg rammer compaction test. As the clay content of the soil is increased the maximum dry density that can be obtained is decreased with a consequent increase in the optimum moisture content. At the optimum moisture content all but one of the soils have air contents of less than 2.5 per cent. The exception is a uniformly-graded sand that has an air void content of about 8 per cent at the optimum moisture content. This is because there are insufficient fines to fill the voids. A denser material could, therefore, be obtained by blending the sand with another suitable soil, a second method of mechanical stabilisation.

8.2.1 Blending.

The blending of materials has two main uses. These are;

(i) Improving the stability of cohesive soils of low strength by adding coarse material.
(ii) Improving the stability of otherwise unstable granular materials by adding a fine material which will provides binding.

The grading of the mixture is important to ensure that after compaction the air void content is low (<5%).
The maximum density grading is given by an equation originally derived by Fuller (1952)

$$P= 100(d/D)^{0.5}$$

where:

- P = Percentage by weight of the total sample passing any given sieve size.
- d = Aperture of that sieve (mm).
- D = Size of the largest particle in the sample (mm).

.

A modification of this equation is used by the Australian Road Research Board (Lay, 1985) such that:

$$P = 100\,(d/D)^n.$$

where:

- P = Percentage of material smaller than d in diameter
- d = Aperture of that sieve (mm).
- D = Size of the largest particle in the sample (mm).
- n = An exponent ranging from 0.35 to 0.50

Mechanical stabilisation of clay soils, by the addition of non-cohesive granular material, needs sufficient granular material to be added to ensure that the granular fragments are in contact and form a particle-particle contact fabric.

Mechanical stabilisation might occasionally be carried out to produce an improved upper fill or capping layer or, more commonly, to produce sub-base and base materials. Care must be taken to ensure that the plasticity of the fines fraction is controlled.

Mechanical stabilisation is usually found to be the most cost-effective process for improving poorly graded materials, however, this cannot always be achieved. It is important to consider the practical limits of this type of processing particularly when the mixing of an appreciable amount of clay is material is involved. Production of a uniform mixture when one material has a high clay content will be very difficult. Even when mixed with a high percentage of non-plastic material it may be found that the clay will still play the dominant part in determining material properties. The strength of a blended material must be determined by testing samples that are representative of the field mixed product and not on well mixed laboratory samples.

Mechanical stabilisation has drawbacks particularly in those countries that are subject to heavy rainfall. If a design is completed on the basis of laboratory mixing it may be found that, after full-scale mixing trials, lime modification is still needed to allow adequate processing of a clayey material.

Although a mechanically stable material is highly desirable it cannot always be achieved, even when it can it may be necessary to add a stabilising agent to bring about a further improvement in the properties of a material.

8.3 Bitumen Stabilisation

Bitumen has strong adhesive and waterproofing properties. It can be made in suitable forms for stabilisation either as a ‘cut-back’ when blended with kerosene and/or diesel fuel; as an emulsion or in a foaming technique where specialised equipment adds water to the hot bitumen at the point of mixing. These binders are most satisfactory when used in hot dry climates, which ensures rapid drying out of solvents or water. However, the use of emulsions is likely to be restricted to stabilising more granular materials

8.4 Chemical Stabilisation

8.4.1 General

Stabilisation of a material by the addition of a chemical additive can enhance the properties of road materials and give pavement layers the following attributes:

- A substantial proportion of the strength is retained when they become saturated with water.
- Surface deflections are reduced.
- Resistance to erosion is increased.
- Materials in the supporting layer cannot contaminate the stabilised layer.
- The elastic moduli of granular layers constructed above stabilised layers are increased.
- Lime-stabilisation can be used to produce a capping layer or working platform with wet or unsuitable in situ materials.
- Characteristics such as plasticity, compressibility and permeability can be reduced.

Additives can also be used to waterproof a material rather than to directly increase its strength.

8.4.2 Portland Cement as a Stabiliser

Portland cement is defined in BS 12: (1978) as, "a product consisting mostly of calcium silicate, obtained by heating to partial fusion a predetermined and homogeneous mixture of materials containing principally lime (CaO) and silica (SiO_2) with a small proportion of alumina (Al_2O_3,) and iron oxide (Fe_2O_3)". Calcareous materials, typically limestone, provide the CaO and argillaceous materials, such as clay, shale, provide the SiO_2, Al_2O_3 and Fe_2O_3. Cement is produced in many countries and should be made to recognised standards (e.g. BS 12: 1978 ; ASTM C595-77).

Cement comprises calcium silicates and aluminates, and calcium oxide. In the presence of water, these form hydrated compounds which harden over time to produce a strong cemented matrix in which the particles of soil or granular material are embedded. Initially, this chemical reaction is quite rapid, but the rate decreases with time. The addition of cement to a material, in the presence of moisture, produces hydrated calcium aluminate and silicate gels which crystallise and bond the material particles together

When mixed with cement a soil may be merely embedded in in the hydrated cement or an additional bonding reaction may occur if the soil contains pozzolanic material which reacts with calcium hydroxide, produced during hydration of the cement. A chemical reaction also takes place between the material and lime that is released as the cement hydrates, leading to an increase in strength.

Cement can be used to stabilise most soils. Exceptions are those with a high organic content, which retards the hydration process, and those with a clay content outside the normal specification, where it is difficult to mix the soil/cement mixture evenly. Addition of cement to base materials results in a reduction in plasticity and swell, and an increase in strength and bearing capacity. CBR values well in excess of the minimum requirement for unstabilised gravels (usually 80 per cent, soaked at the required field density) normally result.

8.4.3 Lime as a Stabiliser

Lime may be in one of the following forms;

- quicklime: calcium oxide (CaO),
- slaked or hydrated lime: calcium hydroxide ($Ca(OH)_2$)
- carbonate of lime: calcium carbonate ($CaCO_3$).

Calcium carbonate has no cementing properties and only quicklime and hydrated lime are used as stabilisers in road construction. They are produced by heating calcium carbonate to more than about 500 °C to drive off carbon dioxide. The reactions are represented by the following equations:

- $CaCO_3$ + heat = CaO + CO_2 (reversible in the presence of carbon dioxide)
- CaO + H_2O = $Ca(OH)_2$ + heat
- $Ca(OH)_2 + CO_2 = CaCO_3 + H_2O$

The most common form of commercial lime used in stabilisation is hydrated (high calcium) lime, $Ca(OH)_2$, but limes can also be produced from dolomitic limestone where magnesium replaces some of the calcium. Monohydrated dolomitic lime, $Ca\ (OH)_2.MgO$, calcitic quick lime, CaO, and dolomitic quicklime, CaO.MgO are also used.

For hydrated lime the majority of the free lime (defined as the calcium oxide and calcium hydroxide that is not combined with other constituents) should be present as calcium hydroxide. British Standard 890 requires a minimum free lime and magnesia content, (CaO + MgO), of 65 per cent.

The reaction between lime and soil causes calcium ions to replace sodium ions until the soil becomes saturated with calcium ions. The pH increases to a desirable value in excess of 12 after which stage soil strengthening can take place. The lime required to satisfy these exchange reactions is known as the initial consumption of lime (ICL), (British Standard 1924 (1990)).

Quicklime has a much higher bulk density than hydrated lime and it can be produced in various aggregate sizes. It is less dusty than hydrated lime but the dust is much more dangerous. For quicklime, British Standard 890 requires a minimum lime and magnesia content, (CaO + MgO), of 85 per cent. Quicklime is an excellent stabiliser if the material is clayey or very wet. When it comes into contact with the soil the quicklime absorbs a large amount of water as it hydrates and the soil dries. This process is exothermic and the heat produced acts as a further drying agent.

When lime is added to a plastic material, it first flocculates the clay and substantially reduces the plasticity. Addition of 2 per cent of lime can increases the plastic limit of a wet and sticky soil changing it from one which is impossible to compact and impassable to traffic, to one which is workable. The removal of water and the increase in plastic limit cause a substantial and rapid increase in the strength and traffickability of the wet material. The effect of lime on the liquid limit is much less marked but the overall effect is usually a considerable reduction in the plasticity index

Lime (calcium oxide, or quicklime) can be used to modify or lower the plasticity of the available materials, or to dry out materials that are wetter than the optimum moisture condition at construction. Some cementation and strengthening of the matrix also occurs. The chemical is normally used in its hydrated or slaked form (calcium hydroxide), and is most often applied where soils have a high plasticity and where mixing in cement is difficult. Sufficient excess lime is added to the material to ensure that the stabilised mixture retains a high pH of at least 12.4. This high pH environment is essential to ensure the long term stability of the hydration products.

The reduction of plasticity is time dependent during the initial weeks, and has the effect of increasing the optimum moisture content and decreasing the maximum dry density in compaction. The compaction characteristics are therefore constantly changing with time and excessive delays in compaction cause reductions in density and consequential reductions in strength and durability. The workability of the soil also improves as the soil becomes more friable. If the amount of lime added exceeds the ICL the stabilised material will generally be non-plastic or only slightly plastic.

For adequate stabilisation with lime, sufficient lime needs to be added to give rise to an excess after the replacement reactions have been completed i.e. the Initial Consumption of Lime (ICL) of the soil should be satisfied and an excess provided. The ICL test developed by Eades and Grim (1966) can be used to give a rapid indication of the minimum amount of lime that is required to be added to a material to achieve a significant change in its properties. The principle of the test is based on the fact that a saturated solution of lime (calcium hydroxide) in distilled water, that is completely free of carbon dioxide, has a pH value of 12.4 at $25^{o}C$. This pH is required to maintain reaction between the lime and any reactive components in the material to be stabilized. Samples of the material are therefore mixed with water and different proportions of the lime being used. The minimum amount of lime needed to give a pH of 12.40 is expressed as the ICL of the material.

The original test was improved by Clauss and Loudon (1971) and details of the test are included in the latest edition of BS 1924 (BS 1924: Part 2: 1990b). The test can be completed in one hour and is thus a rapid means of establishing the minimum amount of lime required for stabilisation. However, it does not dispense with the need to carry out strength determinations as it does not establish whether the soil will react with lime to produce a substantial strength increase. Research has also suggested that lime percentage obtained from the test does not produce the maximum cured compressive strengths for tropical and sub-tropical soils (TRB 1987).

Typically 3 to 5 percent stabiliser is necessary to gain a significant increase in the compressive and tensile strengths. The gain in strength with lime stabilisation is slower than that for cement and a much longer time is therefore available for mixing and compaction. Lime has a much lower specific gravity than cement so, for a given percentage mass, a higher volume is available and it is therefore easier to achieve uniform mixing.

The production of cementitious compounds can continue for ten years or more but the strength developed will be influenced by the materials and the environment. The elastic modulus behaves similarly to the strength and continues to increase for a number of years. Between one month and two to three years after compaction there can be a four-fold increase in the elastic modulus.

In many parts of the world, lime has been produced on a small scale for many hundreds of years to make mortars and lime washes for buildings. Different types of kilns have been used and most appear to be relatively effective. Trials have been carried out by TRL in Ghana to determine the output possible from these small kilns and to assess the suitability of the lime for stabilisation, Ellis (1974). Small batch kilns have been used to produce lime for stabilised layers on major road projects.

8.4.4 Chemical Stabilisers

Chemical stabilisers exist which mostly take the form of strongly acidic, ionic, sulphonated, or oil-based products. A cementitious reaction does not usually occur, but due to ionic exchange, the adsorbed water can be reduced leading to better compaction and increased strength. The stabilised material is also made waterproof. The material must have an appropriate clay content for the stabiliser to have a beneficial effect. When correctly utilised, these products can be cost effective, Paige-Green (1998).

Tests with a stabilising agent should always be compared with the results obtained from the same soil tested under identical conditions without the addition of a stabiliser and also with those of lime and cement, because a better or equivalent effect could be obtained with them at a lower cost.

8.4.5 Secondary Stabilising Agents

This group includes those materials which in themselves do not produce a significant stabilising effect but which have to be used in association with lime or cement. They are often blended before use in which case the blended mixture assumes the role of a primary agent.

Blastfurnace slag, is produced during the smelting of iron. It is formed by the combination of the siliceous constituents of the iron ore with the limestone flux. Although it is not cementitious it contains the same elements as Portland cement and possesses latent hydraulic properties that can be developed by adding an activator such as lime or another alkaline material.

Ground granulated blastfurnace slag (GGBFS) is used in the manufacture of ‘blastfurnace cement’ (e.g. BS 6699:1986 and ASTM C595-76) which has similar properties to those of Portland cement. The French ‘grave-laitier’ process uses between 15 and 20 per cent of GGBFS with about 1 percent activator (usually hydrated lime) to stabilise sands and/or gravels.

A pozzolana is defined in ASTM 1976 as “a siliceous, or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties”.

The reaction between clay and lime is a reaction of this type. Natural pozzolanas are mainly of volcanic origin. Artificial pozzolanas are mainly obtained by the heat treatment of clays, shales and certain siliceous rocks.

One of the primary sources of pozzolan in Europe is the pulverised fuel ash (PFA) collected from the boilers of coal-fired electricity generating stations. PFA is usually mixed with lime in the proportions of 1 of lime to 3 or 4 of PFA but ratios of 1 to 2 up to 1 to 10 are used. The proportion depends on the reactivity of the particular fly ash that varies substantially from source to source. Lime and fly ash treated layers have a similar performance to cement treated layers constructed from the same aggregate material. The final mixtures should be chosen after a series of laboratory tests carried out after 21 days of moist cure and 7 days of soaking to determine the optimum ratio of lime to fly ash and the optimum lime content (expressed as a percentage of dry soil).

Ash from burnt plants, especially rice husks, rice straw and bagasse (from sugar cane) can have a high enough silica content to provide an excellent pozzolana (Mehta 1979, Spence 1980), (Cook and Suwanvitaya (1982). Lime and rice husk ash mixtures gain strength quickly during the curing but little additional strength is obtained after 28 days of moist curing. The long-term strength depends on the stability of the calcium silicate hydrates. Under certain conditions, lime leaching can occur and eventually the strength will be reduced, but the presence of excess lime (free lime) can stabilise this calcium silicate hydrate. Mixtures of lime and rice husk ash in the proportions 2:3 are the most stable and have the highest strength but the durability may be improved by increasing the lime content to give a 1:1 mixture.

Calcium hydroxide is one of the hydration reaction products of Portland cement. It has no strengthening effect and can cause instability. The addition of a pozzolana to ordinary Portland cement can be beneficial by reacting with the calcium hydroxide to produce further cementitious material. Many pozzolanas can be used in the manufacture of Portland-pozzolan cements (ASTM C595-76). Specifications exist for both the properties of pulverised fuel ash used in cement (BS6588: 1985a), in concrete (BS 3892: 1982 and 1985) and for Portland pulverised fuel as cement (ASTM C595-76 and BS 6588: 1985b)

8.5 Application of Chemical Stabilisation

8.5.1 Strength of Stabilised Materials

There are three main types of cement-stabilised materials, ‘Soil-Cement’, Cement Bound Granular Material, and Concrete.

The addition of more than 15% cement usually produces a conventional concrete. Granular materials can be improved by the addition of generally less that 10 per cent of Portland cement. Cement-bound natural gravels and crushed rocks will have high moduli in the range of 2,000 to 20,000 MPa, compared to 200-400 MPa for the unbound material.

Lime (typically 1-3 percent) or small quantities of cement are often used to modify soils simply to improve their engineering characteristics. Australian (NAASRA 1986), New Zealand (Dunlop 1977) and South African (NITRR 1986) specifications all make a distinction between a stabilized soil ‘modified’ for subgrade improvement and ‘cemented’ for use as sub-base or roadbase where higher compressive strengths are required.

A 7-day unconfined compressive strength of 0.8 MPa and an indirect tensile strength of 0.08 MPa have been suggested to represent the boundary between modification and cementation (NAASRA 1986).

Table 8.1, after Ingles and Metealf (1972), summarises the distinction between lime modification and cementation and the principal uses of both types of treatment.

8.5.2 Soil Cement

Soil cement usually contains less than 5% cement. (Lay, 1986). This can either be mixed in situ (usually up to 300mm layer at a time) or mixed using plant such as a pulverisor. The technique involves breaking up the soil, adding and mixing in the cement, then adding water and compacting in the usual way. Croney (1998) recommends that a minimum strength should be 2.5 MPa or, if this material is used to replace sub-base then the strength requirement should be increased to 4 MPa (7 day cube crushing strength).

8.5.3 Cement Bound Granular Material (CBM)

This can be regarded as a stronger form of soil-cement but using a granular aggregate rather than a soil. The process works best if the natural granular material has a limited fines content. This is almost always mixed in plant and the strength requirement is 5-7 MPa (7 day cube crushing strength) (Croney, 1998).

8.5.4 Lean Concrete

This material has a higher cement content than CBM and hence looks and behaves more like a concrete than a CBM. It is made from batched coarse and fine aggregate, but natural washed aggregate can also be used. The UK specification for this material gives a normal strength of 6-10 MPa or a higher strength of 10-15 MPa (7 day cube crushing strength).

8.5.5 Strength Requirements for Pavement Layers

The minimum acceptable strength of a stabilised material depends on its position in the pavement structure and the level of traffic. It must be sufficiently strong to resist traffic stresses but upper limits of strength are usually set to minimise the risk of reflection cracking. The strengths for three types of stabilised layer given in Overseas Road Note 31 (TRL, 1993) are shown in Table 8.2.

The recommendations take account of the way in which asphalt surfacings become very susceptible to cracking through environmental deterioration (ref , ref), and reflection cracking from any shrinkage cracks which may form in the stabilised layer. This means that if a bituminous running surface is required, then this should be a thin seal if the surfacing is placed directly onto a stabilised roadbase. An asphalt layer should only be used if a granular inter-layer is placed between the surfacing and roadbase to prevent reflection cracks in the asphalt.

8.5.6 Two-Stage Stabilisation

Although lime used in equivalent amounts generally produces lower strengths than does cement it is very effective in breaking down heavy soils and clods. A first treatment of the soil with lime makes it much easier to achieve stabilisation with cement.

Early work in the UK using two-stage stabilization with lime and cement was disappointing (Metcalf (1959), Cruchley (1956), Sherwood and Covell (1959) and Lea (1970)) However, more recently the alkali content of British cements has increased and this may well explain why there have been several recent examples of the successful use of lime and cement. Tesoriere et al (1980) found that a heavy clay soil, of high pozzolanic activity, attained higher strengths when stabilized with lime and cement than it did with equivalent amounts of lime or cement used alone.

In tropical climates lime and cement-stabilized soils often develop strengths at a similar rate and attain similar ultimate strengths. However, economic reasons and difficulties in mixing in the stabiliser are likely to determine whether or not two stage stabilisation will be appropriate.

8.6 Testing for Chemical Stabilisation

8.6.1 ICL and ICC Testing

It is recommended that, during any assessment of the suitability of materials for stabilised base or sub-base, the first test to be carried out should be the Initial Consumption of Lime (ICL) or the Initial Consumption of Cement (ICC). The inclusion of the gravel ICL/ICC test into specification limits should alleviate most of the durability problems presently experienced, for example with basic igneous materials.

Should the wet/dry brushing test and/or the residual UCS test be required, they should be carried out on samples with a stabiliser content one per cent higher than the ICL/ICC value. In some cases, the suggested stabiliser content from the gravel ICL/ICC value may exceed that which is economically viable. Using stabiliser contents in excess of about three per cent is often uneconomic. However, there is concern in South Africa that reducing the stabiliser content is more likely to increase the risk of durability problems resulting in costly premature failures in service. This is likely to be true for the more difficult materials like the calcretes and basic igneous gravels. However, there is much evidence in the region that shows that many materials can be successfully modified with small (1-2 per cent) quantities of stabiliser.

A significant amount of work on the ICL test and its interpretation has been carried out by the KwaZulu Natal Roads Department. They now recommend that the one-hour ICL test should be supplemented by a delayed, 28 day ICL test on the same material, sealed after the original one-hour test has been carried out. Calculation of the ICL value is then made from the breaking point of the graphs of the two test results as follows:

$$ICL\ (\%) = a + (b - a) / 3$$

where:

a = one-hour ICL breakpoint
b = 28-day ICL breakpoint

While this method may give a more accurate ICL value, it is time consuming to obtain a result. For the purposes of evaluating stabiliser contents for other durability tests, the gravel ICL value is considered suitable.

8.6.2 Strength Testing

Cement or lime stabilised materials are usually approved on the basis of strength tests carried out on the materials after the stabiliser has had sufficient time to cure. The most commonly used methods are the Unconfined Compressive Strength (UCS) Test, for cement stabilised materials, and the California Bearing Ratio (CBR) Test for lime stabilised or modified materials.

Methods of testing the strength of cement and lime stabilised materials usually depends upon the type of soil being stabilised and its intended position in the road structure. Three classes of stabilized material are defined in BS 1924 (Part 1:1990) together with recommended sizes for test samples for testing purposes, the details of which are given in Table 8.3.

Both BS and ASTM standards specify cylindrical specimens with a height/ diameter ratio of 2:1 and this is generally the same with other standards. The notable exception is a cylindrical specimen which has the same dimensions as a CBR mould i.e. 152 mm high and 127 mm diameter.

If cubical moulds are not available Road Note 31 (1993), in common with the South African standard (NITRR 1986) recommends the use of CBR moulds. Other cylindrical moulds of suitable size may also be used. ASTM Standard Method D 1632-63 for soil cement recommends cylindrical specimens 142 mm high and 71 mm diameter. It does not give details of any larger sizes of mould but instead recommends the removal of any particles in excess of 19 mm in diameter and their replacement,. from the original sample, with an equal weight of material having particles less than 19 mm diameter but greater than 4.75 mm.

Material strength determined with different sizes of sample will vary and for the materials normally used in road building the correction factors given in Table 8.4 should be used to calculate the approximate equivalent strength of a 150 mm cube.

Unconfined compressive strength
The UCS Test is carried out on cylindrical or cubical samples by mixing the material at the desired moisture and stabiliser content, and compacting the material into a mould to either a pre-determined density or a given compactive effort. As the increase of strength of stabilised materials occurs over relatively long periods of time, samples are cured for 7, 14 or 28 days prior to testing. A seven-day curing period, although arbitrary, is often chosen as a convenient reference for cement treated materials, whilst a longer twenty-eight day period is chosen for lime treated materials to take into account their slower strength gain.

California bearing ratio (CBR)
The CBR test is carried out on samples compacted into cylindrical moulds. The reference value for the results is the standard value expected for a well-graded and non-cohesive crushed stone, which has a CBR of 100. Stabilised and modified soils can have CBR values well in excess of this value and at these strength levels the UCS test is preferred.

Whilst the CBR test is allowed for testing sub-base materials it is preferrable to use unconfined compression tests (UCS) wherever possible. This is because the nature of the stabilised material is very different to the material of definition, i.e. a particular size of crushed rock aggregate, and also because different relationships between CBR and UCS have been developed by a number of authorities, as is shown in Table 8.5.

8.6.3 Preparation of specimens.

The optimum moisture content and the maximum dry density for mixtures of soil plus stabiliser are determined according to British Standard 1924 (1990) for additions of 2,4,6 and 8 per cent of cement. These specimens should be compacted as soon as the mixing is completed. Delays of the order of two hours occur in practice and changes taking place within the mixed material result in changes in their compaction characteristics. To determine the sensitivity of the stabilised materials to delays in compaction, another set of tests must be conducted where the samples are compaction after two hours have elapsed since the completion of mixing.

Samples for the strength tests should also be mixed and left for two hours before being compacted into 150 mm cubes at 97 per cent of the maximum dry density obtained, after a similar two hour delay, in the British Standard (heavy) Compaction Test, 4.5 kg rammer. These samples are then moist cured for 7 days and soaked for 7 days in accordance with BS 1924.

Two methods of moist curing are described in the Standard. The preferred method is to seal the specimens in wax but if this is not possible they must be wrapped in cling film and sealed in plastic bags. The specimens should be maintained at 25^{o}C during the whole curing and soaking period. When the soaking phase is completed, the samples are crushed and an estimate made of the cement content needed to achieve the target strength.

When the plasticity of the soil makes it difficult to pulverise and mix intimately with the cement, the workability can be improved by first pre-treating the soil with 2 to 3 per cent of lime, lightly compacting the mixture, and leaving it to stand for 24 hours. The material is then repulverised and stabilised with cement. If this method is used, the laboratory design procedure is modified to include the pre-treatment phase before testing as described above.

8.6.4 Durability Tests

Neither the standard UCS or CBR tests used by themselves are considered to reflect the stability of a chemically stabilised materials in terms of durability against wetting and drying or freeze-thaw (Sherwood, 1993). A number of countries therefore employ additional tests to assess durability, namely:

- Wetting and drying test
- Freeze-thaw test
- Wet/dry brushing test
- Residual UCS test

Wetting and drying. The wetting and drying test (ASTM D559) determines the weight losses, moisture changes and volume changes resulting from repeated wetting and drying of hardened stabilised soil specimens.

Freeze-thaw. The freeze-thaw test (ASTM D560) follows a similar procedure to the previous test except that wetting and drying is replaced by cycles of freezing and thawing. This test is used even in regions where freezing never occurs as it more rapidly exposes weaknesses in the stabilised soil that could occur under natural temperatures above freezing.

Wet/dry brushing. The existing wet/dry brushing test given in method A19 of TMH 1 (NITRR, 1986) is very operator dependent (Sampson 1986). This has led to the development of the mechanical wet/dry brushing test described by Sampson (1988). A limited number of performance-related results were available for method A19 of TMH 1 from which tentative specification limits were developed. A correlation is shown by Sampson (1990) for converting the limits based on method A19 of TMH 1 to the limits shown above for the more reproducible mechanical brush test.

Residual UCS. The original residual UCS test described by De Wet and Taute (1985), included a UCS after 12 cycles of wetting and drying and a UCS after vacuum carbonation. However, the correlation given in Figure 7 of Sampson (1990) showed that the carbonated strength, with one exception, was always lower than the wet/dry UCS after 12 cycles. This was not surprising, as some curing and strength increase could occur during wetting and drying cycles. The results showed that, providing the requirements of the vacuum carbonated seven-day RUCS is met, this will also satisfy a residual wet/dry strength requirement. Thus, it was recommended that only the residual, vacuum-carbonated, seven-day UCS needed to be satisfied

8.7 Selection of Chemical Stabilisation Method

8.7.1 Selection of Stabilisation Type

Figures 8.2 and 8.3 summarise the general approaches to firstly assessing material suitability for stabilisation and secondly the selection of stabiliser type.

The selection of the stabiliser is based on the plasticity and particle size distribution of the material to be treated. The appropriate stabiliser can be selected according to the criteria shown in Table 8.6 adapted from NAASRA (1986).

Some control over the grading can be achieved by limiting the coefficient of uniformity to a minimum value of 5 (cf Chapter 6). If the coefficient of uniformity lies below this value the cost of stabilisation will be high and the maintenance of cracks in the finished road could be expensive. Except for materials containing amorphous silica e.g. some sandstones and chert, material with low plasticity is usually best treated with cement. However, reactive silica in the form of pozzolans can be added to soils with low plasticity to make them suitable for stabilisation with lime. If the plasticity of the soil is high there are usually sufficient reactive clay minerals which can be readily stabilised with lime. Cement is more difficult to mix intimately with plastic materials but this problem can be alleviated by pre-treating the soil with approximately 2 per cent of lime to make it more workable.

If possible the quality of the material to be stabilised should meet the minimum standards set out in Table 8.7. Stabilised layers constructed from these materials are more like to perform satisfactorily even if they are affected by carbonation during their lifetime Materials which do not comply with Table 8.7 can sometimes be stabilised but more additive will be required and the cost and the risk from cracking and carbonation will increase.

Some aspects of construction must also be considered in selecting stabiliser. It is not always possible to divert traffic during construction and the work must then be carried out in half-widths. The rate of gain of strength in the pavement layer must be rapid so that traffic can be routed over the completed pavement as soon as possible. Under these circumstances, cement stabilisation, with a curing period of seven days, is likely to be more suitable than lime stabilisation which requires a much longer curing period.

Certain types of organic compounds in soils can affect the hydration of cement and inhibit the gain in strength. It is recommended that the effects of organic matter are assessed by strength tests as outlined below.

8.7.2 Selection of cement content.

The cement content determines whether the characteristics of the mixture are dominated by the properties of the original soil (grain interlock) or by the hydration products (cemented matrix). As the proportion of cement in the mixture increases, so the strength increases. Strength also increases with time. During the first one or two days after construction this increase is rapid. Thereafter, the rate slows down although strength gain continues provided the layer is well cured. The choice of cement content depends on the strength required, the durability of the mixture, and the soundness of the aggregate.

The minimum cement content, expressed as a percentage of the dry weight of soil, should exceed the quantity consumed in the initial ion exchange reactions. Until research into the initial consumption of cement (ICC) is completed it is recommended that the percentage of cement added should be equal to or greater than the ICL. If there is any possibility that the material to be stabilised is unsound e.g. weathered basic igneous materials, then the Gravel ICL Test (NITRR, 1984) is preferred.

Additional stabiliser is normally incorporated to take account of the variability in mixing which occurs on site. If good control is exercised over the construction operations, an extra one per cent of stabiliser is satisfactory for this purpose.

The durability of the stabilised mixture which satisfies the strength requirements for the particular layer should also be assessed. Mixtures produced from sound materials complying with the minimum requirements can be assumed to be durable if they achieve the design strength. Mixtures produced from other materials should be checked using the wet-dry brushing test (ASTM (1987)) which gives a good indication of the likelihood that a stabilised material will retain adequate strength during its service life in a pavement (Paige-Green et al (1990)).

8.7.3 Selection of lime content.

The procedure for selecting the lime content follows the steps used for selecting cement content and should, therefore, be carried out in accordance with British Standard 1924 (1990). The curing period for lime-stabilised materials is 21 days of moist cure followed by 7 days of soaking.

In tropical and sub-tropical countries the temperature should be maintained at 25^{o}C. Accelerated curing at higher temperatures is not recommended because the correlation with normal curing at temperatures near to the ambient temperature can differ from soil to soil. At high temperatures the reaction products formed by lime and the reactive silica in the soil can be completely different from those formed at ambient temperatures.

8.8 Construction Issues

8.8.1 Carbonation

The process of carbonation is most easily explained with reference to lime, which is produced by heating mineral deposits containing calcium carbonate. Calcium is a relatively active element and calcium oxide (CaO), commonly known as quicklime, is quite dangerous. Therefore, it is often slaked with water to form its hydrated hydroxide. As with the many chemical reactions, the formation process is reversible, depending on the environmental conditions. Both the oxide and hydroxide forms can react with carbon dioxide to form calcium carbonate.

It is therefore important to minimise exposure to CO_2 during the manufacture and storage operations, to mix and compact materials quickly, and to cure the stabilised material properly. In fact, there is a view that premature deterioration in some roads containing stabilised materials may be due as much to the quality of the lime used as to other processes following construction.

The reactions with cement are more complex, but the cemented products, represented as hydrated calcium silicates (CSH) and hydrated calcium aluminates (CAH), are also susceptible to carbonation.

Both CSH and CAH require a minimum pH of about 12.4 to ensure that the reactions progress and that the hydration products remain stable. This high pH environment is possible because of the presence of $Ca(OH)_2$ in the soil water. Above pH 12.4, there is a dramatic increase in the solubility of the silica and alumina in the soil, and reactions proceed to form calcium silicates and aluminates. These are the cementitious materials which bind the soil together. Sherwood (1968) showed that materials treated with 10 per cent cement would not gain significant strengths (greater than 1MPa at 7 days), unless the reaction pH was greater than 12.4. With higher pH, greater long term strengths were possible.

Quicklime (CaO) and hydrated lime ($Ca(OH)_2$) will both react with the carbon dioxide in the air and revert to calcium carbonate on long-term exposure. This is the main problem of carbonation; clearly any carbonation that occurs before the lime has reacted with the soil reduces the amount of lime available for reaction

If cement or lime-stabilised materials are exposed to air, the hydration products may react with carbon dioxide thereby reducing the strength of the material by an average of 40 per cent of the unconfined compressive strength (Paige-Green et al (1990)). This reaction is associated with a decrease in the pH of the material from more than 12 to about 8.3. The presence and depth of carbonation can be detected by testing the pH of the stabilised layer with phenolphthalein indicator and checking for the presence of carbonates with hydrochloric acid (Netterberg (1984)). A reasonable indication of whether the material being stabilised will be subject to serious carbonation can be obtained from the wet/dry test for durability (Paige-Green et al (1990)).

In adverse circumstances carbon dioxide may also react with the hydration products of lime- and cement-stabilized soils causing loss of strength and even increase in plasticity. This has not been considered to be a serious problem but failures of stabilized soils in Southern Africa have been shown to be due to carbonation of the stabilizer (Bagonza et al 1987; Paige-Green et al 1990). This work has shown that, due presumably to the decomposition of organic matter in the soil, the carbon dioxide content of the air-voids may be far in excess of that normally found in air. The carbon dioxide content of the air is about 0.03 per cent but Paige-Green et al found values as high as 15 per cent.

The main modes of distress caused by carbonation of stabilized layers are loosening of unsurfaced or primed layers and loss of bond between the stabilized layer and any layer above. Paige-Green et al point out that the problem may be more extensive than has hitherto been realised. They suggest that some failures in the past that were attributed to other factors, such as inadequate compaction, low stabilizer content etc., may in fact have been caused by carbonation.

An in-situ test for detecting whether or not carbonation of a stabilized layer has occurred is included in BS 1924: Part 2: 1990. The stabilized layer is first sprayed with phenolphthalein indicator which gives a red coloration if the pH of the layer is above 10.0, thus indicating that some free lime must still be present. If the pH is below 10, indicated by the absence of colour with phenolphthalein, a further test is made by spraying the surface with dilute hydrochloric acid which causes effervescence if carbonates are present.

However, effervescence of the stabilized material when treated with hydrochloric acid is not in itself a diagnostic test as carbonates may be present naturally in the unstabilized material.

Carbonation seems to be more of a problem with marginal quality materials and durability tests such as those recommended by the Portland Cement Association (1979) are more appropriate than strength tests in detecting the propensity of carbonation to occur. Carbonation is most critical for materials which depend on a high pH for stability. Weathered or partly weathered basic igneous materials can degenerate in a pavement layer if they contain smectite minerals. These materials will only remain stable if the pH is maintained at a level in excess of 11.5. This can be achieved by cement- or by lime-stabilization provided enough stabilizer has been added to exceed the initial consumption of lime (see Section 7.2) and the stabilizer does not subsequently become carbonated. If carbonation does occur the drop in pH to 8.3 (i.e. the pH value of $CACO_3$) may be followed by the rapid disintegration of the unsound aggregate.

Good curing practices are the best means of preventing carbonation in a stabilized layer. In addition if the stabilized material is durable the problems caused by carbonation are minimised. Although the strength losses determined on the bases in this study did not lead to premature pavement distress, it is apparent that carbonation can cause a loss in strength in some circumstances and it is sensible to take precautions against it during construction. These include:

- Keep the material continuously moist during curing and avoid wet/dry cycles. Wet sand or polythene sheeting should be used if possible;
- Keep out carbon dioxide by minimising the exposure of the layer to the atmosphere i.e. seal as soon as possible, and encapsulate in potentially severe cases;
- Improve construction practice and control, e.g. compact sooner with appropriate equipment to get higher density and lower permeability;
- Compact to a low air void content (e.g. <5%) not to a prescribed percentage of a compaction test, and
- Improve curing procedures and operations.
- Testing the density-strength-delay time relationship in the laboratory and revising the construction procedures accordingly
- Compacting as early as possible after mixing to increase the density and to reduce the permeability
- Compacting at the correct optimum moisture content for the soil-stabiliser mix
- Avoiding over-compaction of the roadbase layer which leads to micro-cracking on the surface and loose material
- Brushing the surface to remove loose material and probably carbonated material which may affect prime penetration and adherence of the seal
- Sealing with a surfacing as soon as possible after compaction to exclude carbon dioxide
- Reducing the possibility of reflection cracking by proper curing during construction
- Where cracking occurs, crack seal as soon as possible, if lower pavement layers are moisture sensitive

8.8.2 Construction Methodologies

The construction of stabilised layers follows the same procedure whether the stabilising agent is cement, lime or mixtures of lime-pozzolan. After the surface of the layer has been shaped, the stabiliser is spread and then mixed through the layer. Sufficient water is added to meet the compaction requirements and the material mixed again. The layer must be compacted as soon as possible, trimmed, re-rolled and then cured.

The effect of each operation on the design and performance of the pavement are discussed below.

<u>Spreading the stabiliser</u>. The stabiliser can be spread manually by "spotting" the bags at predetermined intervals, breaking the bags and then raking the stabiliser across the surface as uniformly as possible. Lime has a much lower bulk density than cement and it is possible to achieve a more uniform distribution

with lime when stabilisers are spread manually. Alternatively, mechanical spreaders can be used to gauge the required amount of stabiliser onto the surface.

Mixing. Robust mixing equipment of suitable power for the pavement layer being processed is best able to pulverise the soil and blend it with the stabiliser and water. The most efficient of these machines carry out the operation in one pass, enabling the layer to be compacted quickly and minimising the loss of density and strength caused by any delay in compaction. Multi-pass machines are satisfactory provided the length of pavement being processed is not excessive and each section of pavement can be processed within an acceptable time. Graders have been used to mix stabilised materials but they are inefficient for pulverising materials and a number of passes are needed before the quality of mixing is acceptable. Graders should only be considered for processing lime-stabilised layers.

Compaction. A stabilised layer must be compacted as soon possible after mixing has been completed in order that the full strength potential can be realised and the density can be achieved without overstressing the material. If the layer is overstressed, shear planes will be formed near the top of the layer and premature failure along this plane is likely, particularly when the layer is only covered by a surface dressing.

Care should be taken to reduce the density gradient in the layer because permeable material in the lower part of the layer makes it more susceptible to carbonation from below. If necessary, two-layer construction can be employed to ensure effective compaction throughout the stabilised material.

The compaction operation should be completed within two hours and the length of road which is processed at any time should be adjusted to allow this to be achieved.

Curing. Proper curing is very important for three reasons:

- It ensures that sufficient moisture is retained in the layer so that the stabiliser can continue to hydrate.
- It reduces shrinkage.
- It reduces the risk of carbonation from the top of the layer.

In a hot and dry climate the need for good curing is very important but the prevention of moisture loss is difficult. If the surface is sprayed constantly and kept damp day and night, the moisture content in the main portion of the layer will remain stable but the operation is likely to leach stabiliser from the top portion of the layer. If the spraying operation is intermittent and the surface dries from time to time (a common occurrence when this methods is used), the curing will be completely ineffective.

Spraying can be a much more efficient curing system if a layer of sand, from 30 to 40mm thick, is first spread on top of the stabilised layer. If this is done the number of spraying cycles per day can be reduced and there is a considerable saving in the amount of water used. After seven days the sand should be brushed off and the surface primed with a low viscosity cutback bitumen.

An alternative method of curing is to first apply a very light spray of water followed by either a viscous cutback bitumen, such as MC 3000, or a slow setting emulsion. Neither of these will completely penetrate the surface of the stabilised layer and will leave a continuous bitumen film to act as a curing membrane. It is essential that all traffic is kept off the membrane for seven days. After this time, any excess bitumen can be absorbed by sanding the surface.

A prime coat cannot serve as a curing membrane. Research has shown that a prime penetrates too far into the layer and insufficient bitumen is retained on the surface to provide the necessary continuous film (Bofinger,1978).

8.8.3 Control of Shrinkage/Reflection Cracks

There is no simple method of preventing shrinkage cracks occurring in stabilised layers. However, design and construction techniques can be adopted which go some way to alleviating the problem.

Shrinkage, particularly in cement-stabilised materials, has been shown (Bofinger et al 1978) to be influenced by

- Loss of water, particularly during the initial curing period.
- Cement content.
- Density of the compacted material.
- Method of compaction.
- Pre-treatment moisture content of the material to be stabilised.

Proper curing is essential not only for maintaining the hydration reactions but also to reduce volume changes within the layer. The longer the initial period of moist cure the smaller the shrinkage when the layer subsequently dries.

When the layer eventually dries the increased strength associated with a high stabiliser content will cause the shrinkage cracks to form at increased spacing and have substantial width. With lower cement contents, the shrinkage cracks occur at reduced spacing.

In order to maximise both the strength and durability of the pavement layer the material is generally compacted to the maximum density possible. However, for some stabilised materials it is sometimes difficult to achieve normal compaction standards and any increase in compactive effort to achieve them may have the adverse effect of causing shear planes in the surface of the layer or increasing the subsequent shrinkage of the material as its density is increased. If it proves difficult to achieve the target density, a higher stabiliser content should be considered in order that an adequately strong and durable layer can be produced at a lower density.

Laboratory tests have shown that samples compacted by impact loading shrink considerably more than those compacted by static loading or by kneading compaction. Where reflection cracking is likely to be a problem, it is recommended that the layer should be compacted with pneumatic-tyred rollers rather than vibrating types.

Shrinkage problems in plastic gravels can be substantially reduced if air-dry gravel is used and the whole construction is completed within two hours, the water being added as late as possible during the mixing operation. It is generally not possible to use gravel in a completely air-dry condition, but the lower the initial moisture content and the quicker it is mixed and compacted, the smaller will be the subsequent shrinkage strains.

Having accepted that some shrinkage cracks are inevitable in the stabilised layer, the most effective method of preventing these from reflecting through the bituminous surfacing is to cover the cemented layer with a substantial thickness of granular material. This is the design philosophy presented in ORN 31 (cf. Charts 2,4 and 6 in Chapter 10). When cemented material is used as a roadbase (Chart 8) a flexible surfacing such as a double surface dressing is recommended. Experience in a number of countries has shown that a further surface dressing applied after 2-3 years can partially or completely seal, or arrest, any subsequent cracking, particularly where lime is the stabilising agent.

8.8.4 Quality Control

A high level of quality control is necessary in the manufacture of cement and lime-stabilised materials, as with all other materials used in the road pavement, but several factors need special consideration.

Storage and handling of stabilisers. Unless cement and lime are properly stored and used in a fresh condition the quality of the pavement layer will be substantially reduced. Cement must be stored in a solid, watertight shed and the bags stacked as tightly as possible. Doors and windows should only be opened if absolutely necessary. The cement which is delivered from the manufacturer first should also be used first. Even if cement is properly stored the following losses in strength will occur:-

After 3 months	20% reduction

After 6 months	30% reduction
After 1 year	40% reduction
After 2 years	50% reduction

Lime should be packed in sealed bags, tightly stacked and stored under cover or at least under a watertight tarpaulin. If it becomes contaminated or damp, it can only be used as a filler. Lime which is older than 6 months should also be discarded.

Distribution of stabiliser. After the layer has been properly processed, at least 20 samples should be taken for determination of the stabiliser content. The mixing efficiency is acceptable if the coefficient of variation is less than 30 per cent. Great care is necessary in multi-layer construction to ensure that good mixing extends to the full depth of the upper layer.

Opening to traffic. Insufficient research has been carried out to determine the precise effects of opening a road to traffic before the completion of the curing period but it is considered that allowing traffic on the pavement during the first two days can be beneficial for some stabilised layers provided the traffic does not mark the "green" surface and all traffic is kept off the pavement from the end of the second day until one week has elapsed (Williams, 1986). Early trafficking has the same effect as pre-cracking the layer by rolling within a day or two of its construction.

Layers which are pre-cracked or trafficked early must be allowed to develop sufficient strength to prevent abrasion of the edges of each crack before the layer is opened to general traffic. The slab strength of these layers is effectively destroyed and it is recommended that early trafficking is only acceptable for layers of cemented roadbase type CB2.

Multi-layer construction. When two or more lifts are required to construct a thick layer of stabilised material, care must be taken to prevent carbonation at the surface of the bottom lift. It is also important that the stabiliser is mixed to the full depth of each layer. A weak band of any type can cause overstressing and premature failure of the top lift followed by deterioration of the lower section. In general the thickness of a lift should not be greater than 200mm or less than 100mm.

Process	Purpose	Requirement
Modification	Improvement of access to sites Improvement of workability and pulverisation	Large increase in PL on wet sites. Rapid increase in bearing strength. Large and rapid decrease in plasticity, increased proportion passing 5 mm sieve
Cementation (Stabilisation)	Improvement of sub-grade material Improvement of sub-base and base material	Increase in bearing capacity and durability. Reduced swell Decrease in moisture susceptability Decrease in plasticity Decrease in swell. Increase in shear, compressive and tensile strength and bearing capacity (CBR >100)

Table 8.1 Suggested application of stabilised mixtures (Modified after Ingles and Metcalf 1972)

Code	Description	Unconfined compressive strength* (MPa)
CB1	Cemented roadbase	3.0 - 6.0
CB2	Cemented roadbase	1.5 - 3.0
CS	Stabilised sub-base	0.75 - 1.5 **or,** Min. CBR of 70% (after 7 days moist curing and 7 days soaking)

* Strength tests on 150 mm cubes (see section ***)

Table 8.2 Recommended strengths of stabilised materials (ORN31, (1993))

Description of soil	Amount retained on a BS test sieve	Shape of test sample	Dimensions of test sample
Fine-grained	<1% on 2 mm	Cylindrical: Cubical:	100mm high, 50mm diameter 150 mm cube
Medium-grained	<10% on 20mm	Cylindrical Cubical	100mm high, 50mm diameter 150 mm cube
Coarse-grained	<10% on 37.5 mm	Cubical	150mm cube

Table 8.3 Definition of soil granularity and size of sample for determining stabilised strength.

Specimen size	Correction factor for 150 mm cube
150 mm cube	1.00
100 mm cube	0.96
200mm x 100 mm dia. cylinder	1.25
142mm x 71mm dia. cylinder	1.25
155mm x 105 mm dia. cylinder	1.04
127 mm x 152 mm dia. cylinder	0.96

Table 8.4 Correction factors for size and shape of test samples

SOURCE	EQUATION	COMMENTS
SADAC	UCS = 0.00468*CBR	
TRANSVAAL	$UCS = 15(CBR)^{0.88}$	Corrected to cube strength
TRH	UCS = 0.0065*CBR	
FHWA	UCS = 0.00315*CBR	

Table 8.5 CBR Versus UCS

Type of stabilisation	Soil properties					
	More than 25% passing the 0.075 mm sieve			Less than 25% passing the 0.075mm sieve		
	PI < 10	10<PI<20	PI > 20	PI < 6 PP < 60	PI < 10	PI > 10
Cement	Yes	Yes	*	Yes	Yes	Yes
Lime	*	Yes	Yes	No	*	Yes
Lime-Pozzolan	Yes	*	No	Yes	Yes	*

Notes. 1. * Indicates that the agent will have marginal effectiveness
2. PP = Plasticity Product (see Chapter 6).

Table 8.6 Guide to the type of stabilisation likely to be effective

BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve		
	CB1	CB2	CS
53	100	100	-
37.5	85 – 100	80 - 100	-
20	60 – 90	55 - 90	-
5	30 – 65	25 - 65	-
2	20 – 50	15 - 50	-
0.425	10 – 30	10 - 30	-
0.075	5 – 15	5 - 15	-
Plasticity			
LL	25 max	30 max	-
PI	6 max	10 max	20 max
LS	3 max	5 max	-

Note: It is recommended that materials should have a coefficient of uniformity of 5 or more.

Table 8.7 Desirable properties of material before stabilisation.

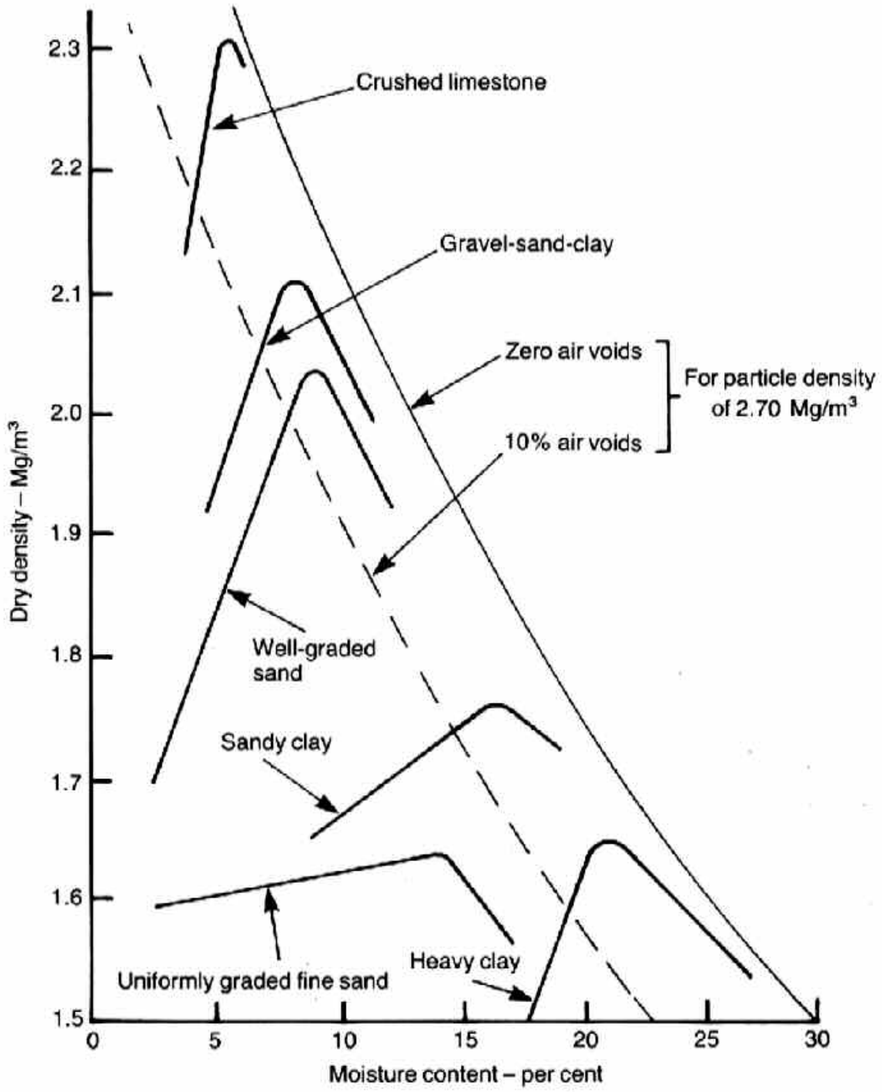
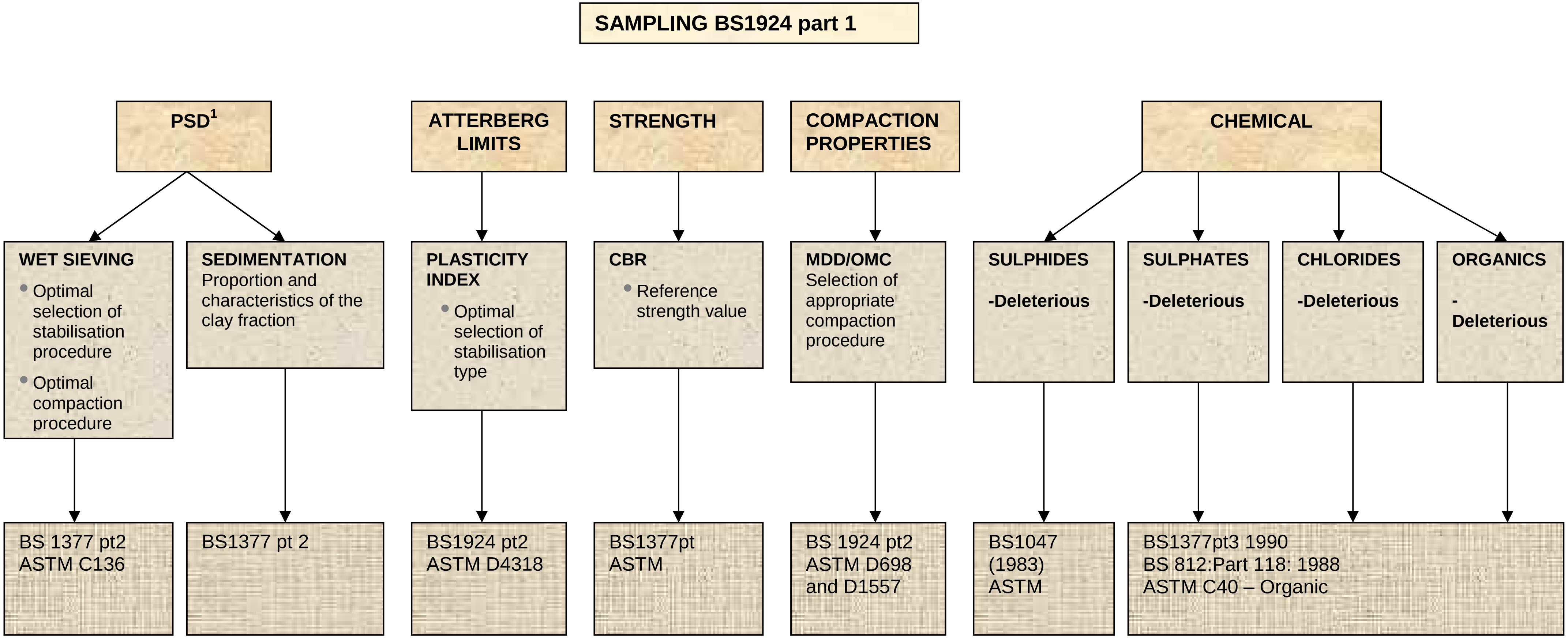


Figure 8.1 Typical moisture content dry density relationship



NB: 1. FOR WASTE MATERIALS AND INDUSTRIAL BY PRODUCTS REFER TO BS 6543

Figure 8.2 Suitability of natural soils for chemical stabilisation

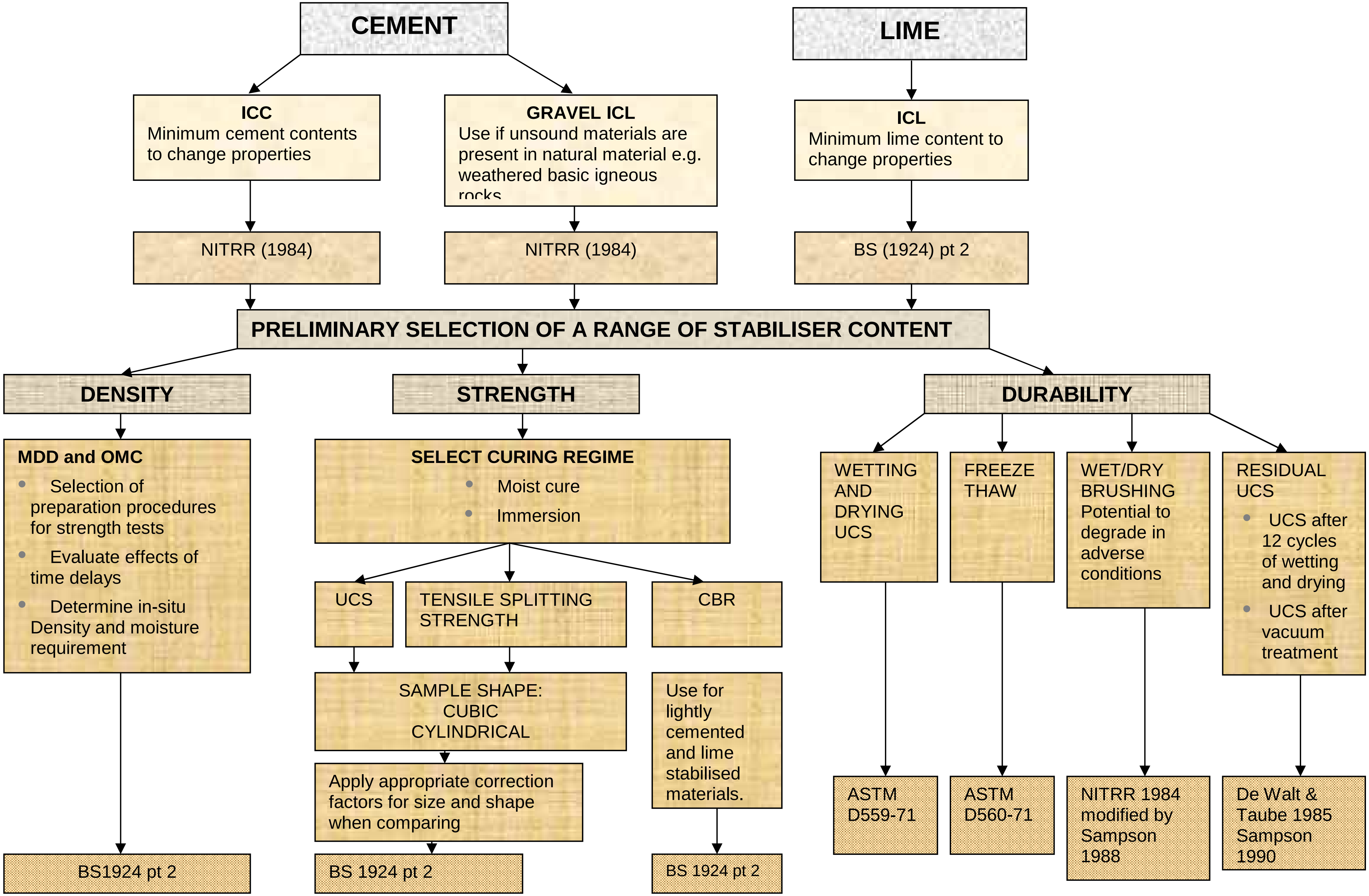


Figure 8.3 Determination of stabiliser content

9 REFERENCES

Anderson M.G, Lloyd D.M. and Kemp M.J.(1997). Hydrological Design Manual for Slope Stability in the Tropics.TRL Overseas Road Note 14.

ASTM (1976). Standard specification for blended hydraulic cements. ASTM C695-76. American Society for Testing Materials.

ASTM (1977). Standard specification for Portland cement. ASTM C595-77. American Society for Testing Materials.

ASTM, (1989). Standard Test Methods for Classification of Soils for Engineering. Purposes (D2487-85) Annual Book of Standards.

Balch J (1972) Supervision and quality control of aggregates and asphaltic concrete construction. Proc. New Zealand Road Symp. 2, 549-561

Barksdale R.D (1991). The Aggregate Handbook. National Stone Association, Washington DC

Baynes J. & Dearman W.R.(1978) The microfabric of a chemically weathered granite. Bull. IAEG. 18 91-100.

Bell F.G. & Maud R.R.(1994) Dispersive soils: A review from a South African perspective. QJEG, 27, 195-210

Bell F.G., Cripps J.C. and Culshaw M. (1990) Field testing methods for engineering geological investigations. IN Field Testing in Engineering Geology (ed F.G. Bell et al), Geol. Soc. Eng Geol. Spec. Pub. No.6, 3-20

Bell F.G., Walker D.J.H. & Jermy C.A. (1998). The nature of dispersive soils in Natal, Soth Africa. 8th Int. IAEG Congress, Balkema. 3357-3363

Belloni L., Morris D., Bellongeri G.A.& Purwoko I. (1988) Compaction and strength characteristics of a residual clay from Bali, Indonesia. Proc. 2nd Conf. on Geomech. in Tropical Soils, Singapore, 1, 343-350

Bemby Sunaryo (1992). Procedures for testing of West Java soils. MSc., Thesis, ITB, Bandung .

Black W. & Lister N.W. (1978). The strength of fill sub-grades, its prediction and relation to road performance.ICE Conf. on Clay Fills. 37-48

Blight G.E. (1988). Construction in tropical soils. Proc. 2nd Conf. on Geomechanics in Tropical Soils, Singapore, 2

Blyth & DeFreitas M H (1990). A geology for Engineers

Bofinger H.E. (1978). Soil-cement: Recent research by the overseas unit of TRRL. Proc. 9th Conf. Aust. Rd. Res. Board.

Bowen N.L.. (1928). The Evolution of the Igneous Rocks Princeton University Press

Boyce J.R. (1985) Some observations on the residual strength of tropical soils.Proc. 1st Int. Conf. Geomech in Trop. Lateritic & Saprolitic Soils, Brasilia, 1, 229-237

Brand E.W. (1985). Geotechnical engineering in tropical residual soils.Proc. 1st Conf. on Geomech. in Tropical Lateritic & Saprolitic Soils, Brasilia, 3.

Brand E.W. & Phillipson H.B. (eds), (1985). Sampling and Testing of Residual Soils.Scorpion Press, Hong Kong

Brindley G.W. & Brown G (1980). X-Ray Diffraction Procedures for Clay Mineral Identification. Min. Soc., London

British Standards Institution (1978). Specification for ordinary and rapid-hardening Portland cement. BS 12. British Standards Institution.

British Standards Institution (1981). Code of Practice for Site Investigations. BS5930.British Standards Institution

British Standards Institution (1983). Specification for pulverised fuel ash for use as a cementitious component of structural concrete. BS 3982; Pt 1.British Standards Institution.

British Standards Institution (1985). Specification for Portland pulverised fuel ash cement. BS6588. British Standards Institution.

British Standards Institution (1989a). Testing aggregates; Method for sampling. British Standard, BS 812: Part 102:1989. British Standards Institution, London.

British Standards Institution (1989b). Testing aggregates; Method for determination of the polished-stone value. British Standard, BS 812: Part 114:1989. British Standards Institution, London.

British Standards Institution (1990a). Testing aggregates; Method for determination of aggregate crushing value (ACV). British Standard, BS 812: Part 110:1990. British Standards Institution, London.

British Standards Institution (1990b). Testing aggregates; Method for determination of aggregate abrasion value (AAV). British Standard, BS 812: Part 113:1990. British Standards Institution, London.

British Standards Institution, (1990) BS 1377: Methods of Test for Soil for Civil Engineering. British Standarda Institution

Broch E. & Franklin J.A., (1972). The point load strength test. Int. Jnl. Rock Mech. Min. Sci. 9, 669-697

Brown & Selig 1991 Aggregates

Buckland A.H.,(1967). The Degradation of Road Aggregates .Proc. New Zealand Road Symp. 2, 622-36

Burland J.B., (1990). On the compressibility and shear strength of natural clays. Geotechnique, 40(3) 329-378

Caldwell G.L. (1985). Rockbuster under test. District Main Roads Dept. Memo No. 59

Cawsey C.D. & Mellon P.A.,(1983). A review of experimental weathering of basic igneous rocks IN Residual Deposits Surface Related Weathering Processes & Materials, (ed. R.C.L.Wilson), Geol. Soc. Spec. Pub. No.11 19-26

Caterpillar Tractor Company (1985). Caterpillar performance handbook. 19th Edition.

Cawsey D.C. & Raymond-Williams R.K., (1990). Stripping of macadams: performance tests with different aggregates. Highways & Transportation, 7, 16-21

Charman J.M. (ed), (1988). Laterite in Road Pavements.CIRIA Spec. Pub. No.47, London

Clark and Walker, (1977).

Clauss K.A. & Loudon P.A. (1971). The influence of the initial consumption of lime on the stabilisation of South African road materials. Proc. 5th Reg. Conf. SMFE, Vol 1, Luanda, Angola.

Cole W.F.& Sandy M.J. (1980). A proposed secondary mineral rating for basalt road aggregate durability. Australian Road Research. Vol 10, 27-37. Aust Road Res. Board, Victoria.

Cook J.R. & Younger J.S.,(1994). The impact of the characteristics of Indonesian soils on construction. Proc. 13th Int. Conf. SMFE, New Delhi

Croney D. & Bulman J.N.,(1975). The influence of climatic factors on the structural design of flexible pavements. 3rd Int. Conf. on Structural Design of Asphalt Pavements, London. 67-71

De Mello V.F.B.,(1972) Thoughts on soil engineering applicable to residual soils. Proc. 3rd S.E. Asian Conf. SMFE, Hong Kong

Dunlop R.J. (1977). Lime stabilisation for New Zealand roads.RRU Tech. Recommendation No.2, Nat. Roads Board, Wellington.

Eastaff D.J., Beggs C.J., and McElhenney M.D.,(1978). Middle East geotechnical data collection. QJEG, 2, 37-55

Eades J.L. & Grim R.E. (1966). A quick test to determine lime requirements for lime stabilisation. Highway Research record 139, Highway Research Board, Wellington.

Ellis C.I. (1974). Village-scale production of lime in Ghana. TRL Sup. Report SR42UC. UK

Fitzpatrick E.A., (1983). Soils: Their Formation, Classification and Distribution. Longman

Fitzpatrick R.W. & LeRoux J., (1977). Mineralogy and chemistry of transvaal black clay topo sequence. Jnl. Soil Sc.28, 165-179

Fookes P.G. & French W.J. (1977). Soluble salt damage to surfaced roads in the Middle East. Jnl IHE, v24, 10-20.

Fookes P.G., Gourley C.S., & Ohikere C., (1988). Rock weathering in engineering time. QJEG, 21(1), 33-58

Frost R.J., (1976). Importance of correct pretesting preparation of some tropical soils. 1st S E Asian Conf. on Soil Eng.

Gee G.W. & Bauder J.W.,(1986). Particle size analysis. IN: Methods of Soil Analysis, A Klute (ed) 9 383-411

Geol. Soc. (1977). The description of rock masses for engineering purposes: Working Party Report. QJEG, vol 10, 355-388.

Geol. Soc.,(1997). Tropical Residual Soils. A Geological Society Engineering Group. Working Party Revised Report. Geological Society Professional Handbooks. 184pp

Geotechnical Control Office (GCO), Hong Kong. (1988). GeoGuide 3: Guide to rock and soil descriptions. Civil Eng. Services Dept.

Gibbs H.J. & Holtz W.G (1956). Engineering properties of expansive clays. Trans. ASCE, vol 121.

Gidigasu M.D (1988). The use of non-traditional tropical and residual materials for pavement construction – a review. Proc. 2nd Int. Conf. On Geomechanics in Tropical Soils, Singapore, Vol 1, 397-404.

Gogo J.O., (1984). Compaction and strength characteristics of decomposed mica schist. Proc. 8th SMFE Regional Conf. for Africa. 275-284

Gourley C.S., Newill D. & Schreiner H.D., (1993). Expansive soils: TRL's Research Strategy. Proc. 1st Int. Symp. Eng. Characteristics of Arid Soils, London

Gourley C.S. & Greening P.A.K. (1999) Environmental damage from extraction of road building materials: Resulst and recommendations from studies in Southern Africa. TRL-DFID Report on Collaborative Research Programme on Highway Engineering Materials in the SADC Region. Volume 3. TRL Report PR/OSC/169/99.

Gourley C.S. & Greening P.A.K. (1999) Establishment of information systems for managing road construction material resources in Southern Africa. TRL-DFID Report on Collaborative Research Programme on Highway Engineering Materials in the SADC Region. Volume 4. TRL Report PR/OSC/170/99.

Hammond A.A., (1984). Semi-Field studies of compaction plant selection for decomposed phyllite. Proc. 8th SMFE Regional Conf. for Africa. 285-292

Harrison D.J. & Bloodworth A.J., (1994) Industrial Minerals Laboratory Manual: Construction Materials. ODA Technical Report by BGS. WG/94/12

Hartley A., (1974) A review of the geological factors influencing the mechanical properties of road surface aggregates. QJEG, 7, 69-100.

Hawkes J.R. & Hosking J.R. (1972). British arenaceous rocks for skid-resistant road surfacings. TRL Report LR488, TRL, UK.

Hawkins A.B. (ed), (1985). Site Investigation Practice: Assessing BS 5930. Eng.Geol. Spec.Pub. No. 2, 423pp

Head K.H., (1986). Manual of Soil Laboratory Testing . Vol 3 Effective Stress Tests. Pentech Press

Head K.H., (1992). Manual of Soil Laboratory Testing . Vol 1 Soil Classification and Compaction Tests (2nd Edition). Pentech Press 387pp

Head K.H., (1994). Manual of Soil Laboratory Testing . Vol 2 Permeability, Shear Strength and Compressibility Tests (2nd Edition). John Wiley & Sons 440pp

Hills J.F & Pettifer G.S. (1985). The clay mineral content of various rock types compared with the methylene blue value. Jnl Chem. Tech.& Biotech. Vol 35a, 168-180.

HMSO (1988). Technical review of the stability and hydrogeology of mineral working. Report for the Dept. of Environment, London,

Hobbs P.R.N. et al, (1988). Preliminary consolidation and triaxial strength test results for some undisturbed tropical red clay soils from West Java. Proc. 2nd Conf. on Geomech. in Tropical Soils, Singapore 1

Hoek E. & Bray J. (1982). Rock slope engineering, 3rd Edition. Insitute of Mining & Metallurgy, London,

Hosking J. R. & Tubey L.W. (1969). Research on low-grade and unsound aggregates. TRL report LR293. TRL UK.

Houben & Guillaud, (1990) Earthworks

Hughes I.R., (1966). Mineral changes of Halloysite on drying. NZ Jnl of Science, 9, 109-113

Hughes R., Lamb D.R. & Pordes O., (1960). Adhesion in bitumen macadam. Jnl of Applied Chemistry.10, 433

Humphries D.W.,(1992). The Preparation of Thin Sections of Rocks, Minerals and Ceramics. Royal Microscopical Society Handbook 24. Oxford Science Publications, 83pp.

Imam Aschuri, (1993). Strength, volume change and index testing for West Java soils. MSc. Thesis, ITB, Bandung

InglesO.G. & Metcalf J.B. (1972). Soil stabilization. Butterworths, Sydney.

International Society of Rock Mechanics (ISRM), (1981). Rock Characterisation, Testing and Monitoring Suggested Methods. (ed. E.T.Brown). Pergamon Press

Jennings J.E. & Knight K.A., (1957). The prediction of total heave from the double oedometer test. Symp. on Expansive Clays. S. African Inst. Civil Eng.

Jennings J.E. & Knight K.A., (1975). A guide to the construction on or with materials exhibiting settlement due to collpase of grain structure. Proc. 6th Regional SMFE Conf. for Africa, \\durban.

Kolaiti E. & Papadopoulus Z., (1993). Evaluation of Schmidt rebound hammer testing: a critical approach Bull. I.A.E.G. 48, 69-76

Lawrence C., Byard R., & Beaven P., (1993). Terrain Evaluation Manual. TRL State of Art Review 7.

Lay M.G.(1985). Source book for Australian roads. ARRB

Leopold L.B.Wolfman M.G. & Miller J.P. (1964). Fluvial processes in geomorphology. Freeman, USA.

Leroueil S. & Vaughan P.R., (1990). The general and congruent effects of structure in natural soils and weak rocks. Geotechnique. 40(3) 467-488

Leung C.F. & Radhakrishnan, (1990). Geotechnical properties of weathered sedimentary rocks. Geot. Eng. 21, 29-48

Linveh M. (1988). In situ testing by indirect methods.

Little A.L., (1969). The engineering classification of residual tropical soils. Proc. Spec. Sess. Eng. Prop. Lat. Soils, 7th Int Conf. SMFE, 1, 1-10

Lohnes R.A. & Demirel T., (1973). Strength and structure of laterites and lateritic soils. Eng. Geol. 7(1), 13-33

Lumb P. (1962). The properties of decomposed granite. Geot. Vol 15, 180-194.

Lyon Assocs., (1971). Laterite and Lateritic Soils and Other Problem Soils of Africa. Study for AID, Building and Road Reserach Inst., Ghana

McGown A., Gabr A.W.A, Barden L., (1975). The importance of representative sampling on measured compression behaviour. 4th S.E.Asian Conf. Soil Eng. Kuala Lumpur, Malaysia, 2, 56-62

McNally G.H. (1998). Soil and rock construction materials. E & F N Spoon, London. 403pp

Mehta P.K. (1979). The chemistry and technology of cements made from rice-husk ash. Proc. Workshop on Cement-like Materials from Agro-Wastes. UNIDO-ESCAP-RCTT-PCSIR. Peshwar.

Mesida E.A., (1986). Some geotechnical properties of residual mica schist derived sub-grade and fill materials in the Ilesha area, Nigeria. Bull. IAEG. 33, 13-17

Metcalf J.B. (1959). A laboratory investigation of the strength/age relation of five soils stabilised with white hydrated lime and cement. RRL Rep. No. RN/3435/1989. Unpublished report, TRL, UK.

Millard R.S. (1990). Road building in the tropics. State of the Art Review.

Mitchell J.K., (1993). Fundamentals of Soil Behaviour. John Wiley & Sons. 422pp

Mitchell J.K., & Sitar N., (1982). Engineering properties of tropical residual soils. Proc. ASCE Geot. Eng. Div. Spec. Conf. Eng. & Const.in Tropical & Residual Soils. 30-57.

Moore P.J. & Styles J.R., (1988). Some characteristics of a volcanic ash soil. Proc. 2nd Conf. on Geomech. in Tropical Soils, Singapore,1, 161-165

Mori H., (1989). Site Investigation and soil sampling for tropical soils. Proc. ASCE Geot. Eng. Div. Spec. Conf., Honolulu, 58-88

Morin W.J., (1982). Characteristics of tropical red residual soils. Proc. ASCE Geot. Eng. Div. Spec. Conf., Honolulu, 172-198.

NAASRA (1986). Guide to stabilisation in roadworks. National Assoc. of Australian State Road Authorities, Sydney.

Netterberg F. (1978). Calcrete wearing courses for unpaved roads. Civil Engineering In South Africa. Vol 20, 129-138.

Newill D., (1961). A laboratory investigation of two red clays from Kenya. Geot. 11, 302-318

Newill D. & Dowling J,W.F, (1969). Laterites in West Malaysia and Northern Nigeria. Int. Conf. SMFE, Spec. Sess. on Eng. Propeties of Lateritic Soils, 2, 133-150

Nik Ramlan, Cook J.R., & Lee Eng Choy, (1994). An approach to the characterisation of tropically weathered soil-rock profiles. Regional Conf. on Geot. Eng. Geotropika '94, Mallacca, Malaysia 3-5

NITRR (1986). Cementitious stabilisers in road construction. TRH 13. National Institute for Transport & Road Research, CSIR, Pretoria.

NITRR, (1987). The Design of Road Embankments: TRH 10. Tech. Recommendations for Highways, CSIR, Pretoria

Novais-Ferreira H. & Meireles J.M.F, (1969). The influence of temperature of humidification on the geotechnical properties of lateritic soils. Int. Conf. SMFE, Spec. Sess. on Eng. Propeties of Lateritic Soils 2, 65-75

O'Connell M.J.& Gourley C.S. (1994). Expansive clay road embankments in arid areas: moisture-suction conditions. IN Characteristics of Arid Soils (eds Fookes and Bray), Balkema, 369-404.

Ogunsanwo O., (1986). Basic index properties, mineralogy and microstructure of an amphiboite-derived laterite soil. Bull. IAEG. 33, 19-25

Ollier C.D., (1969), Weathering. Longman, London

Paige-Green P, Netterberg F. & Sampson L.R. (1990). The carbonation of chemically stabilised road construction materials: guide to its identification and treatment, Research Report DPVT – 123, CSIR, South Africa.

Pettifer G.S.& Fookes P.G. (1994). A revsion of the graphical method fro assessing the excavatablity of rock,QJEG 27, 145-164.

Pike D.C. (1990). International and European standards. IN Standards for Aggregates, Pike D.C (ed). Ellis Horwood, 280pp.

Poole R.W. & Farmer I.W., (1980). Consistency and repeatability of Schmidt Hammer rebound value data during field testing. Int. Jnl. Rk. Mech. Min. Sc.17 167-71

Price D.G., (1993). A suggested method for the classification of rock mass weathering by a rating system. QJEG, Tech. Note. 26(1), 69-76

Ramsay D.M., Dhir R.K & Spence J.M., (1973). Reproducibility of results in the Aggregate Impact Test. The Quarry Manager's Journal, 179-181

Roughton International (2000). Guidelines on borrow pit management for low cost roads. DFID Project Report (Ref. R6852).

Rowe P., (1972). The relevance of fabric to site investigation. Geot. 22, 193-300

Ruddock E.C., (1967). Residual soils of the Kumasi District in Ghana. Geot.17, 359-377

Russam K. & Croney D., (1960). The moisture condition beneath ten overseas airfields. Conf. on Eng. Problems Overseas, ICE, London. 199-206 (+217-234)

Sampson L.R. & Netterberg F., (1989). The durability mill: a new performance-related durability test for basecourse aggregates. The Civil Engineer in South Africa, Sept., 287-294

Sandy M.J. & Cole W.F., (1982). The influence of the degree of weathering of hornfels rock on its physical properties and durability. Proc. 11th ARRB Conf. Part 3, 80-89

Saunders M.K. & Fookes P.G.,(1970). A review of the relationship of rock weathering and its significance to foundation engineering. Eng. Geol. 4, 289-325

Sherwood P.T. (1993). Soil Stabilization with Cement and Lime. TRL State-of the Art Review.

Sherwood P.T & Cavell G.S. (1959). The effect of hydrated lime on the strength/age relations for cement-stabilized organic soils. RRL Note No. RN/3426/1959, (Unpublished TRL report)

Sherwood P.T & Pike D.C., (1984). Errors in the sampling and testing of sub-base aggregates: TRRL Supp. Report 831 TRL, Dept. of Environment and Transport (UK)

Shreiner H.D. & Gourley C.S., (1993). Suction and its measurement. TRL (unpub) Project Report PR/OSC/013/93. 47pp

Smith and Collis (eds), (1993). Aggregates: Sand, Gravel and Crushed Rock Aggregates for Construction (2nd Edition). Geological Society, UK.

Spence R.J.S (1980). Small scale production of cementitious materials. Intermediate Technology Publications, London.

Starkhov N.M. (1967). Principles of lithogenisis. Oliver & Boyd, Edinburgh.

Tesoriere G., Celauro B. & Guiffre O. (1980). Caractere de la stabilisation a la chaux at au ciment d'un sol argileux. Rev. Gen. Routes at Aedr.,570, 67-74.

Terzaghi K., (1958). Design and performance of Sasamua dam. Proc. Inst. Civ. Eng. 9, 369-394

Thornthwaite C.W., (1948). An approach towards a rational classification of climate. Geographical Review. 58, 55-94

TRB (1987). Lime stabilization. Reactions, design and construction. State of the Art Report No. 5, Transport research Board, Washington.

Toole T., (1985). A review of specifications and test methods used in the selection of unbound road bases and surface dressing aggregates in the Tropics. TRL.

Van Schalkwyk A., (1981). Geology and selection of quarry sites. Instit. Quarrying Trans. 414-418

Vaughan P.R., (1985). Mechanical & hydraulic properties of tropical lateritic & saprolitic soils, particularly as related to their structure & mineral components. Proc. 1st Int. Conf. Geomech in Trop. Lateritic & Saprolitic Soils, Brasilia, 3, 231-262

Vaughan P.R., Maccarini M. & Mokhtar, (1988). Indexing the engineering properties of residual soil. QJEG 21 69-84

Wallace K.B.,(1973). Structural behaviour of residual soils of the continually wet highlands of Papua New Guinea. Geot. 23(2), 203-218

Weinart H.H.,(1974) A climatic index of weathering and its application in road construction. Geotechnique 24, 475-488

Weinert H.H. (1980) The natural road construction materials of Southern Africa. Pretoria, Academica.

Weltman A.J. & Head J.M., (1983). Site Investigation Manual. CIRIA Spec. Pub. 25

Wesley L.D., (1973). Some basic engineering properties of halloysite and allophane clays in Java, Indonesia. Geot. 23, 471-494

Williams R.I.T. (1986). Cement-treated pavements. Elsevier. London.

Woodbridge M.E., Cook J.R. & Moestofa B. (1999). Development and implementation of a construction material information system. Proc. 7th Int. Conf on Low-Volume Roads, Louisiana, USA.

Wylde L.J. (1980). Mineralogical factors in the durability of basaltic roadbase materials. Australian Road Research Board Spec. Rep No, 22, 77pp.

Za-Chieh Moh & Mirza Furrukh Mazhar, (1969). Effects of method of preperation on index properties of lateritic soils. Int. Conf. SMFE, Spec. Sess. on Eng. Properties of Lateritic Soils2, 23-35