Scottish Quartz Dolerites: An investigation into the Engineering Properties of Certain Sills and their Aggregates

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by

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MEMORANDUM

The material presented in this thesis summarises the results of three years of research carried out at Glasgow University under the supervision of Dr. C. D. Gribble.

This study is based on my own independent research and any previously published or unpublished results of other researchers used in this thesis have been given full acknowledgment in the text.

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ABSTRACT

The present study sets out to investigate the quartz dolerites of the Midland Valley of Scotland; in particular:

1. Petrographical and geochemical properties.

2. Geotechnical properties of intact rocks and their aggregates.

3. Correlations between their petrography, geochemistry and engineering properties.

Petrographical and geochemical studies were carried out on samples obtained from various sites, and the main variations in the rocks are caused by grain size and degree of weathering (or alteration). Variations occur at the same site, particularly in thick sills, where the upper part of the middle zone of the dolerite sheet is coarser in grain size and more prone to alteration. A micropetrographic index (*Ip*) has been calculated from the ratio of the volume of sound constituents/volume of unsound components (including voids, fractures etc.) which indicates the degree of change from fresh quartz dolerite to altered quartz dolerite extremely well, and this ratio is then used as an index to correlate with engineering tests results. A set of engineering tests was carried out on all the quartz dolerites sampled to evaluate their quality both as intact rock and aggregate.

The correlation between *Ip* and the engineering properties of the quartz dolerites showed that a value of *Ip* above 5 indicates that the rock material would be suitable as roadstone and concrete aggregate under all conditions.

The classification of the quartz dolerites examined here is based upon the results of the petrographical, geochemical and engineering examinations which divide the quartz dolerites into three groups. The relatively fine grained and fresher quartz dolerites from Cairneyhill and Hillend quarries are in group 1; the more altered material from Caldercruix, Duntilland, Tam's Loup 1 and Boards are in group 2, and the coarser and most altered material from Tam's Loup 2 and Westcraigs quarries are in group 3.

Graphing engineering properties test results against each other, and against different indices suggests that some engineering properties of the quartz dolerites can be predicted from other properties. The usefulness of these relationships lies in the simplicity and rapidity of performing some tests such as the Schmidt hammer test on intact rock and the aggregate impact value test on aggregates.

Aggregates from the quartz dolerites are suitable for most roadstone and concrete applications and a satisfactory performance will be achieved from all the quartz dolerite groups. The aggregate impact value is between 7 and 17, the aggregate abrasion value is between 4.7 and 8 and the polished stone value is between 56 and 63. The best results are shown by group 1 whereas group 3 possesses the poorest results, except for PSV where this group showed the best results, due to alteration improving the skid resistant qualities of their aggregates.

ACKNOWLEDGEMENTS

My gratitude and many thanks go to my supervisor Dr C. D. Gribble who was the inspiration behind this project. Any advancement I have been able to make upon this project was largely due to his guidance, fruitful discussions and advice over the period of study. His critical reading of the manuscript was of great value.

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CHAPTER ONE

INTRODUCTION

INTRODUCTION

1.1 Background and aims

The quartz dolerite intrusions of Carboniferous age provide the most reliable source of good construction material in the Midland Valley of Scotland. The quartz dolerites of the Midland Valley area are used extensively for road metal, concrete aggregate and kerbstone. There are more than 15 operating quarries and many more disused quarries. Abundant dolerite dykes, although worked in the past, are too narrow for modern quarrying operations, and the industry now quarries only the sills.

The requirements of road metal aggregates in engineering construction depend upon the type of construction, the pavement layer, the traffic and climatic conditions. Polishing resistance, for instance, is clearly unimportant for road base, and frost susceptibility is of no importance in the tropics.

The main aim of this study has been to try and develop criteria which could be used to relate the mechanical, petrographical and chemical characteristics of the Scottish quartz dolerites to their value as construction materials. It is possible that only when the petrography and chemistry of a rock are studied together that a quantitative system for accurately predicting its mechanical properties can be constructed.

Suites of quartz dolerite rocks from eight sites in the Midland Valley have been examined petrographically, chemically and mechanically. Most of the results of all these investigations are correlated with each other.

1.2 Geology of the Midland Valley

The region has the structure of an ancient rift valley with the parallel Highland Boundary and southern Upland faults forming the limits to the area (Fig. 1.1). The down-faulted strip between the two faults is underlain by rocks mainly of Devonian and Carboniferous age. Comparatively small inliers of lower Palaeozoic rocks occur on the south side of the region and rocks of Permian age occur in central Ayrshire.

Sedimentary rocks of Devonian and Carboniferous age underlie about 80 per cent of the area of the Midland Valley in roughly equal proportions. Igneous rocks; mainly of Devonian and Carboniferous age form about 20 per cent of the area. The oldest exposed rocks are the Ordovician and Silurian sandstones, mudstones and conglomerates which occur as inliers in the Lesmahagow area, the Pentland Hills and south Ayrshire. An upward passage from marine strata to terrestrial fluviatile rocks in the Silurian is followed by the semi-arid fluviatile clastic sediments of the lower Devonian.

Great thicknesses of red and grey sandstones and conglomerates with contemporaneous piles of lava were deposited during the Lower Devonian, and these rocks are well exposed on the coast from the Tay estuary north to Stonehaven. Important fault movements occurred on the boundary faults in Middle Devonian times, and no sediments of this age are known in the Midland Valley.

Red sandstones and siltstones of Upper Devonian age were laid down on a peneplaned surface in the eastern and southern parts of the area. In the Carboniferous period igneous activity occurred more or less throughout the period at one locality or another and large quantities of basalt lavas were extruded. The alkali basalt activity was interrupted during the late Westphalian or early Stephanian by the intrusion of a widespread suite of tholeiitic sills and dykes with no known extrusive equivalents.

Differential movement during the Carboniferous on fractures in the basement caused notable variations in thickness within the region. Coals, ironstones, limestones and oil-shales, formed the basis of the industrialisation of the Midland Valley during the nineteenth and first part of the twentieth centuries.

The Permian sandstones are the youngest sedimentary rocks present in the Midland Valley. Dykes associated with the Tertiary volcanic centres of Mull and Arran are intruded into the strata of the Midland Valley and are the most recent solid rocks in the area. Erosion during the Tertiary and glaciation during the Quaternary combined to create the present landscape (Cameron and Stephenson 1985).

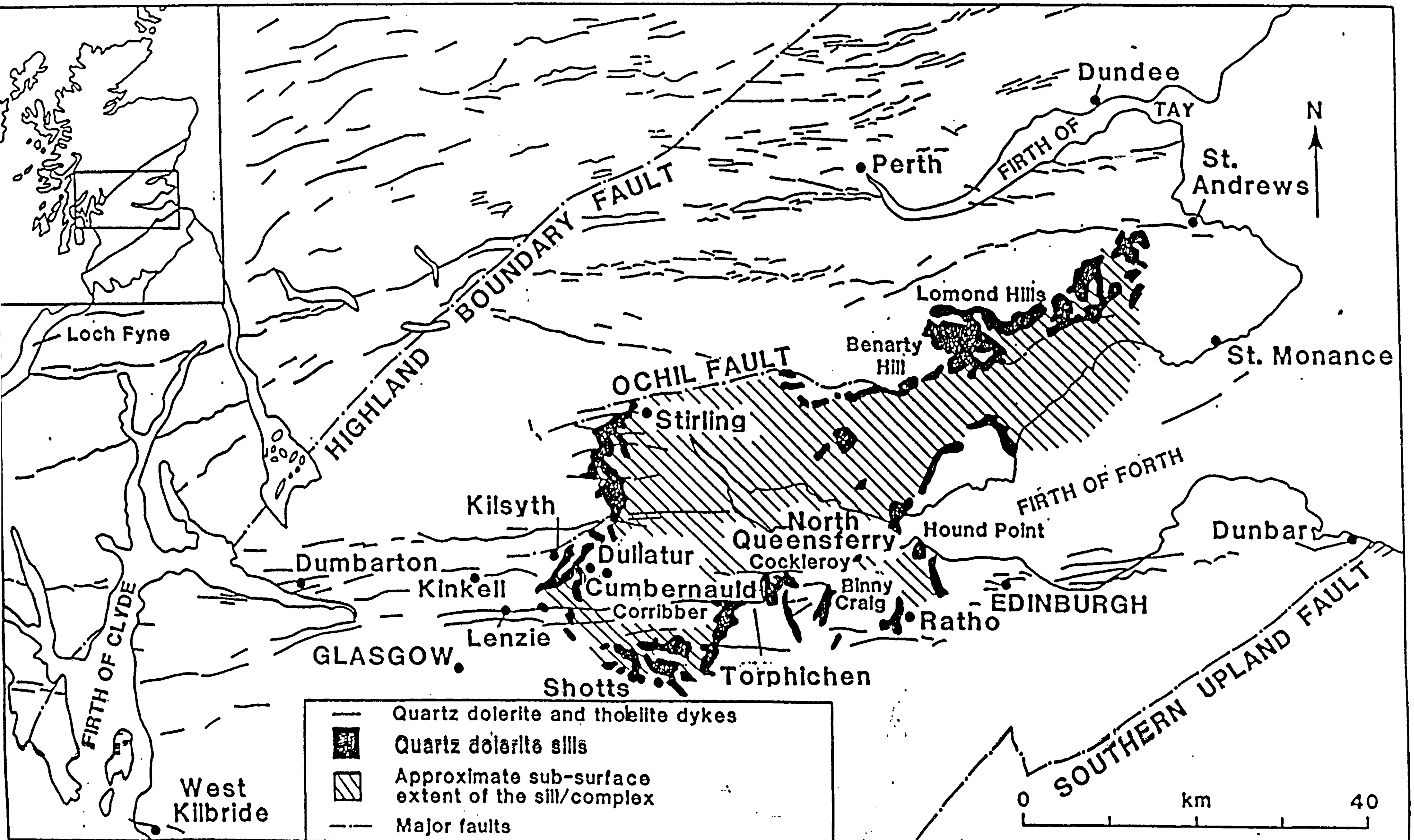

Fig.1.1 Late Carboniferous quartz dolerite and tholeiite intrusions in the Midland Valley. (after Cameron and Stephenson, 1985)

1.2.1 *Quartz dolerite intrusions*

One of the striking features of the geology of central Scotland is the presence of quartz dolerite dykes and sills (Fig. 1.1). The dykes commonly follow E-W fault planes and postdate their main movement. Such faults are known to displace the Coal Measures and in the off-shore Fife coalfield quartz dolerite intrusions cut Westphalian B strata. Field evidence of an upper age limit comes mainly from Fife and is inconclusive. Blocks of quartz dolerite occur in several volcanic vents, most significantly in those of Ardross and St Monance, which vents are considered to be of late Stephanian age, but also in the vents of Viewforth and Lundin Links which are of less certain age.

Dolerite dykes have been traced more or less continuously in the Midland Valley for distances of 20 to 80km; particularly the Campsie dyke which stretches from the Firth of Forth to the Firth of Clyde, and a dyke passing just south of Perth which extends from Fifeshire to Glen Artney. The thickness of these dykes ranges from 15m to 40m, and they intrude rocks ranging from Lower Old Red Sandstone to Coal Measures.

Sills are formed when the structure and stresses in the crust make it easier for the magma to spread laterally rather than vertically. The magmatic pressure may exceed the load-pressure exerted by the overlying rocks.

Thick sills of quartz dolerite are prominent in the Carboniferous sediments and have long been regarded as genetically connected with the dykes. The sills are confined almost entirely to strata of Carboniferous age, the one possible exception being the sill of Corriefoldy in Strathmore where the country rock is of Lower Old Red Sandstone age. They are common in the middle and eastern portions of the Midland Valley but are completely absent from the Carboniferous strata west of Glasgow. The sills often form a prominent hill or scarp feature, and the more important sills vary in thickness from about 20m to 100m, and extend over an area of 1600km^2. The most notable example of these sills is the Bold Craig on which Stirling castle stands, and other important outcrops also occur in the central coalfield near Blackridge, in the Caldercruix-Shotts area, and at Kilsyth (Fig. 1.1).

In most districts, both dykes and sills tend to stand out as prominent grey features of the landscape owing to their superior hardness, and it is only in regions where the country rock is resistant to weathering and thickly covered by drift, that exposures are poor. The grey colour is due to the blending of white and black minerals, the white being feldspars and micropegmatite and the black the pyroxene augite and iron ores (Chapter 2).

As larger structural features, the quartz dolerites of the Midland Valley of Scotland frequently occur in a rude columnar form, to be seen in the large quarries and in the natural escarpments. Dolerite weathers into large cuboidal blocks with rounded edges and corners. In an extreme phase of weathering it breaks down to a brown or red earth. The specific gravity for the normal variety varies from 2.88 to 2.93, whilst a coarse grained rock with large areas of pink micropegmatite has a specific gravity of 2.66

The relationship between the dykes and the complex of sills within the Midland Valley of Scotland has been discussed by many workers. There are no indications of feeder dykes or pipes in the lower, central parts of the complex, but various E-W dykes in the southern part of the regional swarm have long been considered as possible feeders since some are seen to pass locally into sills (e.g., the Lenzie; Torphichen, Dullatur and Cumbernauld dykes). On the northern margin of the sill complex, the Ochil fault intrusion, a series of irregular doleritic pods in the plane of the Ochil fault, is also considered to be connected with the sill complex and is a possible feeder (Cameron and Stephenson 1985). Macdonald *et al.* (1981) stated that there is some direct, and much circumstantial evidence that the dykes acted as feeders for the quartz dolerite sill complex which is almost entirely restricted to Carboniferous strata within the Midland Valley.

The sill complex follows some stratigraphical horizons for long distances, but also changes horizon along high-angled "risers", some of which are faults. Francis (1982) has shown that the shape of the sill complex approximates to a series of "saucers"; the lowest and thickest parts of which coincide with the centres of syn-sedimentary Carboniferous basins. In detail, many transgressions can be shown to have occurred in a downward sense, often leaving a thin continuation sill at the higher horizon. It thus seems likely that magma was able to "flow" down gently-inclined bedding planes into the centres of basins. At the time of intrusion the sedimentary beds were already dipping at angles up to 5 degrees, and the sills are thickest at the bottoms of basins (Craig 1983).

Petrography, chemistry and age relations have long been used to link the Scottish quartz dolerites with the Whin sills and dykes of northern England. Walker (1964) has referred to this association as the central British tholeiitic province, and estimated the volume of magma to be 492km^3. Francis (1978) supported the suggestion by Hjelmqvist (1939) that the suite may also be traced into southern Norway and Sweden. He concluded that the British rocks may, therefore, be part of a much large tholeiitic province extending across the North Sea.

CHAPTER TWO

PETROGRAPHY and GEOCHEMISTRY

2.1 Introduction

2.2 Petrography

2.2.1 Introduction

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2.3 Geochemistry

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PETROGRAPHY and GEOCHEMISTRY

2.1 Introduction

In this chapter the petrography and geochemistry of the quartz dolerites from different sites in the Midland Valley of Scotland are examined. The information will be used in Chapter 5 in correlations with the engineering results obtained in Chapters 3 & 4.

An attempt was made to obtain a rock core giving a complete vertical section from each quartz dolerite sill at particular sites, but unfortunately this was not possible except at one site near Caldercruix, where a 75mm core of about 30m depth was available. This core was studied macroscopically to see whether any mineralogical or textural variations exist and to see, dependent upon the petrographic examinations, whether the core could then be divided into lengths of similar rock upon which engineering tests would be carried out.

The other sites, namely Duntilland quarry (Shotts), Tam's Loup quarry 1&2 (Harthill), Westcraigs quarry (Blackridge), Cairneyhill quarry (Forrestfield), Hillend quarry (Airdrie) and Boards quarry (Stirlingshire), (Figure 2.1) were sampled and a large number of rock samples were collected and brought into the Department of Geology and Applied Geology for preparation for testing.

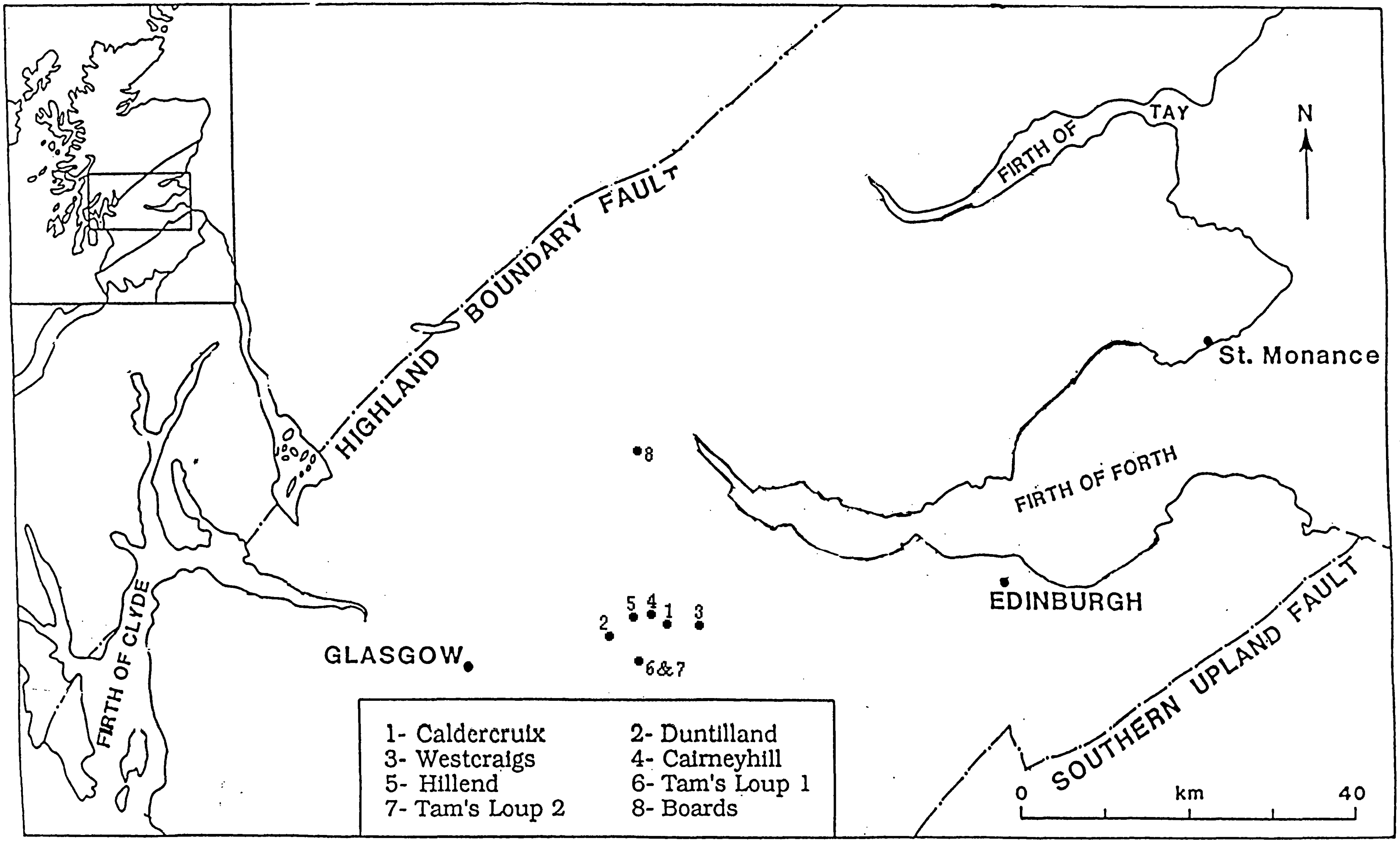


Fig. 2.1 The locations of the quarries sampled in this study.

2.2 Petrography

2.2.1 *Introduction*

An examination of the large and small scale characteristics of a rock used as aggregate is basic to any further testing or selection for use. In many cases, microscopic examination is needed to make a proper assessment of the rock's quality and from this to select the pertinent tests required to be performed on the rock and its aggregate. Important factors such as mineral composition, grain or crystal size, texture characteristics and internal structure must be identified and reasons for their presence understood. These properties either singly or in combination may adversely affect the performance of the rock or its aggregate in engineering services and may affect the safety of the public (e.g., failure of a concrete structure, short pavement life and skid susceptibility of pavement surfaces). Susceptibility of an aggregate to chemical reactions in the high alkaline environments of concrete made with Portland cement is partly a function of the composition of the aggregate.

2.2.2 *Macroscopic features*

In the field, quartz dolerite sills form prominent hills or scarp features due to their resistance to erosion. Excepting the coarse quartz dolerites from Tam's Loup 2 and Westcraigs all quartz dolerites from the other sites are relatively fresh and possess much the same appearance in hand specimen. They are dark grey or black in colour when fresh, and with a greenish tinge when decomposed. Joints and fractures are part of their appearance and the intensity of joints and fractures decreases with increase in depth. The surfaces of the joints and fractures exhibit discolouration due to penetration of subsequent chemical weathering agents, the effects of which gradually decrease from the fracture surfaces inwards. The least altered samples were those collected from the bottom of each quarry face, whereas the most altered samples were those collected towards the upper part of the quarry face and near major discontinuities. In the coarse quartz dolerites sporadic segregation of micropegmatite veins or magma gave rise to pink coloured patches. This coarse variety is invariably restricted to the upper half of the thick sills.

The core obtained from the Caldercruix site was studied macroscopically and the following features were observed; the quartz dolerite core is homogeneous, being hard throughout (it rings when struck by hammer), dark grey to black in colour, and of medium grain size. Another feature recognized was the presence of a pinkish vein of aplite, about 2cm thick, and occuring 18m from the surface. This vein is very conspicuous being of a light pinkish colour in contrast to the dark grey colour of the dolerite and of very much finer grain size than that of the dolerite. The lower part of the core has a finer grain size due to a chilled margin which is just over 1m thick at the lower contact of the sill (Figure 2.2).

The specific gravity of 13 specimens through the core shows a mean value of 2.88. The aplitic vein (specific gravity = 2.66) was not included in this mean value. The drop in the specific gravity of the aplitic vein is due to the reduction of the ferromagnesian minerals in the vein. The apparent slight increase in specific gravity with depth, perhaps due to the increase in ferromagnesian minerals, may reflect gravitational settling of minerals crystallising from the magma.

Rock samples brought from other sites have the same macroscopic features (namely colour, hardness and grain size), except those from Westcraigs and Tam's Loup 2 quarries which are coarser in grain size. Weathered samples from these two quarries are greenish grey in colour and when struck with a hammer do not ring. Some small fissures are present as well.

2.2.3 *Microscopic features*

A microscopic study was carried out for each sample from all sites in order to identify the major mineral constituents of the rock material and also to study the interlocking characteristics of the mineral grains and the weathering state of the quartz dolerites. Off cuts from the cores (which were prepared for sonic and point load tests) were used to prepare thin sections for petrographic examination. Five hundred point counts of mineral constituents were also carried out for each thin section using an automatic point counter to determine the relative frequency of primary and secondary minerals and voids and cracks, if present. Table 2.1 gives the mean and range of the mineral constituents at each site.

Under the microscope, the rocks are generally medium grained quartz dolerites containing essential plagioclase and clinopyroxene, and subordinate iron ore. The texture is sub-ophitic with the plagioclase laths forming an interlocking mesh with the interstitial pyroxenes, which latter are usually subhedral and occuring in granular aggregates. Opaque minerals are mainly iron oxides which in some cases are poikilitically enclosed within feldspar and clinopyroxene. The groundmass mainly comprises intergrowths of quartz and alkali feldspar as well as discrete grains of quartz occupying the interspaces.

Plagioclase

The most abundant constituent in the Midland Valley quartz dolerites is plagioclase (37% - 48%), which occurs as elongate laths exhibiting random orientation, the crystals having a strong tendency towards idiomorphism. Plagioclase laths form an interlocking sub-ophitic mesh with aggregates of large interstitial euhedral or subhedral clinopyroxenes, into which the feldspar laths project. The feldspars range in grain size (longest diminsion) from 0.5 to 0.9mm in the fine grained quartz dolerites and from 1.5 to 2.1mm in the coarse grained quartz dolerites. Albite twinning is common, occasionally combined with Carlsbad twinning. Plagioclase laths show a range of composition (using microprobe) from An_{62} to An_{48}, and the most commonly occurring plagioclase is labradorite (An_{52-60}). The plagioclase crystals occasionally show concentric zoning, resulting from a variation in chemical composition of the melt during crystal growth.

Plagioclase laths in some thin sections are dusky and cloudy, indicating that the mineral has undergone a certain amount of alteration. The degree of alteration varies from site to site; Westcraigs and Tam's Loup 2, for instance, show more altered mineral constituents (Plate 2.1b), in which the feldspars and pyroxenes are partly or sometimes completely altered to secondary minerals. This alteration may be due to either late stage hydrothermal activity during solidification of the rock mass or chemical weathering.

In the interstices between the plagioclase laths a certain amount of alkali feldspar also occurs mainly in micrographic intergrowth with quartz.

Pyroxene

The second most abundant mineral is clinopyroxene, comprising between 16 and 25% of the rock volume. In general it is colourless, occasionally violet to pale brown or purplish with darker central areas under crossed polars.

Clinopyroxene (augite) is the main pyroxene in all of the Midland Valley quartz dolerites and rarely forms well-shaped crystal outlines. The clinopyroxene in the quartz dolerites is rarely zoned though it sometimes has central areas of a darker and browner shade than the margins of the crystals. The central portion of the augite is often occupied by sharply defined rods of chlorite or serpentine. Pleochroism is not noticeable even in comparatively thick sections. Orthopyroxene, altered in varying degrees, is a minor constituent in the dolerites, commonly coarse grained, much cross fractured and sometimes altered to pale green fibrous aggregates of chlorite and hornblende. The amount of orthopyroxene in the coarse grained quartz dolerites is somewhat higher than that in the fine grained ones. It is more liable to decomposition than the clinopyroxene. The fresh mineral shows pleochroism in pink and green tints of variable intensity.

The alteration of the augite is a process of considerable interest. One type of alteration is a peripheral change to brown hornblende. There is no sharply defined boundary with the augite, and irregular flecks of hornblende occur within the unaltered augite. During this alteration iron has been displaced, perhaps reappearing as secondary magnetite associated with the hornblende. Green chlorites are generally fibrous, and with their fibres arranged in curved bands. They are highly pleochroic from straw-yellow to grass-green or, in highly ferrous varieties from reddish brown to dark green. This material certainly includes several varieties of chlorite, and is frequently intergrown with highly pleochroic brown biotite. The final stage of alteration is the production of a pale green serpentine uniformly sprinkled with specks of separated magnetite. The serpentinous material makes its appearance along the cleavages and other cracks, leaving isolated areas of unaltered augite as kernels surrounded by the alteration products.

Iron oxides

Iron ores such as magnetite and some skeletal ilmenite are present in amounts ranging from 5.9 to 8.4%. They occur as large grains, granular clusters and long narrow plates, showing a variation in grain size from 0.2 to 1mm. Ilmenite is a very deep purplish-black in reflected light, whereas, magnetite is black.

Other minerals

The proportion of the groundmass in the rocks varies between 7% and 10% with no systematic variation. It consists mainly of alkali feldspar and grains of quartz, and a greenish chloritic and serpentine material may also be present, particularly in thin sections where the rock has undergone a certain amount of alteration. In the relatively coarse, fresh rocks this material is easily identified as groundmass, but as the rock becomes finer in grain size, the groundmass decreases in quantity and becomes difficult to recognise appearing as a greyish or reddish cryptocrystalline rock glass, riddled with apatite needles, filling up the interstices between the other constituents.

It is, however, prone to decomposition, and occasionally channels have been formed in it which have permited the percolation of weathering agents, and the deposition of secondary substances from solution. The quartz crystals or grains in the groundmass occur throughout all rocks and not just in those which have suffered decomposition. A primary origin of the quartz grains is strengthened by the occurrence in quartz crystals of needles and isolated crystals of apatite and granular augite.

Apatite needles of variable length and breadth are present in very small amounts (0.4% - 0.7%) between plagioclase laths. Hatch *et al.* (1975) stated that apatite seems to crystallize from late stage residuum liquids, and its form reflects an inherent tendency to succeed in building perfect crystals in competition with other, less well endowed minerals by reason of its "pressure of crystallization".

Other accessory minerals present include traces of brown hornblende in the coarse varieties; cinnamon-brown in colour and found in parallel marginal intergrowth with the common pyroxene. The hornblende shows moderate pleochroism. Trace amounts of small dark red-brown biotite flakes occur in some thin sections. The proportions of these two minerals does not appear to be influenced by any other minerals present in the rocks.

No fresh olivine is found either in the coarse or the fine grained varieties of the quartz dolerites, but some pseudomorphs after olivine are present in some thin sections particularly at the margin of the Caldercruix sill. However, it is difficult to decide whether the pseudomorphs are after olivine or orthopyroxene.

Thin sections from the pinkish aplitic vein (in the Caldercruix core) show that the rock is very fine grained. A large number of feldspar crystals occur, among which a certain proportion are orthoclase, as might be expected from the recalculation of the chemical analysis. The amounts of pyroxene and iron oxides are very small, which may explain the drop in the specific gravity of the rock to 2.66, compared with average specific gravity of the dolerite rock of 2.88.

Grain size

The sizes (longest diminsion) of 50 grains of each of plagioclase and clinopyroxene in each thin section were measured, and the average of these measurements taken as the grain size of the rock in that particular thin section.

The average grain sizes of the quartz dolerites from the different sites are between 0.7mm and 1.7mm, with quartz dolerite from Hillend quarry having the smallest average grain size and the largest average grain size being from quartz dolerite from Westcraigs quarry. The average grain size measurements in 15 thin sections from the Caldercruix quartz dolerite are plotted against depth in Figure 2.2, which shows that there is a small decrease in grain size with increasing depth (towards the bottom of the sill).

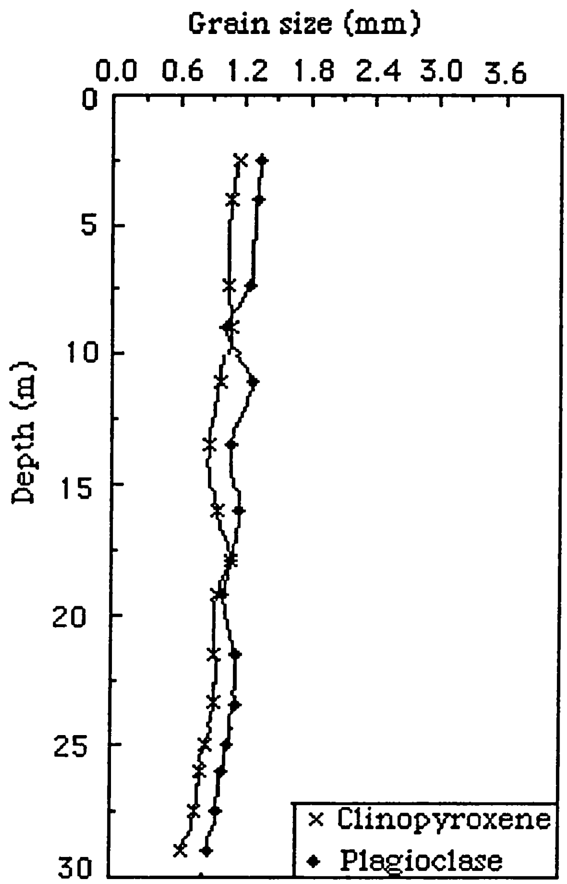
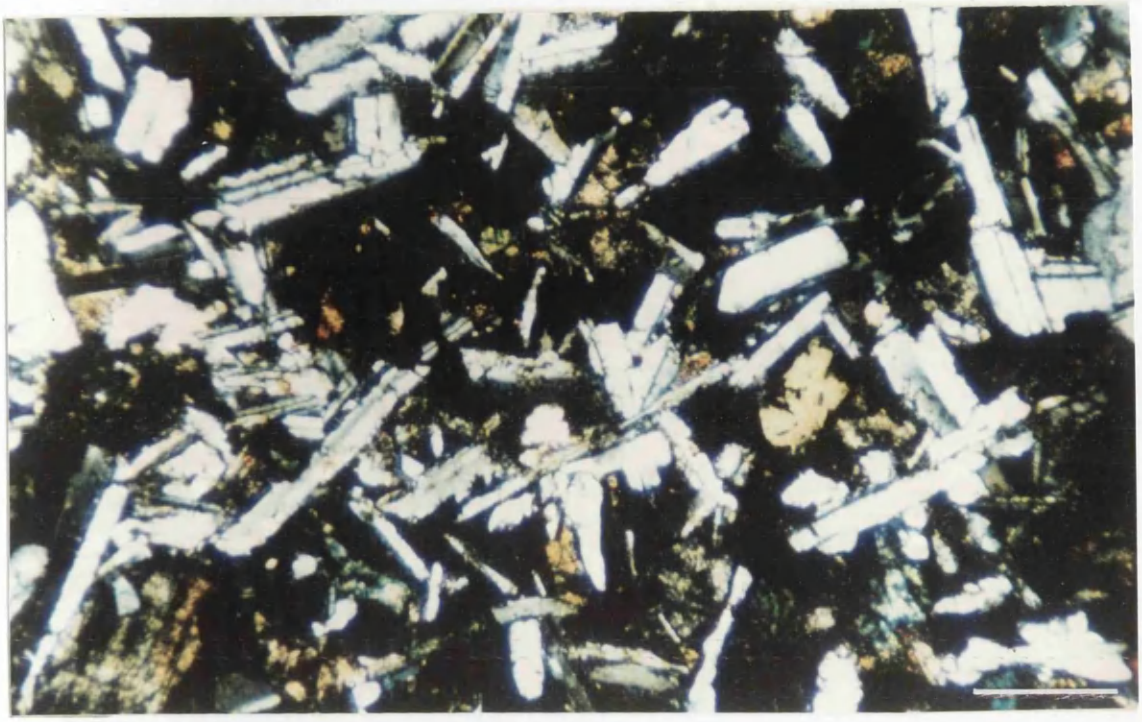


Fig. 2.2 Grain sizes plotted against depth for Caldercruix sill.

Plate 2.1 Photomicrographs of quartz dolerites from the Midland Valley; (a) Hillend quarry, and (b) Tam's Loup quarry. (note the differences in grain size and state of weathering) (Crossed polars, scale bar = 0.5mm)



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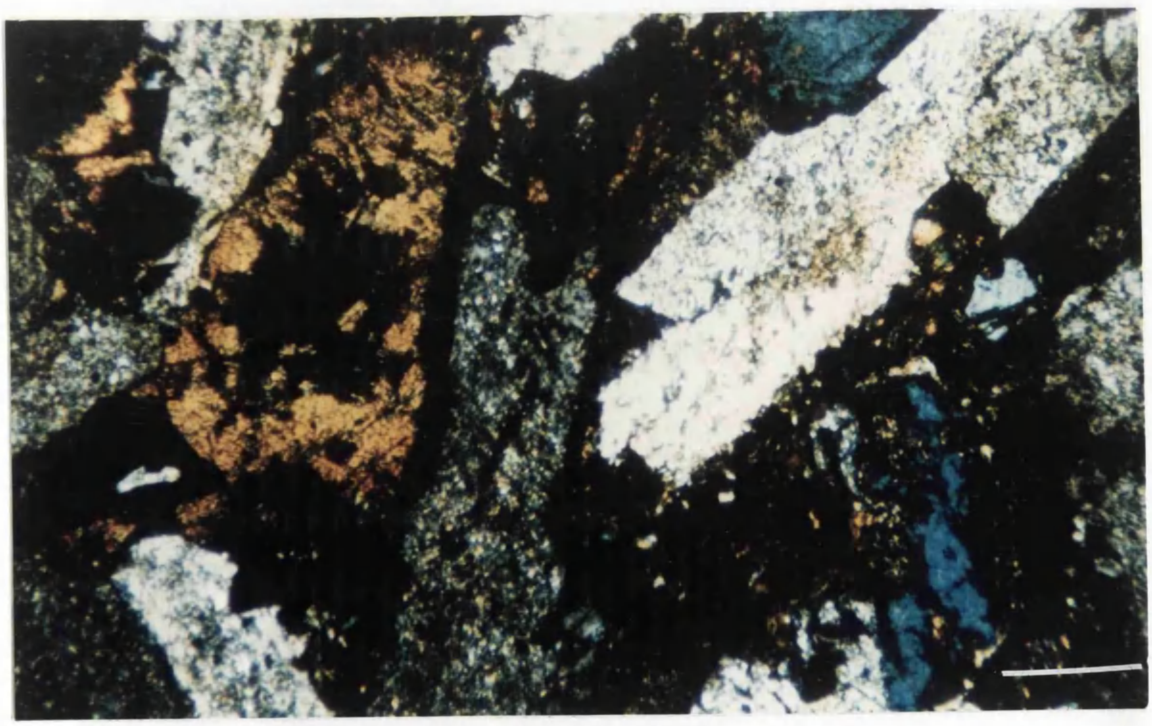

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Site		mineral constituents							grain size
		plagioclase	clinopyroxene	iron ores	ground-mass	quartz	apatite	sec. min.	m m
Caldercruix	mean	45.3	25.0	6.5	7.2	2.7	0.5	11.7	1.0
	range	(43-48)	(23.5-28)	(5.2-7.9)	(6.1-8.3)	(2.0-3.2)	(0.3-0.6)	(9.8-13)	(0.7-1.3)
	no. of samples	4							
Duntilland	mean	44.8	23.4	5.9	7.7	3.5	0.4	13.6	0.9
	range	(42.1-46)	(21-25)	(5-6.3)	(7-8.2)	(2.4-4)	(0.3-0.5)	(12-15)	(0.7-1.0)
	no. of samples	5							
Wsetcraigs	mean	37.2	16.8	7.3	9.6	3.4	0.5	24.0	1.7
	range	(33-39)	(14-19)	(6.9-8.1)	(8.1-10.2)	(2.3-4)	(0.4-0.6)	(21-26)	(1.5-2.1)
	no. of samples	5							
Cairneyhill	mean	47.8	25.8	6.5	7.5	2.6	0.6	9.0	0.8
	range	(45.1-52)	(24-27.1)	(6.0-7.0)	(6.9-8.0)	(2-3.2)	(0.5-0.7)	(7.2-10)	(0.7-0.9)
	no. of samples	5							
Hillend	mean	48.7	24.4	8.4	7.0	2.6	0.4	8.0	0.7
	range	(46-52)	(23-26)	(7.5-8.7)	(6.1-8.0)	(2.0-3.0)	(0.31-0.5)	(7-9.2)	(0.5-0.8)
	no. of samples	5							
Tam's Loup 1	mean	46.3	19.7	7.4	8.0	2.9	0.7	14.2	1.25
	range	(44-49.6)	(17-22.4)	(6.5-8.0)	(7.4-8.8)	(2-3.9)	(0.56-0.77)	(12-16)	(0.9-1.4)
	no. of samples	4							
Tam's Loup 2	mean	39.4	17.1	7.8	10.0	2.8	0.6	21.3	1.6
	range	(36-41)	(14.4-20)	(7-8.2)	(9.1-11.2)	(2.3-3.5)	(0.5-0.7)	(19.1-23)	(1.5-1.9)
	no. of samples	5							
Boards	mean	47.1	24.8	6.9	8.1	3.0	0.4	9.4	1.0
	range	(44.2-50)	(22-27)	(6-7.3)	(7.6-8.9)	(2.0-4.0)	(0.32-0.50)	(8.0-10.0)	(0.9-1.1)
	no. of samples	5							

Table 2.1 Modal analysis data of the quartz dolerites of the Midland Valley of Scotland.

2.3 Geochemistry

2.3.1 *Introduction*

Chemical analyses were carried out on all rock samples which had been selected for petrographic examination and engineering testing to determine whether any chemical differences occurred, and to see if these differences influenced the engineering properties of the quartz dolerite rocks.

The analyses are carried out in a Philips PW1450 sequential X-ray fluorescence spectrometer equipped with a 60 position sample changer and on-line SuperBrain microcomputer for data processing. Ten major elements were determined from fused glass discs prepared from powdered rock according to the method of Harvey *et al.* (1973) and using factors to correct for remaining absorption-enhancement effects.

The major oxides determined were: SiO_2, TiO_2, Al_2O_3, Fe(total), MnO, MgO, CaO, Na_2O, K_2O, P_2O_5. Since the X-ray spectrometer gives total iron, the FeO content was determined by rapid chemical titration methods. The FeO percentage, determined by titration, was then used to calculate the amount of Fe_2O_3 present. The amounts of H_2O and CO_2 in the samples were determined by the combustion method.

2.3.2 *Major oxides variation.*

Table 2.2 gives the average analyses of the rocks at each site. The aplitic vein from Caldercruix is not included in the average results of chemical analysis of dolerite from this site.

The most obvious feature of the analyses is the remarkable chemical similarity of all the fresh quartz dolerites, a feature which has been noticed by many authors (Walker 1935; Francis 1978; Macdonald *et al.* 1981; Cameron & Stephenson 1985). All the analysed rocks are quartz normative, which confirms the petrographic evidence that these rocks are either quartz-bearing or tholeiitic types.

The SiO_2 content shows a narrow range of 49.5% to 51.2% with the quartz dolerite from Boards having the highest amount. There is no systematic variation in SiO_2 content between the fresh and altered quartz dolerites. The Na_2O and CaO contents in fresh samples reflect the abundance of plagioclase feldspar which has been referred to earlier in this Chapter. Weathered samples show a decrease in the amount of Na_2O and CaO due to the alteration of plagioclase feldspar to secondary minerals.

The MgO content of the rocks varies between 4.21% and 5.95% with the highest values being shown by the altered rocks from Tam's Loup 2 and Westcraigs quarries. This is probably because chloritisation of the ferrmagnesian minerals present tends to increase the MgO/FeO ratio of the rock with the secondary chlorite being richer in MgO than the primary mineral (augite etc.) from which it was derived.

An AFM diagram often gives a good indication of the differentiation of a magma (Hatch *et al.* 1975). In Figure 2.3 the quartz dolerites of the Midland Valley show a cluster with a degree of iron enrichment, above the line separating the tholeiite field from the calc-alkaline field according to Irvine and Baragar (1971). Figure 2.4 also illustrates the iron-rich nature of the original magma by plotting the mafic index (Fe_{tot} x 100/ Fe_{tot} + MgO) against the felsic index (K_2O + Na_2O x 100 / K_2O + Na_2O + CaO); from Simpson (1954) and Wager (1956). The mafic indices of the quartz dolerites of the Midland Valley lie between 63% and 70%, and the felsic indices lie between 26% and 46%.

Although the sill complex is usually considered as being representative of quartz tholeiitic magmas, a plot of the total alkalis against silica (Fig. 2.5) shows that the complex is actually intermediate in alkali content between the tholeiitic and alkali basalt magma type, a result confirmed by the work of Cameron and Stephenson (1985). This transitional nature shown in Figure 2.5 tends to support the view that if the magma composition falls near the plagioclase point in the tetrahedron nepheline-olivine-clinopyroxene-quartz (Yoder and Tilley 1962), then small differences in composition may radically change the mineralogy.

Data for the major elements indicate that the chemical variation within the quartz dolerite sills sampled in this study is negligible. The main chemical differences which occur, namely between the fine grained quartz dolerites and coarse grained quartz dolerites, are due to the alteration processes which the coarse grained type has undergone, either from late-stage hydrothermal activity or weathering or both of these.

Weathered quartz dolerites of the Midland Valley in general showed the following changes in major elements geochemistry: a decrease in FeO, CaO and Na_2O, and a relative increase in Fe_2O_3, MgO, K_2O, Al_2O_3 and H_2O.

Some major elements are plotted against depth for the Caldercruix quartz dolerite (Fig. 2.6), to see if variations occur through the sill due to differentiation or alteration. The Fe_2O_3/FeO ratio is also plotted against depth in Figure 2.6, and this ratio gives a measure of the amount of oxidation the rock has undergone and is strongly related to weathering (Moore and Gribble 1980). This ratio does show a slight decrease with increasing depth. SiO_2, MgO and Al_2O_3 remain relatively constant, whereas Na_2O and K_2O also decrease with depth.

The aplitic vein in the quartz dolerite of Caldercruix gives a chemistry typical of an acid to intermediate igneous rock. It is a minor rock type occurring within the dolerite sequences and is common throughout the Midland Valley dolerites. The aplitic analysis therefore has not been included with the normal dolerite plots except in Figure 2.3a. Its chemistry (Table 2.2) shows some affiliation with that of the dolerites and the aplite may represent the final relatively acidic magmatic liquid (from the dolerite magma) which was then injected as a vein complex into the consolidated quartz dolerite sheets.

Table 2.2 Average of major oxides and normative minerals of the quartz dolerites of the Midland Valley.

Wt%	Cal.	Dun.	Wes.	Cai.	Hil.	Ta.1	Ta.2	Bo.	Ap.
SiO_2	51.10	50.00	50.00	50.50	49.91	50.46	49.50	51.20	62.00
TiO_2	2.56	2.20	2.60	2.19	2.19	2.22	2.30	2.23	0.99
Al_2O_3	13.38	13.44	14.37	13.21	13.36	13.55	13.99	13.48	11.80
Fe_2O_3	6.10	5.58	7.61	5.85	5.53	5.77	7.84	4.28	3.00
FeO	7.10	7.59	5.00	7.00	6.97	6.77	5.70	7.93	5.00
MnO	0.17	0.16	0.15	0.16	0.17	0.16	0.16	0.17	0.15
MgO	5.10	5.00	5.75	4.60	5.94	4.21	5.95	4.98	3.10
CaO	7.46	8.07	6.20	8.74	8.93	7.79	6.70	8.84	4.00
Na_2O	2.40	2.30	2.11	2.50	2.29	2.40	2.00	2.31	4.10
K_2O	1.61	1.70	2.00	1.25	1.28	1.76	2.10	0.85	2.89
P_2O_5	0.29	0.26	0.40	0.28	0.27	0.27	0.35	0.26	0.25
H_2O	2.19	2.23	3.00	2.14	1.92	2.39	3.10	2.10	1.91
CO_2	0.24	0.24	0.61	0.29	0.26	0.21	0.51	0.46	0.50
Total	99.70	99.38	99.80	100.0	99.00	99.70	100.20	99.10	99.49
Norm									
Q	6.64	4.94	9.80	8.50	7.76	7.00	10.00	8.62	14.33
Or	9.52	10.10	14.83	7.37	7.57	10.90	14.18	5.10	15.00
Ab	20.83	20.89	12.69	21.46	21.66	21.47	12.94	19.58	20.30
An	22.51	21.98	19.00	22.16	22.32	21.53	24.26	23.88	18.96
Il	4.86	4.16	3.99	4.17	4.19	4.82	3.69	4.23	3.00
Mt	8.80	8.30	8.90	8.48	8.00	8.33	8.57	6.21	5.60
Di-Wo	6.69	7.46	5.11	8.00	8.23	6.60	5.21	6.69	5.67
Di-En	4.22	4.78	4.21	5.27	5.42	4.75	3.81	4.00	3.09
Di-Fs	1.40	2.20	1.10	1.34	1.68	1.26	0.90	1.99	1.30
Hy-En	7.98	7.70	11.10	9.05	9.32	10.30	8.63	10.83	8.56
Hy-Fs	2.37	3.50	2.12	2.32	2.90	2.74	2.00	5.36	3.21
Ap	0.68	0.62	0.93	0.64	0.61	0.61	1.10	0.70	0.48
C	0.55	0.56	2.50	0.67	0.59	0.50	3.31	0.93	1.87

Cal.= Caldecruix, Dun. = Duntilland, Wes. = Westcraigs, Cai. = Cairneyhill, Hil. = Hillend, Ta.1 = Tam's Loup 1, Ta.2 = Tam's Loup 2, Bo. = Boards and Ap. = Aplite.

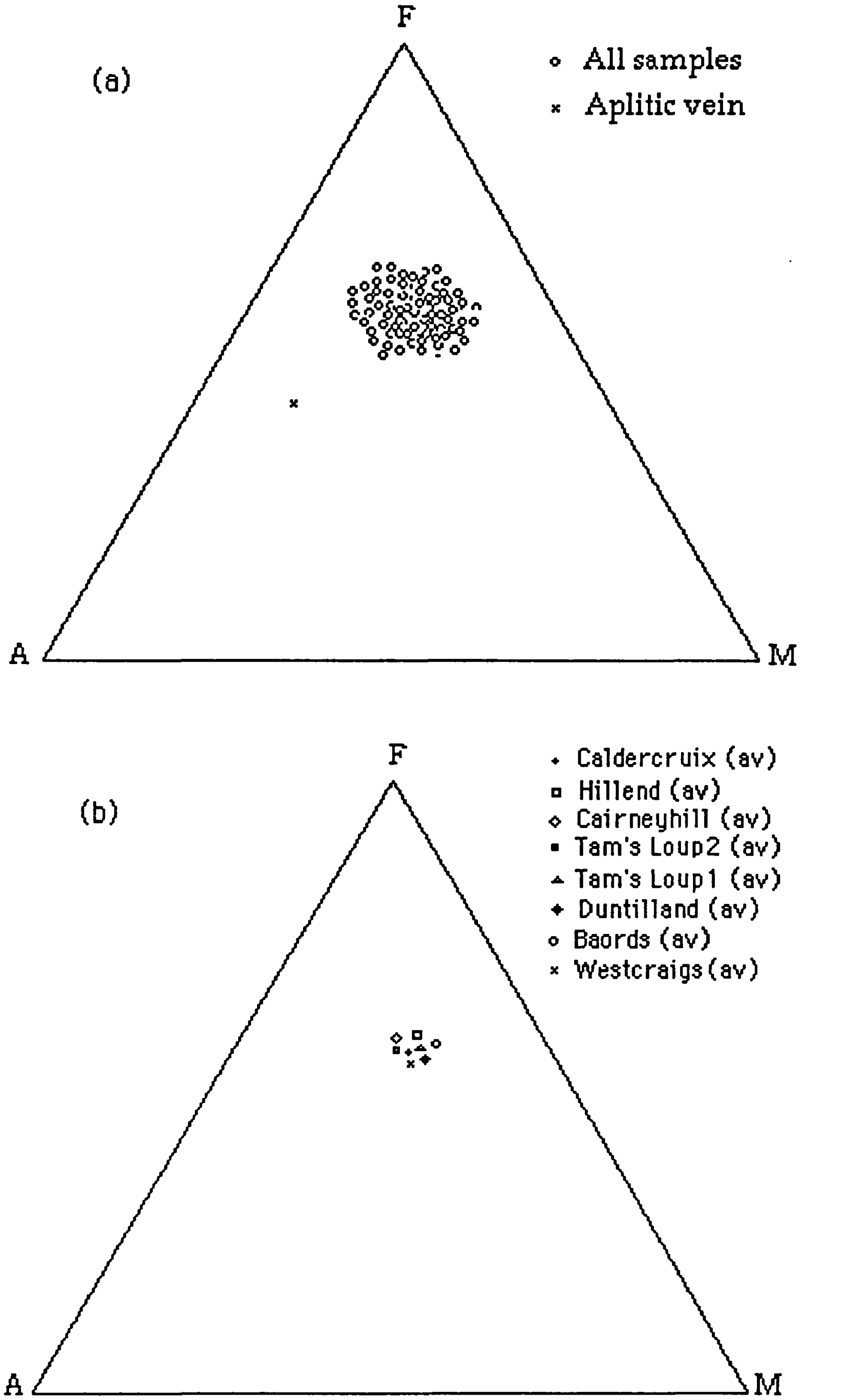


Fig. 2.3 AFM diagram of the quartz dolerites showing the points fall into a cluster with iron enrichment, a. all data and b. the average at each site.

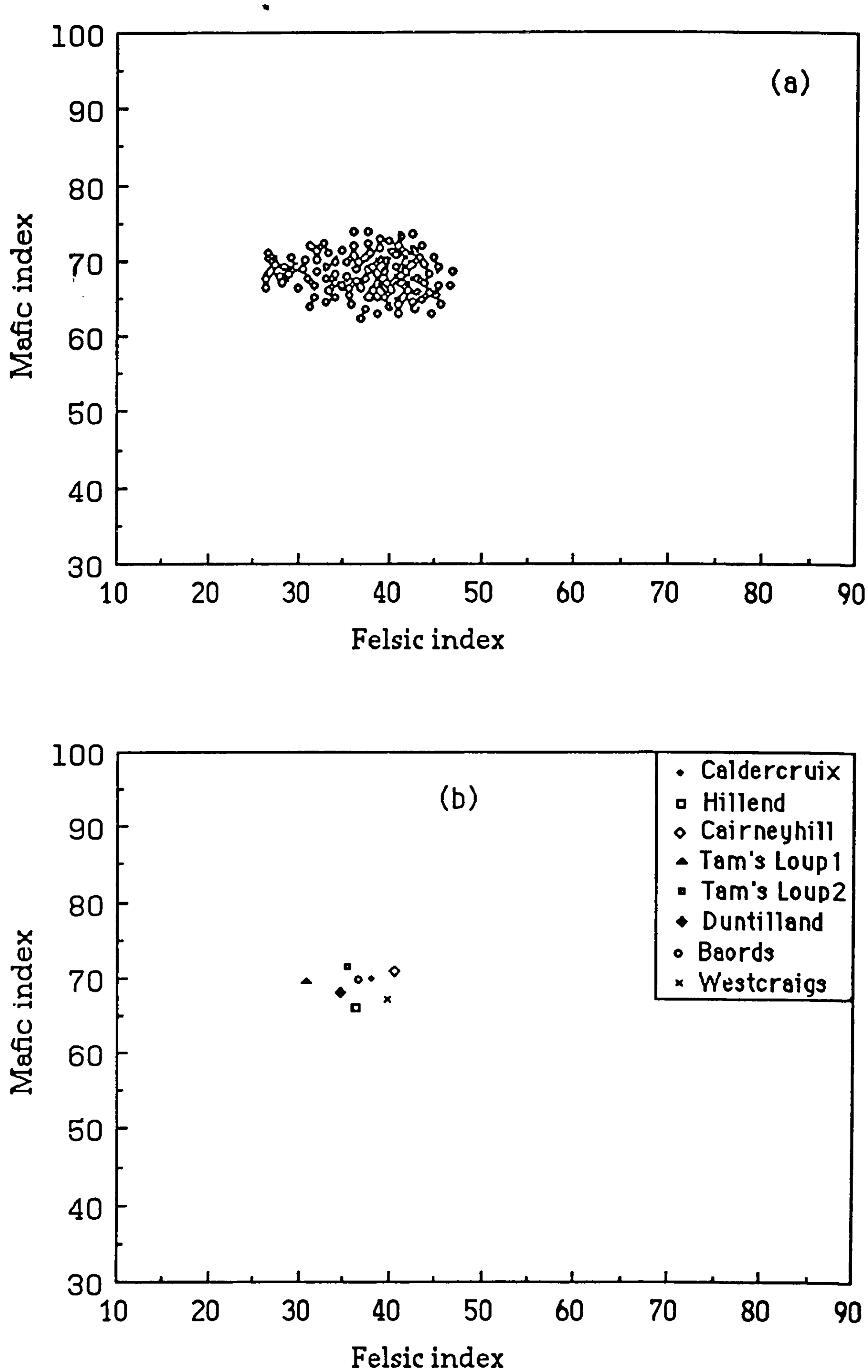
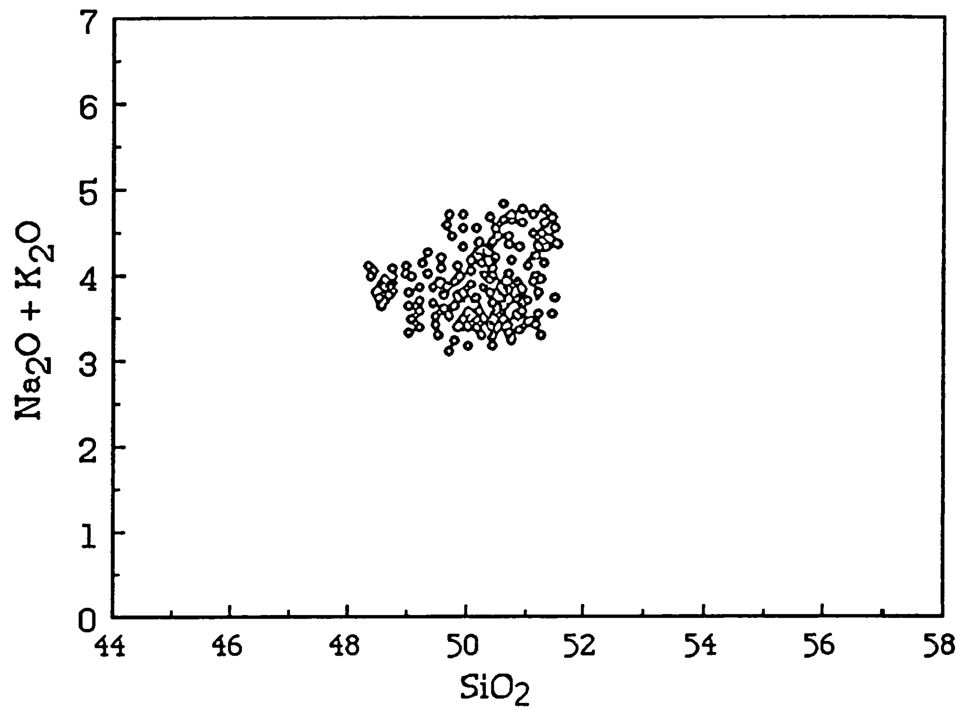
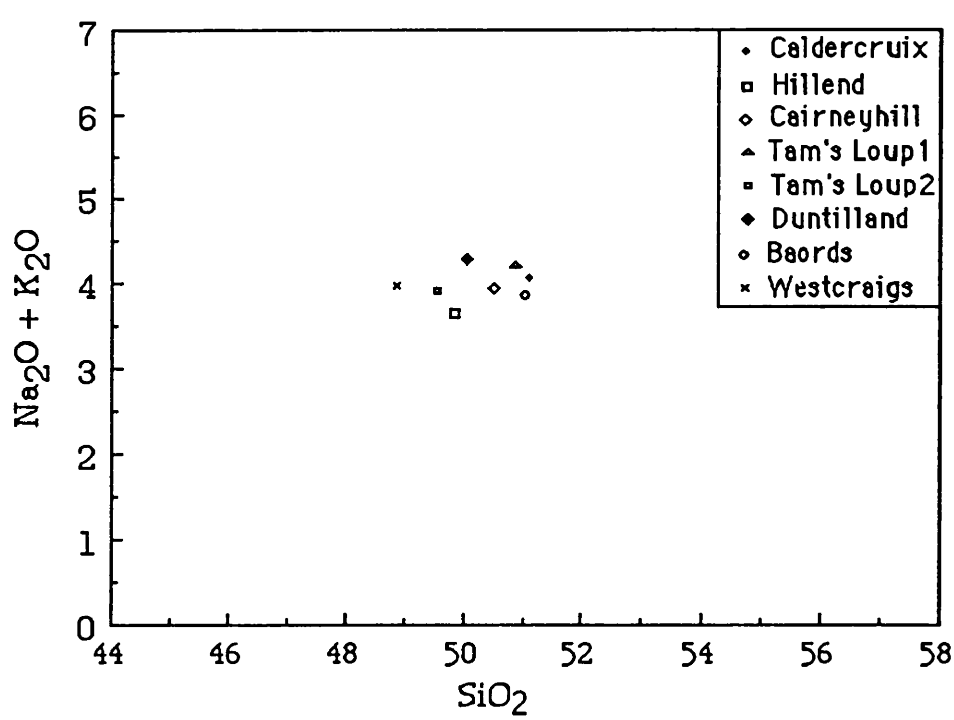


Fig. 2.4 Plot of mafic index against felsic index of the quartz dolerites.
(a. all data and, b. the average at each site)



(a)



(b)

Fig. 2.5 Plot of $Na_2O + K_2O$ against SiO_2 of the quartz dolerites.

(a. all data and, b. the average at each site)

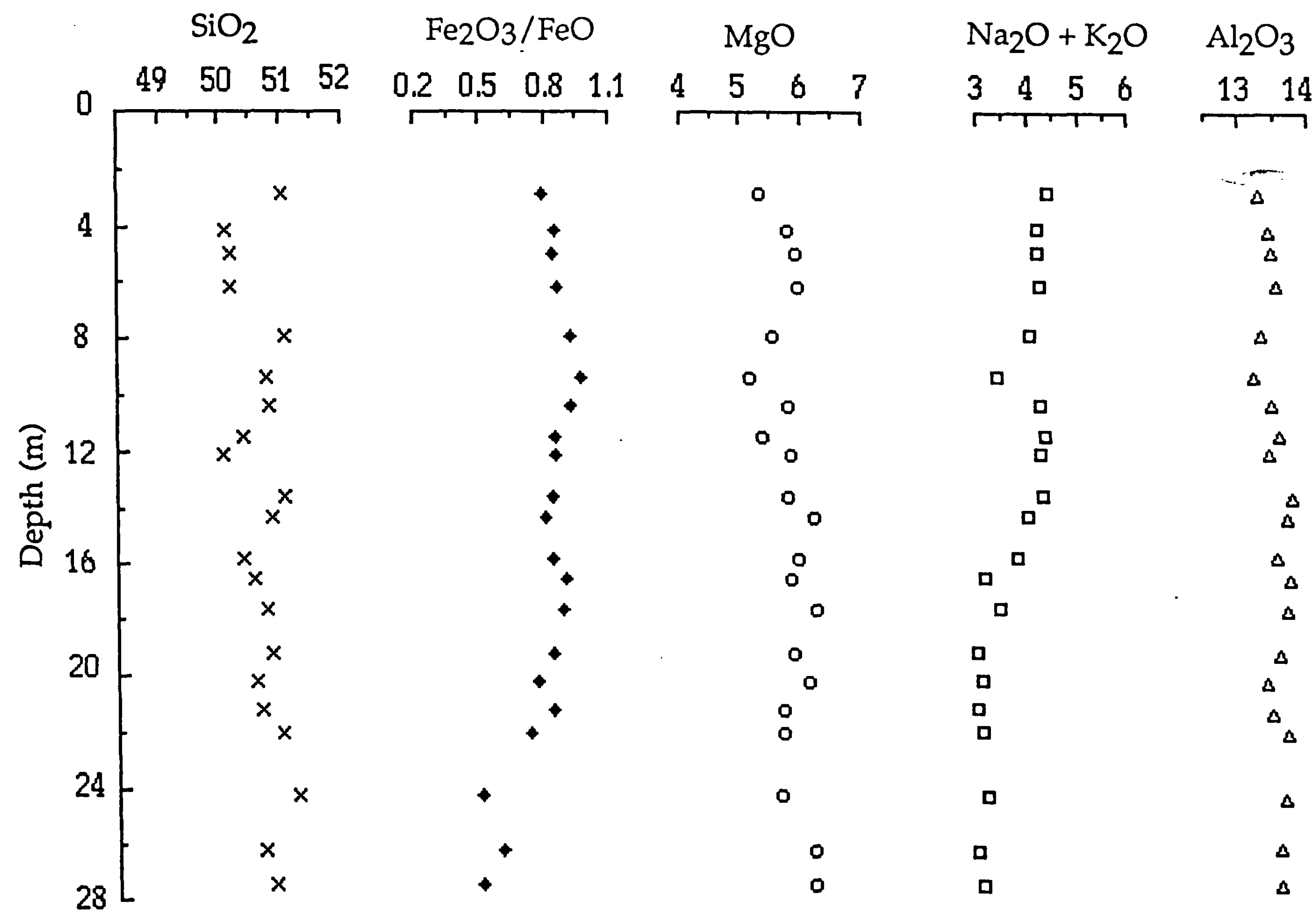


Fig. 2.6 Some major elements plotted against depth of Caldercruix quartz dolerite.

CHAPTER THREE

ENGINEERING PROPERTIES of the QUARTZ DOLERITES

ENGINEERING PROPERTIES of the QUARTZ DOLERITES

3.1 Introduction

This chapter deals with testing of the engineering properties of the quartz dolerites. There are a variety of field and laboratory tests that provide information on strength, deformability and weathering grade for intact rock. The results of such tests are directly applicable to many geotechnical applications such as mining, tunnelling, drilling, cutting, crushing and blasting. The knowledge of rock mechanical properties will help to identify the effects of weathering on the rock mass, and indicate the quality of the rock and the likelihood of its use as aggregate. For the present study, the following tests were carried out to determine the engineering properties of the quartz dolerites.

1. Ultrasonic pulse velocity
2. Uniaxial compressive strength
3. Modulus of elasticity
4. Point load index
5. Schmidt rebound number

Samples of large blocks of various sizes ranging from 200 to 600mm in thickness and 300 to 500mm in width or length were sampled for testing. During the sampling process great care was taken to collect samples which were representative of the dolerite in a particular area. In order to make the best use of the rock blocks brought into the laboratory, they were first cored to provide cylinders for different tests. Off cuts from the cylinders' ends were used in the preparation of thin sections and for XRF analyses for petrographical and chemical investigations. The remainder of each block was crushed to a suitable fragment size for aggregate tests.

Results of tests carried out on dolerites from Caldercruix show that the quality of the rock dose not change throughout the 30m core except the lower 1.5m where the rock becomes darker in colour and finer in grain size. The amount of aggregate from the lower part of the core was not enough to carry out all tests and this part of the sill is not exposed in the field. Thus only some tests were carried out on this section of the sill. However, the results showed very little difference from the rest of the core. The results show that this part of the sill is characterised by lower water absorption and slight increase in the specific gravity. It is thought that this part of sill is not thick enough to influence the engineering properties of the whole sill.

3.2 Ultrasonic pulse velocity

3.2.1 *Introduction*

The ultrasonic pulse velocity test was first used by Anderson and Nerenst (1952) to study the dynamic properties of concrete and, later, Sutherland (1962) used this test to study the dynamic properties of rocks.

In the field sonic velocity in rocks is influenced by a number of factors of which depth of burial, degree of compaction and cementation, discontinuities, fluid saturation of voids, texture, composition and degree of weathering are the most important (McLean and Gribble 1985).

The value of the longitudinal wave velocity, Vp, has been recognized as an indicator of rock mass quality; the higher the velocity the better the rock quality. The basis of this lies in the relationships existing between Vp and various engineering properties of an intact rock sample. In most cases the only additional factor on the larger rock mass scale is the presence and amount of discontinuities, which can impede transmission of wave travel resulting in low velocity and consequently lower rock quality. P-wave (longitudinal wave) and S-wave (shear wave) velocities in a rock provide a means of determining the modulus of elasticity and Poisson's ratio for the rock.

The ratio of the field velocity to the laboratory velocity is the velocity ratio or fracture index (Deere *et al.* 1967; Knill and Price 1972). As the number of discontinuities decreases, field velocity will approach and ultimately equal laboratory velocity for a discontinuity-free rock mass. Knill (1974) found that, in general, a velocity ratio of less than 0.5 indicated significantly fractured rock. The squared velocity ratio (velocity index) has been found by Coon and Merritt (1970) to be approximately the same as RQD in some rock masses (Fig. 3.1).

Caterpillar Tractor Co. (1972) has given ranges of velocity for different types of rocks. The highest velocities occur in fresh crystalline rocks (e.g., igneous rocks and limestones). Dearman *et al.* (1978) have tabulated ranges of velocity for various degrees of weathering in granites and gneisses (Table 3.1).

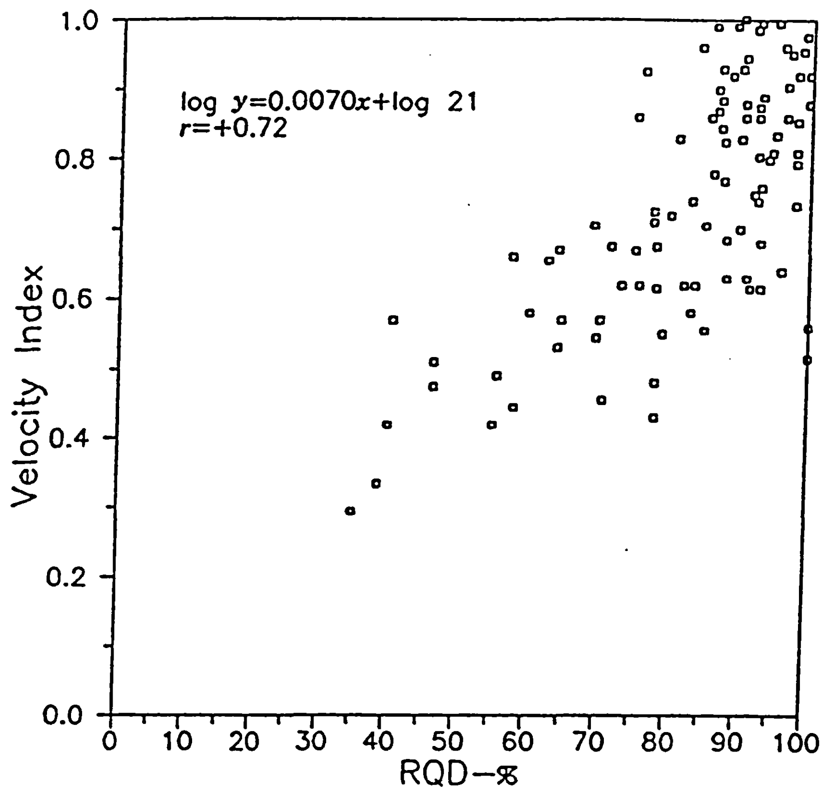


Fig. 3.1 Relationship between RQD and velocity index. (after Coon and Merritt, 1970)

Table 3.1 Ranges of P-wave velocity in granites. (after Dearman *et al.*, 1978)

Fresh	3500-5500m/sec
Slightly weathered	2500-4000m/sec
Moderately weathered	1500-3000m/sec
Highly weathered	1000-2000m/sec
Completely weathered to residual soil	500-1000m/sec

3.2.2 *Results and discussion*

In the present study the P-wave velocity was measured with an ultrasonic timing unit known as the PUNDIT (Portable Ultrasonic Non-destructive Digital Indicating Tester). This generates suitable pulses and accurately measures the time of their transmission through the material being tested. Suitably sized transducers coupled for perfect contact with the specimen were used. A film of grease was smeared on each core end in order to get a better contact with the transducers. However, on many occasions the test had to be repeated many times (because of poor contact) until good contact was achieved. The distance which the pulses travelled in the material (the path length) was measured by Vernier calliper. Having measured the P-wave transit time in the rock specimen and the length of the specimen, the P-wave velocities were calibrated by dividing the path length by the transit time as follows:

$$Vp = L/T \tag{3.1}$$

Where

Vp = P-wave velocity (m/s)

L = distance traversed by the wave (m)

T = wave transit time in seconds (s)

More than 100 core specimens, each of 100mm height and 51mm diameter, were prepared for the ultrasonic pulse velocity test. P-wave velocity measurements were carried out on three specimens from each sample, at atmospheric pressure and room temperature conditions. The mean of these three values was taken as the P-wave velocity of that particular sample (Appendix 2).

Table 3.2 gives the mean and range value of P-wave velocity values of all the samples tested at each site. The highest velocity values are from Hillend quartz dolerite (5573 m/sec, sat. & 5442 m/sec, dry) which may be related directly to the finer grain size and lower porosity of rock. On the other hand the lowest velocities obtained were from Westcraigs quartz dolerite (3480 m/sec, sat. & 3050 m/sec, dry) because of the weathering and higher porosity. However, P-wave velocity measurement results, in general, show very little variation particularly in quartz dolerites from Caldercruix, Duntilland, Cairneyhill, Hillend, Tam's Loup 1 and Boards due to the similarity in composition, texture, porosity and state of weathering of the quartz dolerites from these locations.

In Figure 3.2, a positive relationship was obtained between P-wave velocities of saturated and dried specimens, with the velocities of saturated specimens being higher than the velocities of dry ones. Saturated rocks have higher P-wave velocities than dry ones since sonic velocity in water is higher than that in air. The regression line equation is:

$$Vps = 845 + 0.86\ Vpd \qquad r = 0.99 \qquad (3.2)$$

where Vps = P-wave velocity in saturated samples

Vpd = P-wave velocity in dried samples

The relationship between the P-wave velocity and the dynamic modulus of elasticity (E_d) for the quartz dolerites is shown in Figure 3.3. It is a linear relationship, as is expected because of the use of the P-wave velocity in calculating E_d. This relationship has a correlation coefficient of 0.97.

One of the most useful correlations involving P-wave velocity is its relationship to uniaxial compressive strength, and in Figure 3.4 the P-wave velocity values obtained in this study are plotted against uniaxial compressive strength values and a reasonably good positive relationship is obtained with correlation coefficient of 0.93 and 0.92 for saturated and dry samples respectively; high velocities are associated with high strength values. As the values of the P-wave velocity and the uniaxial compressive strength increase the difference between results of dry and saturated samples decreases. This shows that rocks with high P-wave velocity also possess low porosity and high strength. Such a graph can be used to predict the uniaxial compressive strength using P-wave velocity, which may be particularly useful since the sonic velocity test is non destructive and may be carried out on non-standard sized specimens.

Table 3.2 P-wave velocity measurements of the quartz dolerites(m/s).

Site		saturated	oven dry
Caldercruix	mean	5264	5166
	range	(5120 - 5350)	(5000 - 5280)
	no. of samples	(4)	
Duntilland	mean	5347	5240
	range	(5230 - 5496)	(5130- 5370)
	no. of samples	(5)	
Westcraigs	mean	3480	3050
	range	(3290 - 3600)	(2967 - 3300)
	no. of samples	(5)	
Cairneyhill	mean	5514	5390
	range	(5471 - 5600)	(5299 - 5460)
	no. of samples	(5)	
Hillend	mean	5573	5442
	range	(5486 - 5640)	(5390 - 5500)
	no. of samples	(5)	
Tam's Loup 1	mean	5100	4825
	range	(5000 - 5250)	(4900 - 5100)
	no. of samples	(5)	
Tam's Loup 2	mean	4290	4000
	range	(4050 - 4420)	(3898 - 4250)
	no. of samples	(4)	
Boards	mean	5339	5214
	range	(5270 - 5460)	(5100 - 5290)
	no. of samples	(5)	

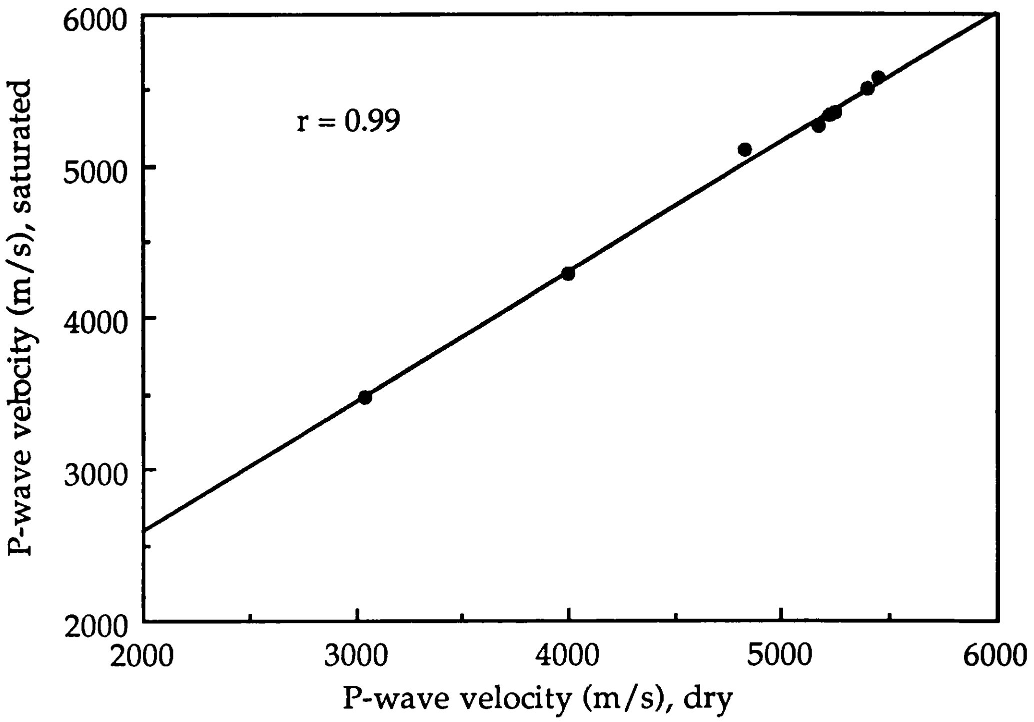


Fig. 3.2 P-wave velocities in dry and saturated specimens plotted against each other.

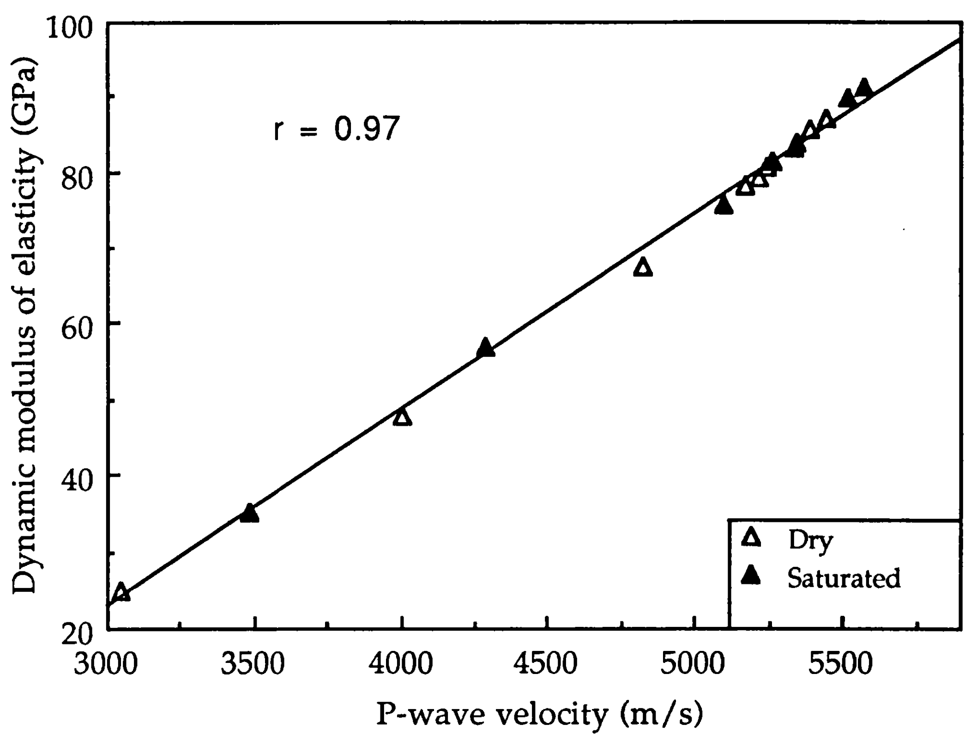


Fig. 3.3 Relationship between P-wave velocity and dynamic modulus of elasticity of the quartz dolerites.

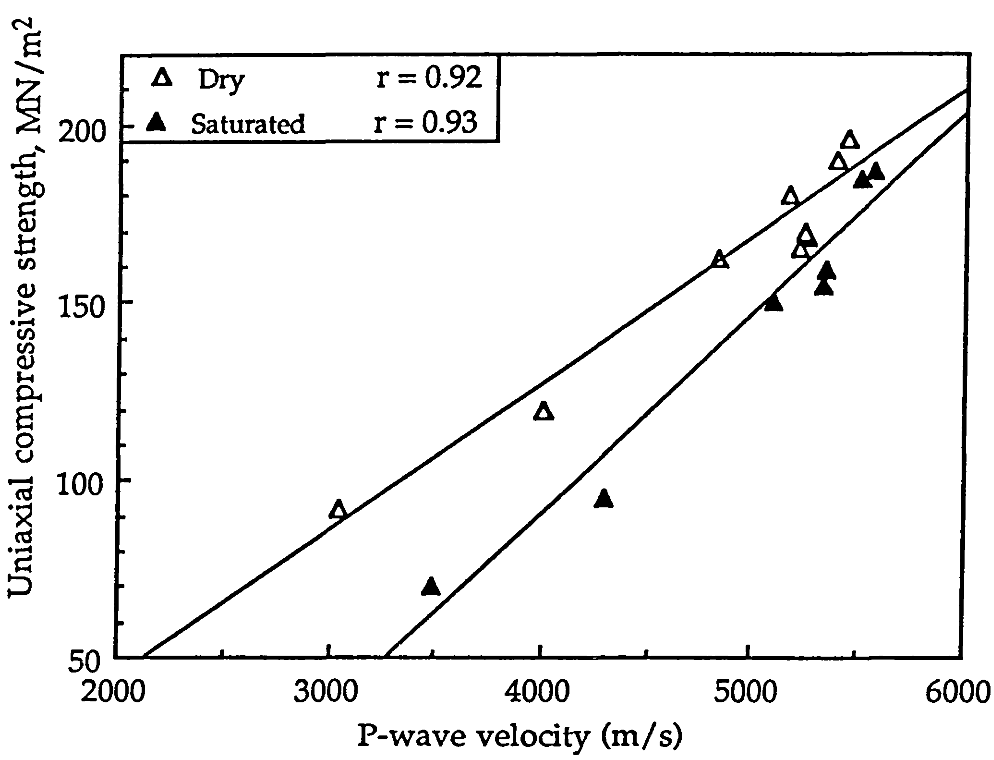


Fig. 3.4 Relationship between P-wave velocity and uniaxial compressive strength of the quartz dolerites.

3.3 Uniaxial compressive strength

3.3.1 *Introduction*

Strength is a fundamental design property of a rock engineering material. Since aggregates come from solid rocks which have been crushed, strength tests on the parent rock can give an indication of the likely strength of the aggregate. Such tests can be carried out directly by the uniaxial compressive strength test (Hawkes & Mellor 1970) or by an indirect test such as either the Brazilian test (Mellor & Hawkes 1971) or the point load index test (Broch & Franklin 1972). The uniaxial compressive strength is the most measured and used test in rock mass classification schemes, and as a basic measure of the rock strength, because it is easier and faster to carry out and yields the minimum value of rock strength under compression. Many authors have tried to define uniaxial compressive strength of rocks and the clearest definition comes from Krynine and Judd (1957) as "the stress required to break a loaded sample that is unconfined at its sides".

In the uniaxial compression test a cylindrical specimen of rock is loaded axially between platens in a testing machine. The load value at failure is devided by the cross section area of the specimen. The result is defined as the compressive strength of the specimen and is given by the relationship:

$$S_c = N / A \qquad (3.3)$$

Where S_c = uniaxial compressive strength of the specimen

N = normal applied load at failure

A = cross-sectional area transverse to the direction of force

Hawkes and Mellor (1970) in their detailed study of the uniaxial compression test defined three broad modes of failure which are observed in compression testing. The first, cataclasis, consists of a general internal crumbling by development of multiple cracks parallel to the applied load at mid-height of the specimen near the surface and its extension to the ends and into the centre of the specimen; when the specimen collapses conical fragments are left, together with long slivers of rock from around the periphery. The second is "axial cleavage", or vertical splitting, in which one or more major cracks split the specimen along the loading direction resulting in a series of columns. This type has been termed by Gramberg

(1965) axial-cleavage fracture. Gramberg (1965) was the first to suggest the tensile nature, reflecting indirect or induced tensile stress, of axial cleavage fracturing in the uniaxial compression test. The third mode of failure is the shearing of the test specimen along a single oblique plane.

The first mode of failure is the most common. The second one is observed in some cases, caused by tensile stress generated by differential compressive loading (Gramberg 1965). The third mode of failure is likely to be due to either platen rotation or lateral translation of the platens relative to each other or the presence of oblique cracks or weaknesses.

3.3.2 *Factors influencing the uniaxial test*

Factors influencing the uniaxial compression test have been discussed by many authors; among them, Krynine and Judd (1957), Hawkes and Mellor (1970), Goodman (1989) and others. The important factors can be divided to two major categories, namely the nature of the rock, and methodological factors.

A. *Nature of the rock*

1. <u>Mineralogy and texture</u>

Many authors, such as Price (1960), Vutukuri *et al.* (1974) and Johnson and DeGraff (1988) have confirmed the effect of constituent minerals on the strength of a rock. The presence of clay minerals in igneous rocks due to weathering processes tends to reduce the strength of the rock (Dearman

1976; Irfan and Dearman 1978). Teme (1991) showed that the strength of limestones from Nigeria increased as the quartz content increased. The mechanical properties of a rock also depend on the way in which the crystals are assembled (i.e., texture). Among igneous and metamorphic rocks those with better and higher degrees of crystal interlocking are generally stronger than those with a poorer or lower degree of crystal interlocking. In sedimentary rocks, the degree and type of cementation has a great influence upon their strength.

2. Porosity and density

For rocks of similar mineralogy the compressive strength will decrease with increasing porosity (Kowalski 1966; Smorodinov *et al.* 1970; and others). The bonding between minerals in a rock depends on the total area of contact and the cementing material between individual particles. This is inversely related to the amount of pore space. The effect of pore space in sedimentary rocks is obvious when the strengths of well cemented and low porosity sandstones are compared with those of high porosity and weakly cemented sandstones. The latter possess low strengths and porosities as high as 30 per cent. In metamorphic and igneous rocks a large proportion of the pore space is in planar cracks termed fissures. In igneous rocks, porosity is usually less than 1 or 2% unless the rock is weathered, and as the weathering progresses the porosity tends to increase. As a result of this the strength of the rock decreases.

A relatively small porosity due to fissures affects the properties of the rock to the same degree as a much larger percentage of sub-spherical pore space (Goodman 1989). Pores and fissures create a nonlinear load/deformation response (especially at low stress levels), reduce the tensile strength and produce variability and scatter in test results. Hoshino (1974) has established an empirical relationship between porosity and strength for clastic sedimentary rocks. Rocks of low porosity have, in general, high density. Thus density has a direct relationship with strength, i.e rocks of high densities give high strength values. Figure 3.5 illustrates the approximate relationship between strength and bulk density.

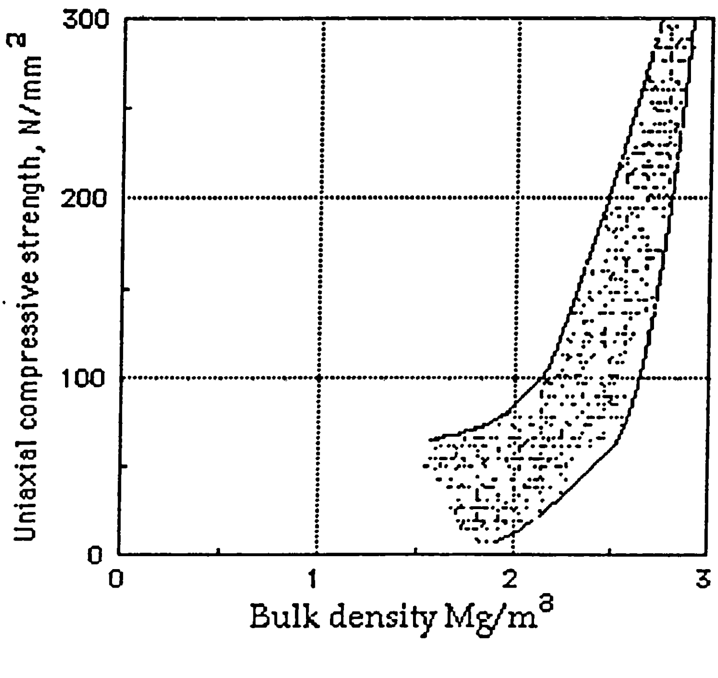


Fig. 3.5 Approximate relationship between strength and bulk density of.
(after Attewell and Farmer, 1976).

3. Grain size and shape

The controlling defect structure, known as the Griffith Crack, is considered either to lie within the grain or along the grain boundary. Since failure is identified with crack initiation, the longer the grain boundary the more rapidly will any cracks spread; and the shape and size of the constituent grains would control the crack propagation and therefore the strength of a rock (Attewell and Farmer 1976). Thus, resistance to failure should be greater in a fine grained rock than in a coarse grained one, and the strength should be consequently higher.

4. Water content.

It has been shown (Colback and Wild 1965; Duncan 1969; Parate 1973; Broch 1974; Turk & Dearman 1986) that the uniaxial compressive strength of rocks decreases with increasing water content. Colback and Wild (1965) have defined relationships between uniaxial compressive strength and water content for sandstone and shale. They showed that the strength of saturated rocks was only about half the strength of completely dry ones. Ruiz (1966) has given data for fifteen rock types, showing in most cases that dry strength is higher than wet strength. Hawkes and Mellor (1970) have found similar relationships for uniaxial compressive strength and Brazil tensile strength of sandstone, limestone and granite as a function of water content. Broch (1974) carried out an experiment on the influence of water on some rock properties and concluded that water content plays a major role during the uniaxial compressive and point load tests. Turk and Dearman (1986) showed that the saturated strength is lower than the dry strength and gave the following relationship:

$$S_cs = 0.23\, S_cd \qquad r = 0.96 \qquad (3.4)$$

where S_cs = uniaxial compressive strength of saturated specimen

$S_c d$ = uniaxial compressive strength of dried specimen

To understand the role played by water in affecting the uniaxial compressive strength of a rock, a number of hypotheses have been put forward by many workers. The pressure of water on the internal surfaces within the rock produces static fatigue, which may involve reduction of surface energy (Vutukuri *et al.* 1974), whereas Pugh (1967) suggested that modification of the interlocking bond between particles (or minerals) due to the existence of pore water pressure in a saturated specimen may result in decreasing rock strength.

5. Anisotropy

Variation of compressive strength according to the direction of the principal stresses is termed "strength anisotropy" (Donath 1964). Jovanovic (1970) described anisotropy in rocks as variation of physical properties in different directions. Certain rocks, particularly non-igneous rocks are characterised by orientation of mineral grains, bedding planes, laminations, or foliations. They exhibit specific directional properties, and hence compressive strength depends on the orientation of the rock specimen to the direction of acting load (Goodman 1989). The highest compressive strength is obtained when the compressive stress is normal to bedding (Al-Jassar and Hawkins 1977). The existence of fissures and cracks in an igneous rock specimen may make it anisotropic and greatly influence its strength.

B. *Methodological factors*

In order to obtain reliable results from tests on rock, careful and precise specimen preparation and testing are very important. This is influenced by methodological factors such as:

1. <u>Coring rate</u>

In the preparation of a test specimen the optimum drilling speeds vary with bit size and rock types, and to some extent with the condition of the bit and the characteristic of the machine (Hawkes and Mellor 1970). The main objections to high drilling speeds seem to be "chatter" and vibrations in the machine which could produce bending on the test specimen. The rate of coring should be kept more or less constant to eliminate any bending on the rock core, since this would have the effect of reducing the measured compressive strength. The feed-rate is also important, too much load may produce distortion.

2. <u>Ends of the core</u>

The core specimen must be carefully prepared to achieve satisfactory flatness and parallelism, so as to obtain an evenly distributed load during testing, which is a prerequisite to obtaining reliable and reproducible results. The ISRM commission (1979) recommends that in a 50+ mm diameter specimen, the ends should be flat to within 0.02mm and should not depart from the perpendicular to the specimen axis by more than 0.05mm.

3. Aspect ratio

The stress distribution varies throughout the specimen as a function of specimen geometry (length/diameter) ratio. Hawkes and Mellor (1970), Szlavin (1974) and Vutukuri *et al.* (1974) showed that the uniaxial compressive strength increases as the aspect ratio (L/D) decreases. Specimens with large L/D ratio fail due to elastic instability, while specimens with small L/D ratio exhibit very high compressive strength. The fall in compressive strength is very rapid as the ratio increases until it equals 2 after which strength becomes more or less constant (see Fig. 3.6). Hobbs (1964) has shown that there is an apparent dependence of fracture angle on L/D for very short samples. It is recommended (ISRM 1979) that the L/D ratio is to be ≥ 2 so that a plane of failure may be formed without being influenced by the restraints of the ends.

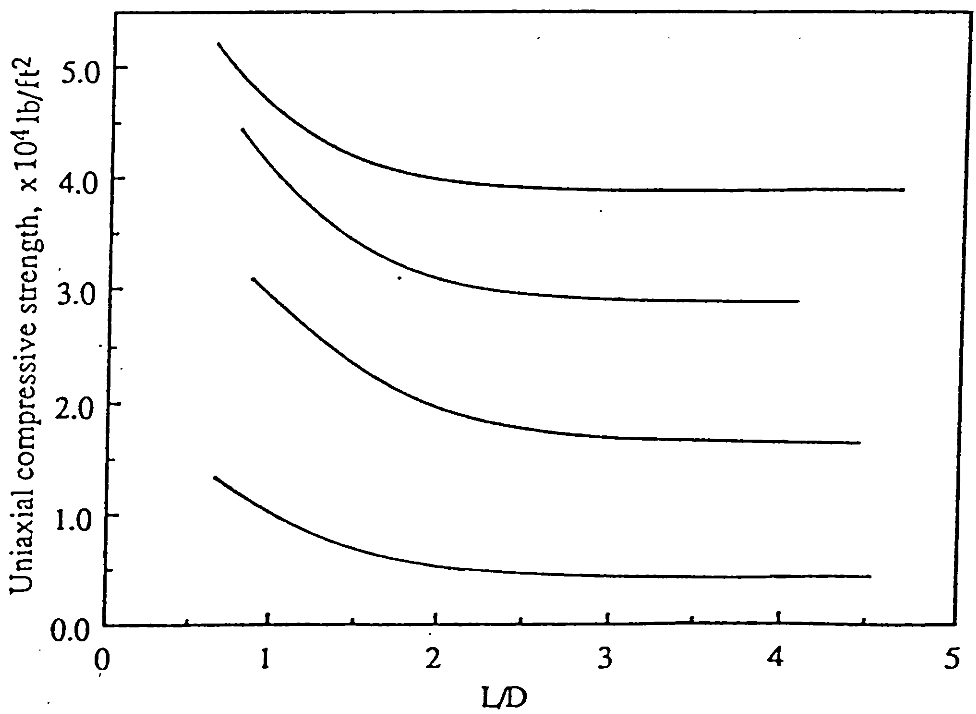


Fig. 3.6 Influence of length/diameter ratio (L/D) on uniaxial compressive strength. (modified after Hawkes and Mellor, 1970)

4. Loading rate

The loading rate must be under control and the chosen rate should be maintained constant throughout the test. When the loading rate increases the strength and modulus of elasticity also increase and subsequently the strain at failure decreases (Houpert 1970 and Perkins *et al.* 1970). Houpert (1970) investigated the effect of the loading rate on compressive strength of granite. His results (Fig. 3.7) reveal an increase in strength with an increase in loading rate. The suggested stress application rate for a uniaxial compression test is 0.5 - 1.0 MPa per second (ISRM 1979).

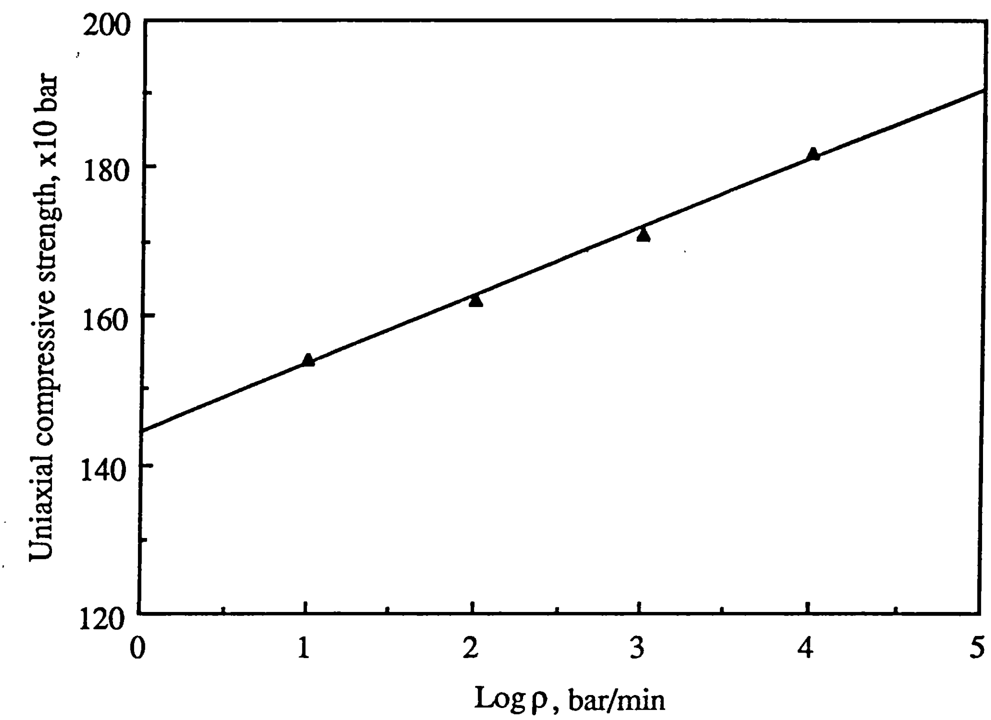


Fig. 3.7 Variation of compressive strength with loading rate, ρ.

(after Houpert, 1970)

5. <u>End effects</u>

Brady (1971) showed that due to friction between the specimen ends and the platens and differences between the elastic properties of rock and steel, the specimen will be restrained near its ends and prevented from deforming uniformly. To eliminate end effects some workers suggested treating the specimen-platen interface with a lubricant or inserting a sheet of material between the specimen and the platen. Hawkes and Mellor (1970), Jaeger and Cook (1969) and the ISRM (1979) recommend that treatment of the sample ends, other than by machining, be avoided.

3.3.3 *Results and discussion*

Approximately 300 cylindrical test specimens were prepared from block samples brought from each site, using a 25.4mm hollow core bit, each core having an aspect ratio of two. The test specimens were carefully prepared to achieve satisfactory flatness and parallelism. The rate of loading of 0.5MPa per second was applied, using an advanced digital loading machine (EL31-3275/01 ELECT 2000).

For the quartz dolerites under investigation an average of four tests was taken for each result. The mean and range of the uniaxial compressive strength tests carried out on dry and saturated specimens from each site are listed in Table 3.3. The mode of failure was a combination of the different modes described earlier. The results range between $92MN/m^2$ and $196MN/m^2$ for dried samples and between $70MN/m^2$ and $187MN/m^2$ for saturated samples. The highest values were obtained from the Hillend quartz dolerites, which are the freshest and have the finest grain size of all of the studied quartz dolerites. Weathered and relatively coarse grained rocks gave lower strength values; for example Westcraigs quartz dolerites gave uniaxial compressive strength values of $92MN/m^2$ and $70MN/m^2$ for dried and saturated specimens respectively.

The amount of water absorbed by a rock affects the strength values of that rock, as can be seen in Table 3.3 where the strength results from saturated specimens are compared with those of oven dried ones and are seen to be lower. In Figure 3.8 the dry uniaxial compressive strength results are plotted against the dry point load index I_S values, and a best fit line is drawn which has a correlation coefficient of 0.973; high uniaxial compressive strength values are associated with high point load index values. It is calculated that the most appropriate conversion factor for converting I_S values to uniaxial compressive strengths for 51mm cores used in this study is between 22 and 24. Figure 3.9 shows that a good correlation exists between uniaxial compressive strength and the dynamic modulus of elasticity (E_d). The correlation coefficients of the best fit lines of the relationship between these two parameters are 0.95 and 0.94 for dry and saturated samples respectively.

Table 3.3 Mean and range of uniaxial compressive strengths of the quartz dolerites (MN/m^2).

Site		oven dried	saturated
Caldercruix	mean	180	168
	range	(172 - 184)	(160 - 173)
	no. of samples	(4)	(3)
Duntilland	mean	170	159
	range	(165 - 177)	(152 - 167)
	no. of samples	(5)	(5)
Westcraigs	mean	92	70
	range	(86 - 96)	(63 - 75)
	no. of samples	(5)	(5)
Cairneyhill	mean	190	185
	range	(185 - 197)	(175 - 189)
	no. of samples	(5)	(4)
Hillend	mean	196	187
	range	(189 - 201)	(180 - 192)
	no. of samples	(5)	(4)
Tam's Loup 1	mean	162	150
	range	(158 - 166)	(145 - 159)
	no. of samples	(4)	(4)
Tam's Loup 2	mean	120	95
	range	(114 - 124)	(88 - 100)
	no. of samples	(5)	(5)
Boards	mean	165	155
	range	(160 - 172)	(150 - 169)
	no. of samples	(5)	(4)

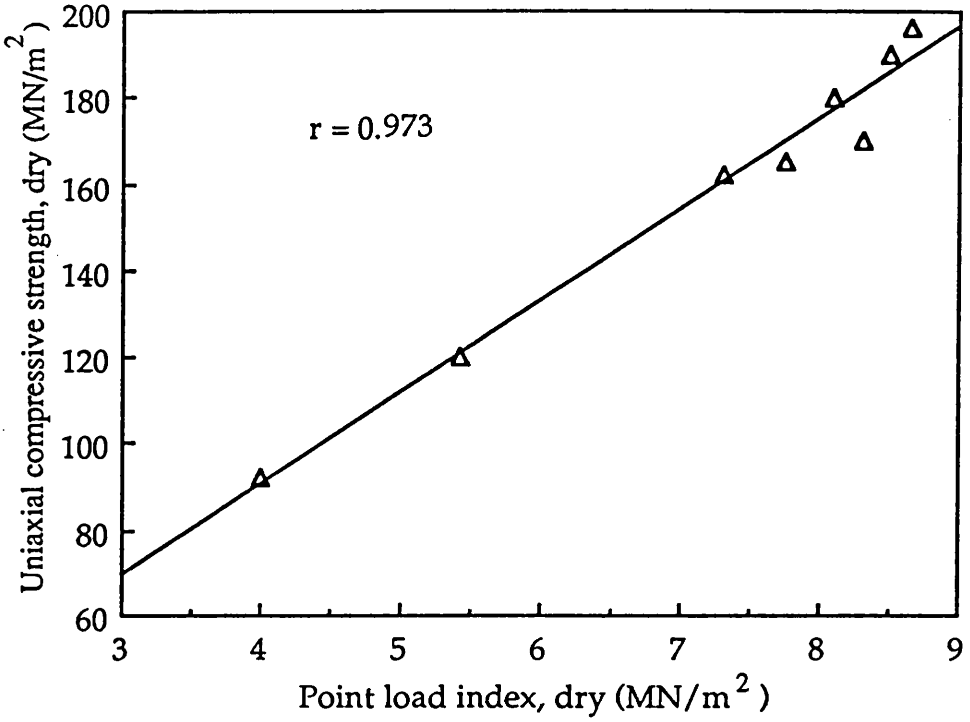


Fig. 3.8 Relationship between point load index (for 51mm diameter core) and uniaxial compressive strength of the quartz dolerites.

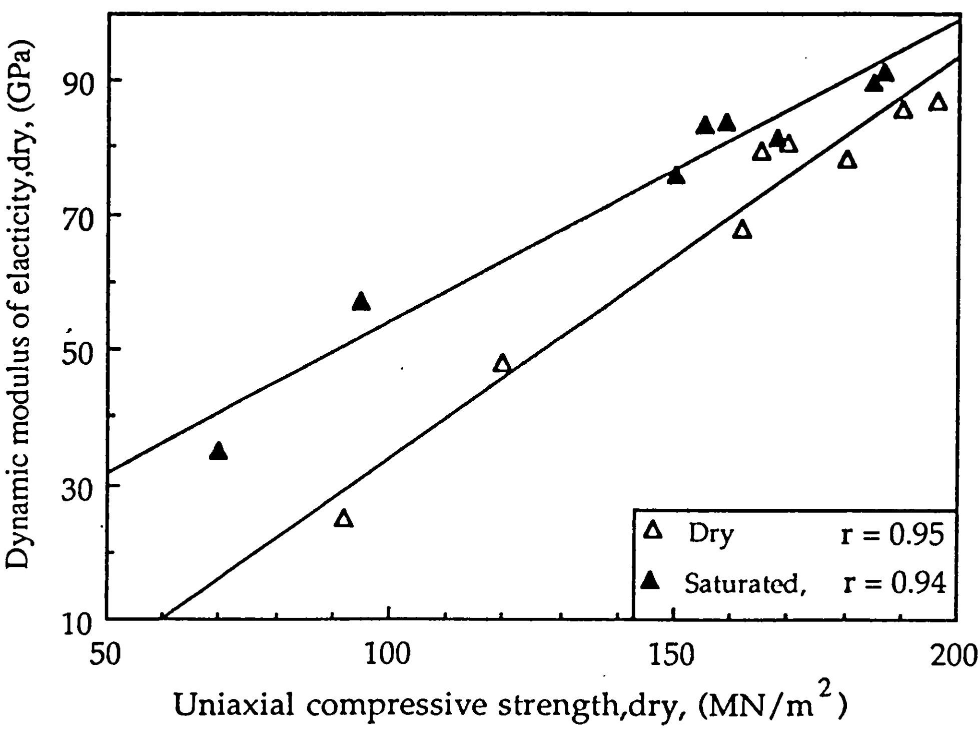


Fig. 3.9 Relationship between dynamic modulus of elasticity and uniaxial compressive strength of the quartz dolerites.

3.4 Stress-strain relationship

3.4.1 *Introduction*

The response of rocks to uniaxial stress is controlled by rock properties and test conditions. Deformability of rocks means the capacity of rocks to strain under applied load or in response to unloading in excavation. Three modes of deformation are known to appear in rocks under stress namely; quasi-elastic, semi-elastic, and non-elastic. Strong rocks show quasi-elastic behaviour almost until fracture occurs and the stress-strain relationship is almost linear throughout. Semi-elastic behaviour is characteristic of weaker rocks and in this regime the stress-strain ratio becomes constant after the initial concavity of the stress-strain curve. In the third category the stress is not recoverable after the initial stage of loading.

<u>Static elastic moduli</u>

A more adequate description of the reaction of intact rock materials to uniaxial compression can be obtained by considering their axial or volumetric deformation characteristics, both short term (elastic) and long term (creep or fatigue) properties. The term elastic applies when a rock obeys Hooke's law, which states that "stress is directly proportional to strain". This means that the stress-strain relationship is linear and the strain energy input is recoverable.

The modulus of elasticity (Young's modulus) is the most commonly determined elastic constant used to obtain information about the deformability of a rock which is basic to design details. It defines elastic normal strain in a body by the following relationship:

$$E = \sigma n / \varepsilon \qquad (3.5)$$

Where:

E = modulus of elasticity (Young's modulus)

σn = normal stress

ε = axial strain

Different rates of stress application will result in a different value of E for the same material. Under constant test conditions, very strong rocks display a linear relationship and the strain is recoverable. The weaker rocks, of low strength, have a non-linear stress-strain relationship with an initial concavity zone due to closing of micro cracks or pore spaces, and a terminal plastic zone approaching failure point. There is a linear zone in between these two zones, where the rock behaves approximately in an elastic manner (Attewell and Farmer 1976). The slope of this linear zone is the average modulus of elasticity (E_{av}) of the rock. The slope of a line drawn from the origin to a point on the curve (usually at failure or 50 per cent failure) is called the secant modulus (E_{s50}) and the slope of a line drawn tangentially to the stress-strain curve (at 50 per cent of ultimate strength) is considered to be the tangent modulus of elasticity (E_{t50}), (see Fig. 3.10).

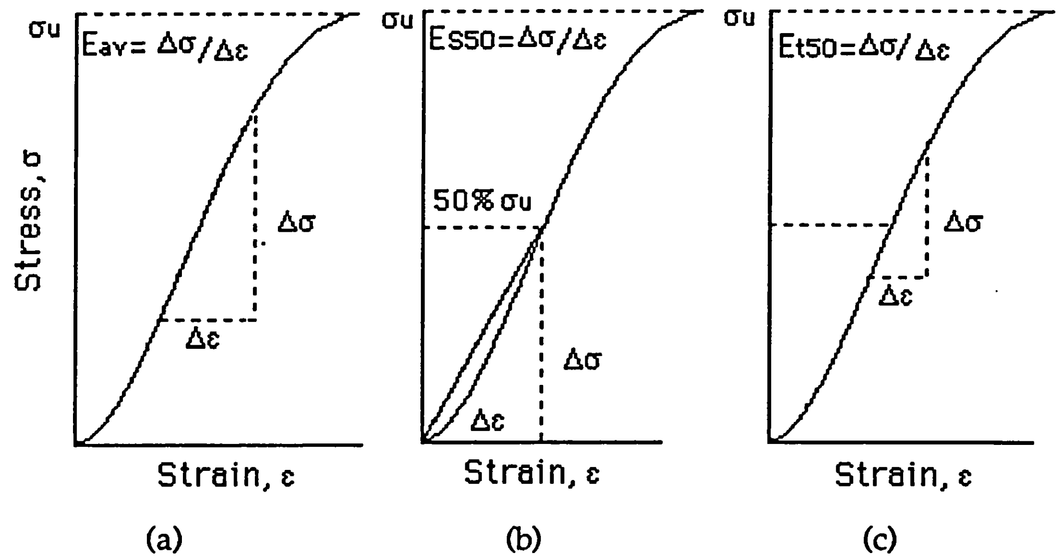


Fig. 3.10 Determination of modulus of elasticity from axial stress strain diagram. (a) Average modulus, E_{av}, (b) Secant modulus at 50% of ultimate strength of σ_u, E_{s50}, (c) Tangent modulus at 50% of ultimate strength, E_{t50}. (after Johnson and DeGraff 1988).

The same rock property factors that control intact rock compressive strength influence the modulus of elasticity. The relationship between compressive strength and modulus of elasticity is direct and linear as determined by many workers (Judd and Huber 1962; D'Andrea *et al.* 1965; Deere and Miller 1966 and Kulhawy 1975), with a correlation coefficient of 0.72 reported by D'Andrea *et al.* (1965).

Lateral or transverse strain, which occurs with axial contraction and elongation is represented by the dimensionless Poisson's ratio. It is obtained from the following equation:

$$\nu = \Delta d \,/\, \Delta l \qquad (3.6)$$

Where:

ν = Poisson's ratio

Δd = change in diameter

Δl = change in length

Poisson's ratio cannot exceed 0.5, a value obtained from an ideal compressible material. Thus all natural materials have Poisson values less than 0.5.

The shear or rigidity modulus, *Gs*, is a measure of the shearing strain resulting from shear stress on a plane. The shear or rigidity modulus *Gs* is obtained from the following relationship:

$$Gs = \tau / \gamma \tag{3.7}$$

Where:

Gs = shear modulus

τ = shearing stress

γ = shearing strain

Volumetric strain or dilation, which occurs when a body is subjected to hydrostatic stress, is called the bulk or incompressibility modulus, *K*. The value of *K* is obtained by the following:

$$K = \sigma o / \varepsilon v \tag{3.8}$$

Where:

K = modulus of incompressibility

σo = hydrostatic pressure

εv = volumetric strain

The various elastic constants mentioned above have an important role in many aspects of engineering geology. The response of natural materials to construction and in-service deformation resulting from structures such as tunnels, dams and buildings are examples of applications.

Dynamic elastic moduli

As well as determining moduli of elasticity by static methods in a laboratory, they may also be obtained by dynamic methods. There are two methods to achieve this (ISRM 1978), based on either the theory of vibrations, the 'resonance' method, or on the theory of wave propagation, the 'pulse' method. Properties of materials measured by dynamic methods are referred to as dynamic properties. Duncan (1969) used sonic velocity values to calculate the dynamic modulus of deformation using the equation:

$$E_d = f\, Vp^2 / g \qquad (3.9)$$

where:

E_d = dynamic modulus of elasticity

f = bulk density of the rock material

Vp = P-wave velocity in the rock material

g = acceleration due to gravity.

Typically, the dynamic modulus of elasticity (E_d) is greater than the static one (E_s), because the response of the specimen to the very short duration strain and low stress level is essentially purely elastic (Clark 1966). The dynamic shear modulus, G_{sd}, exhibits a similar relationship, that is, shear waves are measured at very low shear strain, resulting in G_{sd} being greater than G_s.

3.4.2 *Results and discussion*

Strain measurements were carried out on specimens which were, at the same time, undergoing uniaxial compressive strength tests. To minimize any bending effects the results were taken by averaging readings on 3 electrical strain gauges mounted every 120° on the circumference, parallel to the longitudinal axis. The electrical strain gauges were attached mid way on the cores to eliminate any end effects. Seven to ten readings of load and strain were taken at evenly spaced load increments. While most engineering tests could be carried out under both dry and saturated conditions, static modulus of elasticity was measured only in the dry condition. The results obtained are plotted on Figure 3.11. The static moduli of elasticity, E_s, were computed from the stress-strain diagrams produced, as the secant modulus at 50% of the ultimate stress level, and the dynamic moduli of elasticity were determined by using equation 3.9, which makes use of the P-wave velocity and density measurements.

Table 3.4 shows the mean and range of the static and dynamic modulus of elasticity determined in this study. In all cases, values of the static moduli of elasticity thus derived were found to be less than the dynamic moduli computed from the sonic characteristics. The average ratio of E_d / E_s was found to be 1.34.

The stress strain analyses revealed that stronger rocks have a higher modulus of elasticity. Beyond the initial stages of loading all rocks tested behaved as semi-elastic materials.

The static moduli, E_s, were then plotted against the dynamic moduli, E_d. Figure 3.12 indicates that a direct linear relationship between the two moduli may be expressed by the following equation:

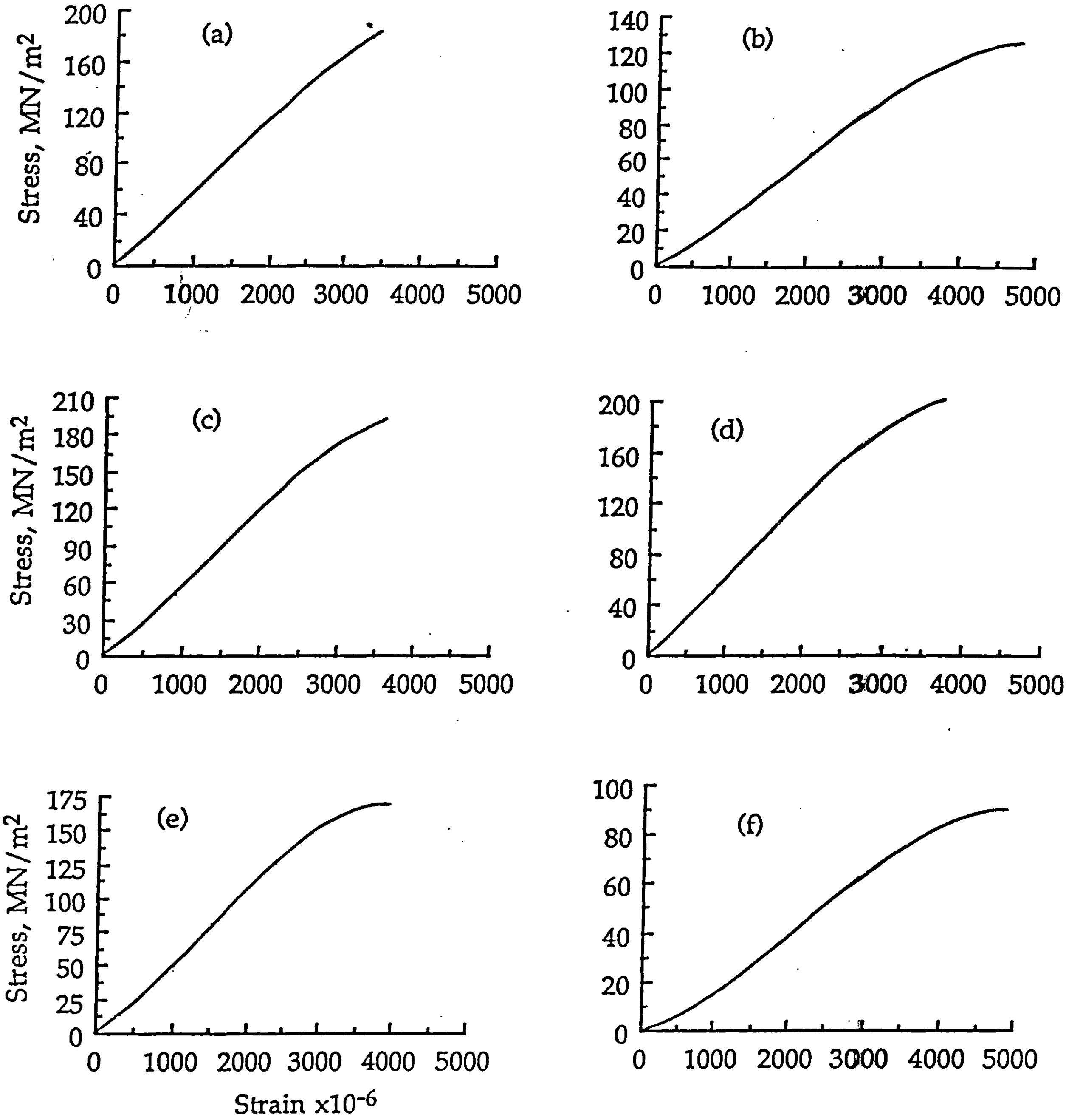
$$E_s\,\mathrm{d} = 4.6 + 0.67\,E_d\,\mathrm{d} \qquad r = 0.96 \qquad (3.10)$$

where E_s d = dry static modulus of elasticity
E_d d = dry dynamic modulus of elasticity

According to Attewell and Farmer's classification (1976) of intact rocks on the basis of their static modulus of elasticity, the quartz dolerites from all sites are semi-elastic; dolerites from Caldercruix, Duntilland, Cairneyhill, Hillend, Tam's Loup 1 and Boards are stiff, and dolerites from Westcraigs and Tam's Loup 2 are medium stiff.

Table 3.4 Mean and range values of the static (E_s) and dynamic (E_d) modulus of elasticity measurements of the quartz dolerites (GPa).

Site		$E_s d$	E_d d
Caldercruix	mean	57	78.2
	range	(56 - 59)	(77 - 80)
	no. of samples	(3)	(4)
Duntilland	mean	-	80.45
	range	-	(78 - 82)
	no. of samples	-	(5)
Westcraigs	mean	20.2	25.0
	range	(18 - 22)	(23 - 28)
	no. of samples	(3)	(5)
Cairneyhill	mean	60.1	85.0
	range	(57 - 62)	(84 - 87)
	no. of samples	(3)	(5)
Hillend	mean	61	87.10
	range	(58 - 63)	(85 - 90)
	no. of samples	(3)	(5)
Tam's Loup 1	mean	54	67.75
	range	(52 - 57)	(63 - 70)
	no. of samples	(4)	(5)
Tam's Loup 2	mean	36.2	48.0
	range	(35- 38)	(45 - 50)
	no. of samples	(3)	(4)
Boards	mean	56	79.38
	range	(54 - 59)	(77 - 83)
	no. of samples	(3)	(5)



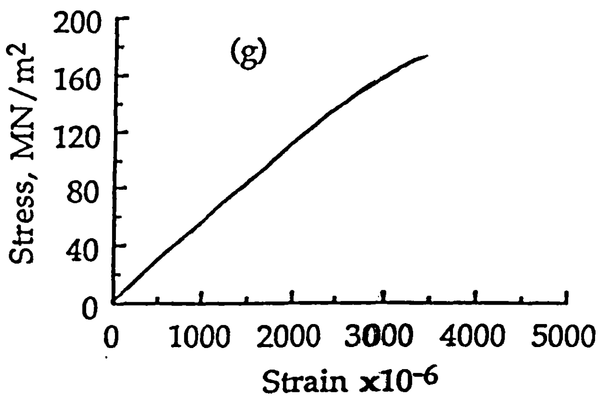


Fig. 3.11 Stress-strain relationship of the quartz dolerites, (a) Caldercruix, (b) Duntilland, (c) Cairneyhill, (d) Hillend, (e) Tam's Loup 1, (f) Tam's Loup 2, and (g) Boards.

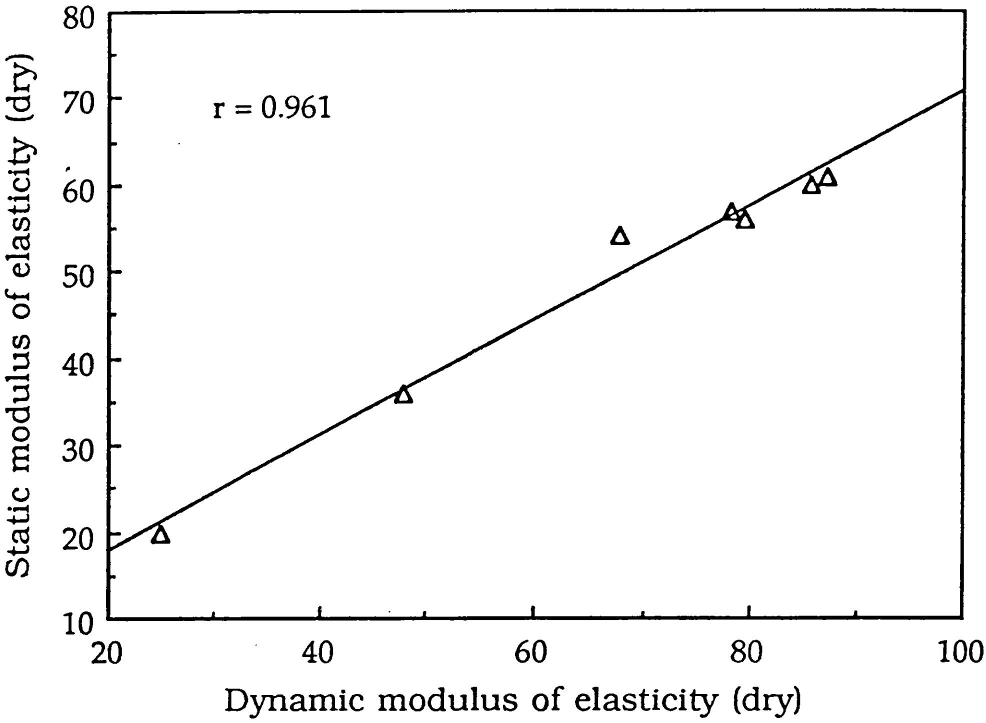


Fig. 3.12 Graph showing the relationship between dynamic and static moduli of elasticity of the quartz dolerites.

3.5 Point load index test

3.5.1 *Introduction*

The point load test is a much simpler field and laboratory test, which gives a result that can be related quite closely to uniaxial compressive strength results. This test has been described in detail by Broch and Franklin (1972). The apparatus comprises a small hydraulic ram mounted in a loading frame. The ram compresses a core (or lump) of rock across its diameter between two standard pointed platens (plate 3.1a). The test results are expressed in terms of a point load strength index I_s, using the following equation:

$$I_s = P / D^2 \tag{3.11}$$

where:

P = the force required to cause failure

D = the distance between loading points.

Broch and Franklin (1972) proposed the point load test as a replacement for the uniaxial compression test, and stated that it was preferred for the following reasons:

(1) Smaller forces are needed so that a small and portable testing machine can be used.

(2) Specimens in the form of core or irregular lumps can be used and require no machining.

(3) Results show less scatter than those for the uniaxial compression test.

(4) Fragile or broken material can be tested.

(5) Measurement of strength anisotropy is simplified.

Bieniawski (1975), however, disagreed with numbers 3 and 4, and showed that the point load test produced higher standard deviations than the uniaxial compression test. He suggested that an acceptable point load test should not give an average standard deviation of more than 15 per cent. And he mentioned that the weakest rock material that can be tested in the point load test should not possess a uniaxial compressive strength of less than 25 MN/m^2.

Fookes *et al.* (1971) used the point load test on irregular shaped specimens to study the mechanical properties of weathered rocks. Since rock cores were not available and regularly shaped specimens could not be produced from such weathered rocks. However, the results obtained from tests on irregularly shaped specimens are much more variable than those obtained from cores.

Bieniawski (1975) studied the shape and size effect on the point load test and showed that the irregular lump test is less accurate than the diametrical and axial tests, and concluded that the diametrical test is the more convenient and simple to use. He also showed that an increase in specimen diameter caused a decrease in point load strength. The internationally recommended core size for site investigation drilling is 54mm (NX), and Broch and Franklin (1972) recommend 50mm as a reference core diameter to be used in their proposed size correlation chart for the point load index. Bieniawski (1975) suggested that core diameter should not be less than 42mm.

The relationship between point load test and uniaxial compressive strength has been studied by many authors, and found to be a linear relationship. Bieniawski (1975) proposed a conversion factor of 25 to calculate uniaxial compressive strength from the point load index on specimens of 54mm diameter, whereas Attewell and Farmer (1976), who used data from Franklin *et al.* (1971) and D'Andrea *et al.* (1965) proposed a conversion factor of 16. Bastekin (1985) carried out point load tests on different core diameters and plotted the results against their uniaxial compressive strength values. Bastekin (1985) stated that the most appropriate conversion factor is: 16 - 22 for 38mm diameter cores; 22 - 29 for 50mm diameter cores and 29 - 34 for 76mm diameter cores. Bastekin (1985) concluded that the conversion factor of 25 proposed by Bieniawski (1975) is approximately the mean value of all conversion factors recommended by Bastekin (1985).

3.5.2 *Results and discussion*

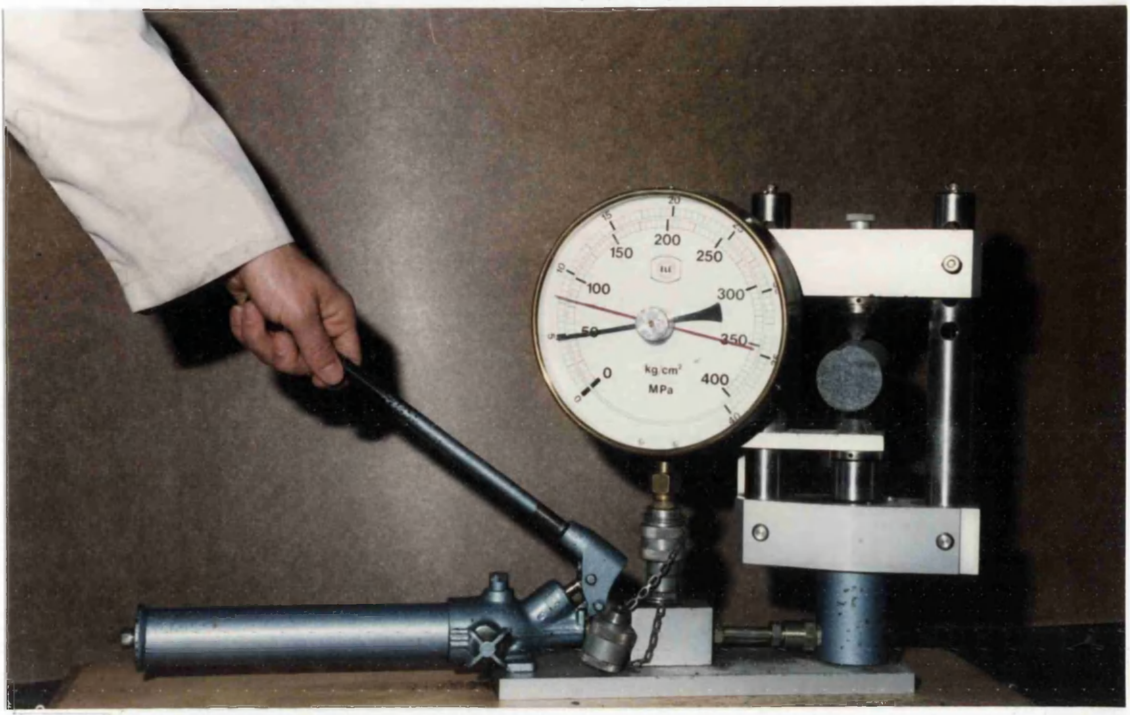
The testing was carried out in accordance with suggestions from the Commission on Standardization of laboratory and field tests of the ISRM (1973) on the diametral test on cores. The portable (ELE) point load tester (Plate 3.1a) was used for testing cores in both oven dry (24h at 105 ± 5 °C) and saturated (48h immersion in water at atmospheric pressure) conditions. Bastekin (1985) mentioned that testing different core sizes and correlating these with uniaxial compressive strength is not considered worthwhile unless 50mm cores cannot be produced from the rock undergoing testing. Only one core size was used in this study, and cores of 51mm (2in bit size) with an aspect ratio (length/diameter) of 2 were prepared from core or block samples from each site. This size of core is preferred by most research workers using this particular test.

The point load strength index is calculated using equation 3.11. The mean and range of the results for diametral tests carried out on dry and saturated cores are given in Table 3.5. This Table shows that the results of the point load test carried out on rock samples from the different sites used in this study are similar, except for weathered samples from Westcraigs and Tam's Loup 2 which gave the lowest values; and the standard deviation of samples from each site is small. This supports the views of Broch and Franklin (1972) about the repeatability of the test which they considered as one of its advantages. In Figure 3.13 the point load index values are plotted against the P-wave velocities of the same samples, and high index values are shown to be associated with high P-wave values, suggesting that the behaviour of rocks in one test can predict what their behaviour will be in the other test. Once again the difference between dry and saturated results is less in strong quartz dolerites than that in weak quartz dolerites. Other correlations between the point load index strength and other rock properties are mentioned in section 3.4, 3.6 and in chapter 5.

Plate 3.1

a.The (ELE) point load tester

b. The NR Schmidt hammer tester



-a-

-b-

Table 3.5 Mean and range value of point load index results of the quartz dolerites (MN/m^2).

Site		saturated	oven dried
Caldercruix	mean	7.56	8.1
	range	(7.0 - 8.11)	(7.7 - 8.5)
	no. of samples	(4)	(3)
Duntilland	mean	7.00	8.30
	range	(6.7 - 7.3)	(7.9 - 8.5)
	no. of samples	(5)	(4)
Westcraigs	mean	2.5	4.00
	range	(2.0 - 2.9)	(3.25 - 4.36)
	no. of samples	(5)	(5)
Cairneyhill	mean	8.00	8.5
	range	(7.7 - 9.4)	(8.2 - 9.0)
	no. of samples	(5)	(3)
Hillend	mean	8.20	8.65
	range	(7.8 - 8.4)	(8.32 - 9.0)
	no. of samples	(5)	(4)
Tam's Loup 1	mean	6.35	7.3
	range	(6.0 - 7.0)	(6.9 - 7.6)
	no. of samples	(5)	(5)
Tam's Loup 2	mean	3.61	5.42
	range	(3.0 - 4.0)	(4.9 - 5.7)
	no. of samples	(4)	(4)
Boards	mean	6.78	7.75
	range	(5.9 - 7.0)	(7.34 - 8.0)
	no. of samples	(5)	(4)

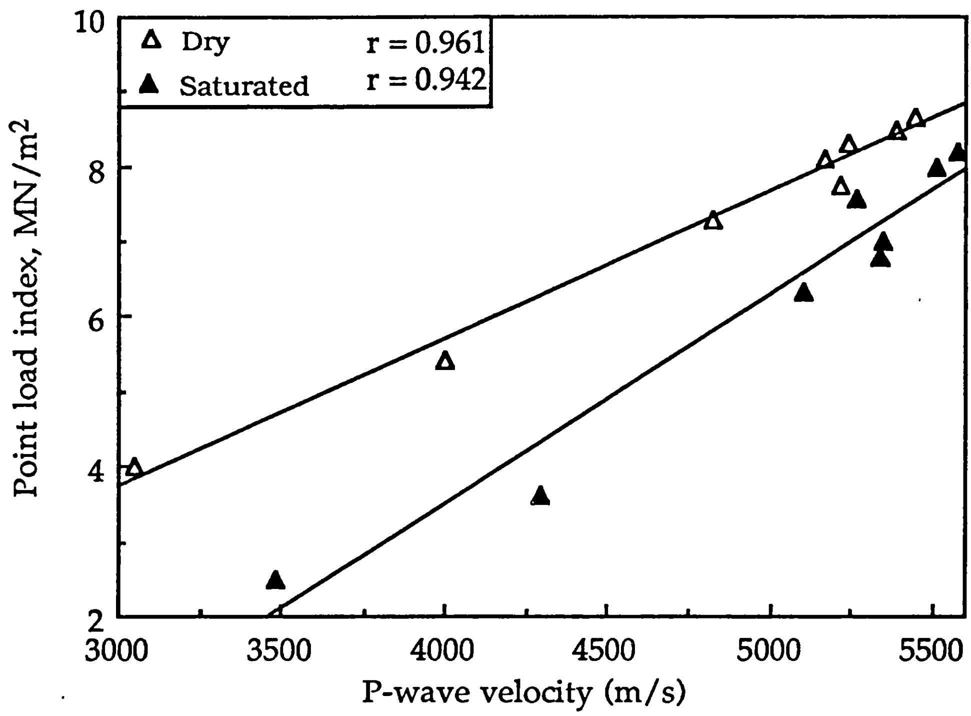


Fig. 3.13 Graph showing the relationship between point load index strength and P-wave velocity.

3.6 Schmidt hammer rebound number test

3.6.1 *Introduction*

The Schmidt hammer is a portable device originally developed by Schmidt (1951) for the non-destructive testing of concrete. Since then it has been applied in assessing the strength of rocks.

The Schmidt hammer consists of a large, narrow solid steel piston (the hammer) held inside a steel cylinder (plate 3.1b). A spring allows the hammer to be primed inside the cylinder when it is pressed against a level spot on a rock surface with the hammer held at right angles to the surface; and a catch holds the hammer in position. When the piston is fully primed the catch is released and the hammer rebounds from the rock surface to a height recorded either by a simple needle or by a graph recorder attached to the hammer.

The reading, known as the rebound number or 'Rn', depends on the elasticity of the rock under testing and the hardness of its minerals. Poole and Farmer (1980) recommended that the maximum value from at least 5 continuous impacts at a point should be selected, because rebound values have a tendency to rise and to show considerable variation during the first 3 or 4 impacts at a point. The results of such tests can be correlated with either the uniaxial compressive strength or the point load index, or other rock properties such as sonic velocity and water content. Attewell and Farmer (1976), Carter and Sneddon (1977), Irfan and Dearman (1978), Al-Jassar and Hawkins (1979) and others have plotted the Schmidt rebound number against uniaxial compressive strength and demonstrated that a good relationship exists.

It has been suggested (Geol. Soc. Rep. 1977) that the uniaxial compressive strength of a rock can be estimated by multiplying its Schmidt hammer rebound number by the dry unit weight of the rock specimen. In general, however, it is estimated (Geol. Soc. Rep. 1977) that there is only a 75% probability that the laboratory determined uniaxial strength will lie within 50% of the strength derived from correlation charts prepared by Deere and Miller (1966). Thus, the Schmidt hammer test is considered to be less accurate in predicting rock strength than the point load test (Johnson and DeGraff 1988). Carter and Sneddon (1977) examined the relationship between rebound numbers and the point load index and produced a straight line relationship in a log-normal graph. The Commission on Testing Methods of the International Society for Rock Mechanics (ISRM 1978) included Schmidt hammer testing in their standard techniques of laboratory and field testing.

Factors influencing the Schmidt hammer test include:

(1) The surface of the specimen being tested should be flat and homogeneous over the chosen spot.
(2) Impacts should not to be applied on the same spot more than once.
(3) The water content of the tested rock should be taken into consideration since saturated rocks give lower results.
(4) The rock being tested must be free of discontinuities within 60mm of the impact surface.

3.6.2 *Results and discussion*

Before testing the rocks it was often necessary to make the surface sufficiently smooth to take readings with the NR type Schmidt hammer (impact energy 0.225mkg) (plate 3.1b). In such a situation the rock surface was rubbed with a carborundum stone to make it smooth and then cleared of dust before testing with the Schmidt hammer. At least 20 readings of rebound numbers were taken at each station. However, for calculation purposes, the lowest 5 readings were discarded and the average rebound was calculated on the basis of the 15 highest readings. The laboratory test was carried out either on cores or large blocks of quartz dolerite. The mean and range values of the rebound numbers at each site are listed in Table 3.6, which shows that the field results are higher than the laboratory ones.

Figure 3.14 shows that the field and laboratory results are in agreement, in categorising tested rocks, since high field results are associated with high laboratory results.

In Figure 3.15 Schmidt hammer rebound number values are plotted against point load indices (laboratory test data only have been used) and a reasonable relationship is obtained. High rebound number values are associated with high point load strength index values. The best fit line of this relationship has a correlation coefficient of 0.91. This indicates that the point load strength index values can be predicted from the rebound number.

Table 3.6 Mean and range values of Schmidt hammer rebound numbers of the quartz dolerites.

Site		field	laboratory
Caldercruix	mean	-	55
	range	-	(50 - 58)
	no. of samples		(4)
Duntilland	mean	60	52
	range	(55 - 64)	(50 - 55)
	no. of samples	(5)	(5)
Westcraigs	mean	40	35
	range	(36 - 44)	(30 - 37)
		(5)	(5)
Cairneyhill	mean	64	58
	range	(59 - 66)	(55 - 60)
	no. of samples	(5)	(5)
Hillend	mean	62	58
	range	(58 - 65)	(54 - 60)
	no. of samples	(5)	(5)
Tam's Loup 1	mean	58	55
	range	(55 - 62)	(52 - 57)
	no. of samples	(5)	(5)
Tam's Loup 2	mean	45	41
	range	(42 - 47)	(37 - 43)
	no. of samples	(4)	(4)
Boards	mean	60	57
	range	(55 - 64)	(52 - 59)
	no. of samples	(5)	(5)

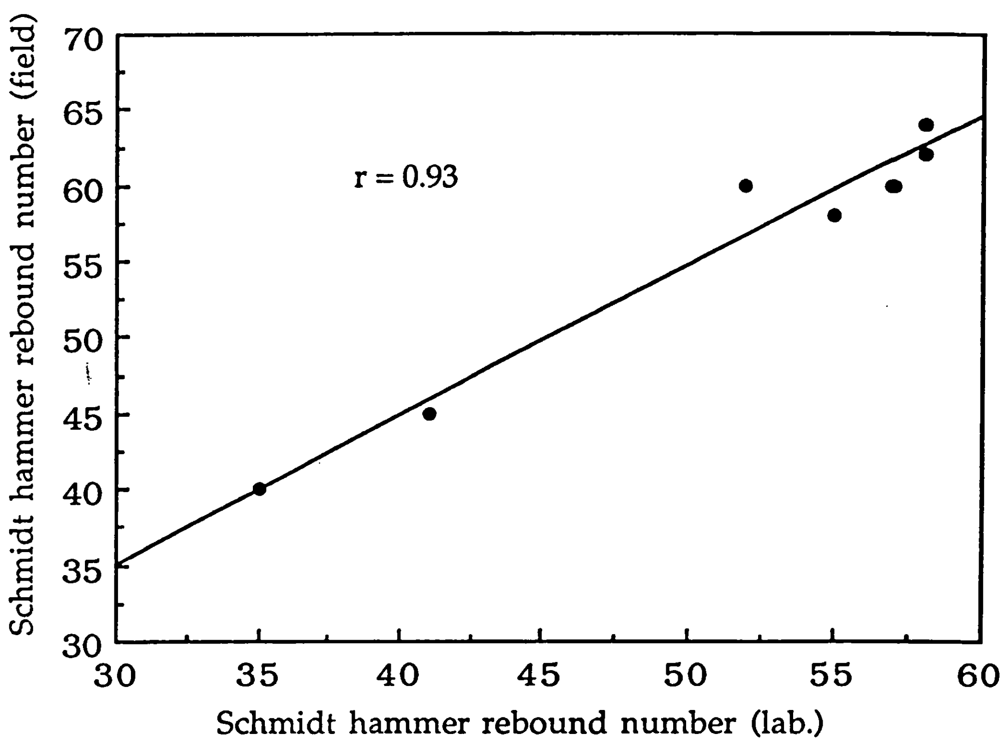


Fig. 3.14 Schmidt hammer rebound numbers in field and laboratory of the quartz dolerites are plotted against each other.

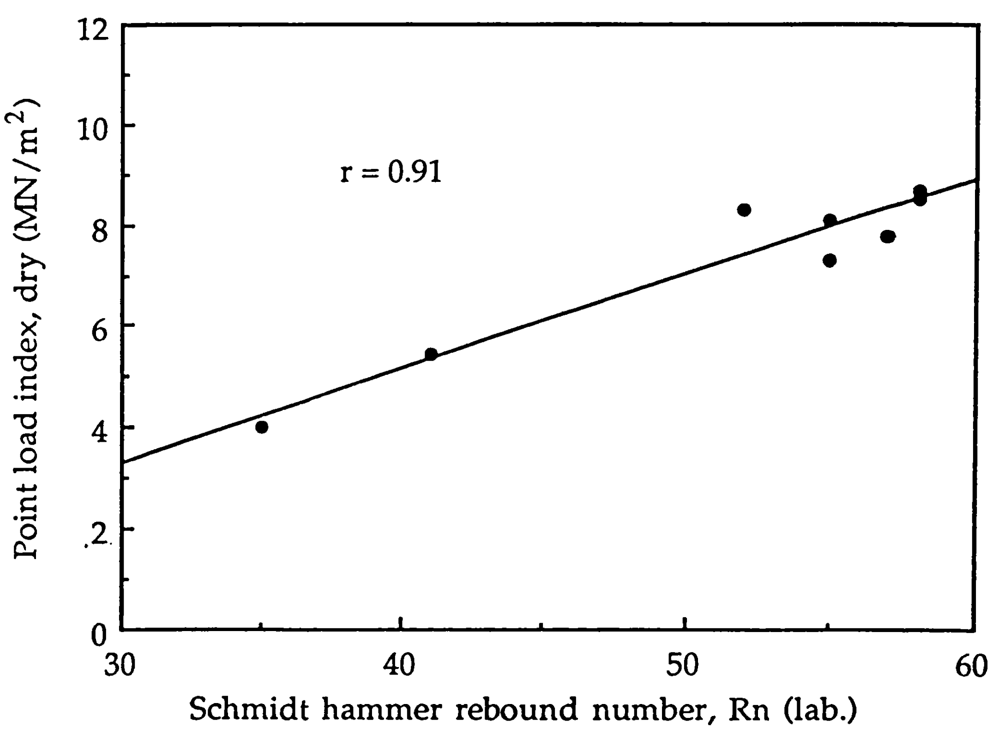


Fig. 3.15 Graph showing the relationship between rebound number (Rn) and point load index (Is) of the quartz dolerites.

CHAPTER FOUR

QUARTZ DOLERITES AGGREGATES

4.1 Introduction

4.2 Density and water absorption

4.3 Aggregate impact value test

4.4 Aggregate crushing value test

4.5 10% fines value test

4.6 Aggregate abrasion value test

4.7 Los Angeles abrasion value test

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4.9 Soundness

4.10 Quartz dolerites aggregates used in concrete

4.11 Adherence to bitumen

QUARTZ DOLERITE AGGREGATES

4.1 Introduction

Aggregates are used for a variety of purposes, the two main ones being for roadstone and concrete works. They normally comprise up to 60% of the volume of Portland and asphalt cement concrete mixes, and therefore it is not surprising that the quality of aggregates is of considerable importance in determining their suitability for any specific engineering application. The requirements of an aggregate in engineering construction depend upon the type of construction, the pavement layer, the traffic and climatic conditions. Polishing resistance is clearly unimportant for a base, and frost susceptibility is of no importance in the tropics.

In a typical high grade modern flexible pavement the lowest layer is the *sub-base*, which serves the dual purposes of distributing the load of passing vehicles over the underlying soil or rock; the *sub-grade*, and of providing a "working platform" from which to construct the main load distributing layer, which is the *road-base*, then comes the surfacing which is divided into two layers; the *base-course* and the *wearing-coarse* (see Fig. 4.1). The requirements for the aggregate to be used in each layer vary accordingly.

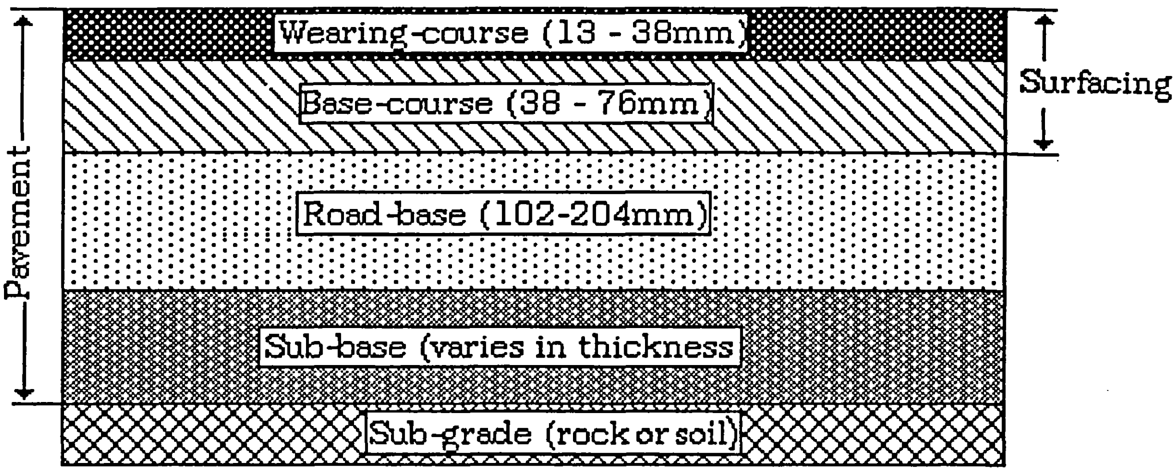


Fig. 4.1 Cross-section of a typical highway. (after Harris, 1977)

The quartz dolerite intrusions provide the most reliable source of good construction materials in the Midland Valley of Scotland. They are used extensively for road metal, concrete aggregate and kerbstone. Since the rocks under consideration in this study have been widely used as aggregate and quarried in many localities, their aggregate properties are examined in this chapter.

The standard aggregate testing procedures described in British Standards 812 and others were employed. A variety of tests examine aggregate strength particularly the aggregate impact value and aggregate crushing value, aggregate abrasion value and polished stone value and aggregate soundness. Some other tests which are not included in the British Standards have also been employed (e.g., the Los Angeles abrasion test). The tests were carried out in the aggregate testing laboratories of the Geology and Applied Geology Department of Glasgow University. When crushing the rock material in order to obtain the appropriate aggregate sizes, it was noticed that the fresh quartz dolerites produced higher elongation and flakiness indices whereas the altered quartz dolerites generate much better shape but also a greater percentage of fines.

4.2 Relative density and water absorption

4.2.1 *Introduction*

The unit weight of a rock depends on the density of its constituents; its porosity; and the amount of water in its pores. Reported density determinations show that values varying between 2.4 and 2.9 appear to be strongly dependent upon the iron and magnesium rich mineral content of igneous and metamorphic rocks, and the accumulation of such minerals in sedimentary rocks. Porosity has an inverse effect on rock density; that is, in general, the higher the porosity the lower the density.

Several methods of measuring the relative density of aggregates are used which make use of the weight of the sample in air and in water. A wire basket, a gas jar or a pycnometer are employed to contain the sample; the choice being governed by the grain size of the sample concerned. The relative density of a sample is usually expressed either on an oven dried basis, or on a saturated and surface dried basis, or as apparent density. Relative density on an oven dried basis is normally given for road aggregates.

Bulk density (either compacted or uncompacted) is measured by weighing a sample of aggregate which has been placed in a specified container. The result is expressed in kg/m^3. Bulk density is very important in concrete mix design, particularly for light weight aggregates. It enables concrete mixes specified by volume to be converted into gravimetric proportions, and thus, enables batch masses to be determined (Collis and Fox 1985).

Water absorption value is expressed as a percentage of the ratio of the weight of water absorbed by a sample to the weight of the same sample after being oven dried (for 24h at 105±5 °C), and normally obtained at the same time as the density. Water absorption is an indirect measure of the porosity of an aggregate, which in turn, can relate to other physical characteristics such as mechanical strength, shrinkage, soundness and to its general durability potential (Collis and Fox 1985). Water absorption is important in the mix design of concrete and may also give indications as to the frost susceptibility of aggregates.

A reliable classification of water absorption values for United Kingdom natural aggregates is not available, although BS 1201 includes a recommendation that the aggregate absorption should not be greater than 3 per cent.

4.2.2 *Results and discussion*

Relative density and water absorption measurements in this study have been carried out according to the BS 812:1975. Relative density results are based on three different conditions.

1- Relative density based on oven dry base = D/A-C

2- Relative density based on saturated base = A/A-C

3- Apparent density = D/D-C

where D = weight of sample after 24h in oven

A = weight of sample after 24h in water

C = weight of sample suspended in water

Table 4.1 gives the range and the mean values of relative density and water absorption of all the samples tested at each site.

The results from all sites show very little variation, except for weathered samples, either in mean values or in the range of values obtained. This suggests that the rocks are remarkably similar in mineralogy, texture and porosity, the last is confirmed by the close similarity of water absorption values (1.24 - 1.58) for fresh materials. Small variations do exist, however, especially within individual dolerite intrusions. For example, the Caldercruix dolerites show density increasing in the lower half of the bore hole. This is due to an increase in the ore and pyroxene contents (see Chapter 2), perhaps coupled to a decrease in grain size corresponding to a decrease in porosity and therefore water absorption. The water absorption values of the quartz dolerites studied seems to be related to the amount of alteration and weathering that each sample has undergone, since weathered samples (Westcraigs, Tam's Loup 2) usually possess relatively high water absorption values (1.72 - 2.1).

Gribble (1990) plotted water absorption against mechanical properties (AIV, ACV, 10% fines) of the Kelvin gravel and used the recommended value of water absorption (ICE 1976) in aggregates of 3%, and found that it correlated with 10% fines value of 175kN. He concluded that if a deposit has a water absorption value of 3% or less, then all other engineering properties will fall within the BS specifications. The above results show that all rocks have water absorption values lower than 3%, and thus, should have acceptable test values in every other test carried out.

Water absorption values obtained in this study are compared with strength values; aggregate impact, crushing and 10 per cent fines values, Figures 4.2, 4.7 and 4.14. These Figures show that the comparatively finer and fresher rocks (e.g., Hillend rocks) possess lower water absorption values than coarser and weathered rocks (e.g., Westcraigs). The Figures also show that the water absorption values obtained for the quartz dolerites studied correlate well with mechanical properties such as AIV, ACV and 10% fines; high water absorption values are associated with low strength values, i.e., low AIV, ACV, and high 10% fines values.

Table 4.1 Mean and range values of the relative density and water absorption measurements on quartz dolerite aggregates from each site.

Site		Density			Water abs.%
		OD	SSD	AD	
Caldercruix	mean	2.88	2.92	2.94	1.36
	range	(2.87-2.90)	(2.90-2.94)	(2.92-2.95)	(1.3-1.5)
	no. of samples	(4)			
Duntilland	mean	2.88	2.91	2.94	1.46
	range	(2.86-2.90)	(2.89-2.92)	(2.92-2.96)	(1.4-1.6)
	no. of samples	(5)			
Westcraigs	mean	2.73	2.79	2.84	2.1
	range	(2.70-2.76)	(2.77-2.80)	(2.79-2.85)	(1.8-2.5)
	no. of samples	(5)			
Cairneyhill	mean	2.90	2.93	2.97	1.3
	range	(2.89-2.92)	(2.90-2.94)	(2.94-2.99)	(1.1-1.4)
	no. of samples	(5)			
Hillend	mean	2.91	2.93	2.97	1.24
	range	(2.89-2.93)	(2.91-2.94)	(2.95-3.0)	(1.0-1.3)
	no. of samples	(5)			
Tam's Loup 1	mean	2.85	2.89	2.93	1.58
	range	(2.83-2.86)	(2.87-2.91)	(2.90-2.94)	(1.5-1.7)
	no. of samples	(5)			
Tam's Loup 2	mean	2.75	2.79	2.85	1.75
	range	(2.72-2.77)	(2.75-2.82)	(2.81-2.86)	(1.6-1.9)
	no. of samples	(4)			
Boards	mean	2.83	2.87	2.92	1.35
	range	(2.81-2.84)	(2.85-2.89)	(2.89-2.94)	(1.1-1.4)
	no. of samples	(5)			

OD = Oven dried, SSD = Saturated and surface dried and AD = Apparent density

4.3 Aggregate impact test

4.3.1 *Introduction*

This test gives a relative measure of the resistance of an aggregate to pulverisation as a result of a sudden impact. In this test a prepared test sample (14 - 10mm), representing a typical grading in the wearing course of roads, is placed in a cylindrical steel cup with an internal diameter of 102mm and an internal depth of 50mm. The sample is subjected to a total of 15 blows from 13.5±0.5kg hammer, falling freely from a height of 380±5mm above the sample. The amount of fine material produced (< 2.36mm) is calculated as a percentage of the initial sample weight, and is expressed as the aggregate impact value (AIV). An average of two tests for each sample is taken as the result. Low AIV values are an indication of tough aggregates, and thus, the lower the value the higher the aggregate quality. This test, combined with the aggregate crushing value test (page 95), are two standard tests recommended by the British Standard Institute (BS 812:1975) to assess the strength of road aggregate under different conditions of loading, namely repeated impact loading or continuous loading in the AIV and ACV respectively.

Ramsay (1965) in his investigation of the aggregate impact test stated that the choice of fines passing the 2.36mm BS sieve is an arbitrarily chosen value and does not take into account the percentage of original size range of aggregate retained on a 10mm sieve after the impact test, which he called "Aggregate Impact Value Residue". In dealing with igneous, metamorphic and fluvio-glacial gravel material, he concluded that the aggregate impact value residue (AIVR) is more sensitive to size and shape of the aggregates than the aggregate impact value (AIV). Higher AIVR values usually indicate resistance to comminution, and therefore a stronger aggregate.

A median fraction value (M) has been suggested by Ramsay *et al.* (1977) which represents the aggregate ranging between 2.36 and 10mm in size, and they recommended that the impact test results should state the three values (AIV, AIVR, and M) in order to explain the behaviour of rock material under the impact test. However, the consensus of opinion among aggregate researchers at present is that AIV and AIVR are important values; but that the M value does not serve any useful purpose.

4.3.2 *Factors affecting the AIV test.*

a. <u>Petrology and petrography of rock material</u>

Hosking (1967); Hosking and Tubey (1969); Williams and Lees (1970); Maupan (1970); Hartley (1974) among others, discussed this factor and showed that it is very important in terms of examining the aggregate strength. They indicated that the main petrographic factors which affect the impact test were mineral content, grain size, texture and degree of mineral alteration. The presence of hard and strong minerals affects the toughness of the rock and this can be assessed by comparison between calcareous rocks and fine grained igneous rocks. Although both groups of rocks exhibit an interlocking grain texture, they behave differently due to the hardness of their constituents.

Spence (1979) compared fine grained basic igneous rocks with their medium grained equivalents, and found that the former had a mean value of the petrographic constant (C), which is the AIV at 0% flakiness index value, of 9.49 with a range from 7.60 to 11.90, while the latter had a mean value of 10.62 with a range from 9.13 to 11.72. The results showed that the mean value of the AIV at 0% flakiness index value for fine grained basic igneous rocks was lower than that for the medium grained equivalents. However, the range of values for fine grained rocks was essentially similar to that exhibited by their medium grain equivalents, and he concluded that for basic igneous rocks, a small increase in grain size does not have a significant effect on the strength of the aggregate.

The bond between the mineral constituents of a rock and other factors such as texture, also have a significant effect on the strength of the rock. Igneous and non-schistose metamorphic rocks have an interlocking mineral texture with strong intergranular bonding forces holding the grains together and giving the characteristic toughness of these rock types. Chemical alteration can reduce the strength of a rock by altering the hard primary minerals to soft secondary ones (e.g., feldspars to clay minerals), and weathering can also affect the cohesion between the grains by weakening the intergranular bonds between the grains. Moore and Gribble (1980) studied the effect of alteration and weathering on rock aggregate strength of the Peterhead granite and concluded that when the Na_2O and CaO contents drop to less than the half of the original values, or when the oxidation ratio (Fe_2O_3/FeO) is increased to 150% of the original, the rock would not be suitable for use as aggregate in construction projects.

b. Aggregate shape and size.

The shape of aggregate is described in BS 812:1975 under four categories: cuboidal, elongate, flaky and flaky elongated. A flaky particle is one in which the smallest dimension is a maximum of 0.6 times the mean sieve size. An elongated particle is one whose maximum dimension is greater than 1.8 times its smallest dimension. The proportion of grains in each category of shape is influenced by the textural characteristic of the rock, and the method of crushing employed in the quarry. The typical range of index of flakiness for aggregates in normal production falls between 15 and 45, and is only exceptionally outside these limits.

The influence of aggregate shape on the aggregate impact value was demonstrated by Ramsay (1965). Ramsay *et al.* (1977) showed that the impact and crushing values increased with an increase in the flakiness index as follows:

$$\text{AIV or ACV} = C + nI_F$$

where C is the value of AIV or ACV at 0% flakiness value; and n is the coefficient of the flakiness index values, or alternatively, the gradient of the linear relationship between AIV or ACV and the flakiness index. Aggregate impact value residue (AIVR) is shown to be consistent and a more sensitive indicator of the influence of aggregate shape than the standard test. Attewell (1971) studied the effects of aggregate size on the aggregate impact and crushing tests, and showed that decreasing aggregate size resulted in increasing impact and crushing values (i.e., lower strength).

c. Procedural factors

Two procedural factors in the test were found (Ramsay *et al.* 1973) to affect the aggregate impact and residue values, and to influence the reproducibility of the results. These are 1) the rigidity of the apparatus, and 2) the nature of the substrate on which the aggregate impact test machine is placed (Ramsay *et al.* 1973). Any loosening of the upright support retards the fall of the hammer, reduces its energy, and consequently gives non-reliable results.

The British standard specifies that the test should be performed with the apparatus standing on a concrete or stone block, or a floor at least 450mm thick. It was observed that a rebound followed each drop of the hammer, which was combined with considerable movement of the machine across the floor, possibly causing a vibration of the aggregate and a segregation of the fine products to the base of the cup. This resulted in a diminished buffering of the material at the top of the sample by fine aggregate, thus keeping it in the condition of the first impact, leading to higher impact value and lower impact value residue. (Ramsay *et al.* 1973) suggested the use the apparatus on wooden block instead of concrete. Bastekin (1985) in his study of Scottish limestones found that poor lubrication of the guide runner resulted in low AIV.

4.3.3 *Results and discussion*

The aggregate impact test was carried out in accordance with BS 812 : 1975, whereas the impact residue testing was carried out using the procedure introduced by Ramsay (1965). The impact test was performed on each sample in this study, as an average of 2 tests. In order to overcome the problems of the apparatus moving across the substrate and the rebound of the hammer, a wooden block of 400mm side was used as a base instead of a concrete block. For comparison, however, some tests were carried out with the apparatus on a concrete block (Table 4.2).

The mean value and range of the impact and residue values for the quartz dolerites are given in Table 4.2. The results when a wooden block is used are less than those obtained with a concrete block as would be expected. The impact values for the quartz dolerites vary between 7.3 and 17.1 (with wooden block). The highest impact value (17.1) was from the rocks which are coarser in grain size and had suffered some alteration, and these also have the lowest residue (30.8). Hillend rock has the lowest impact value (7.3) and the highest impact value residue of all (55.7), as well as the smallest range of values. This rock is fresher and finer in grain size than the other dolerites.

Ramsay 1965; Dhir *et al.* 1971; Attewell 1971 and Spence 1979 showed that a good relationship exists between the impact value and the impact value residue. The results (AIV & AIVR) of the quartz dolerites are plotted against each other in Figure 4.3 and an inverse relationship is demonstrated, which has a correlation coefficient of 0.960. This figure shows the sensitivity of impact value residue with changing impact value. In each case the value of impact value residue falls rapidly with increasing impact value.

The influence of varying flakiness index values both on the aggregate impact value (AIV) and the aggregate impact value residue (AIVR) for each of the quartz dolerites is shown in Figures 4.4 and 4.5. Aggregate impact value increases with increasing aggregate flakiness index value while the aggregate impact value residue decreases. At zero flakiness value the range of values (for the finer in grain size and fresher rocks) lie within a relatively narrow band; ranging from 5.5 to 9.1 for AIV and 50.6 to 60.0 for AIVR, with mean values of 7.1 for AIV and 55.2 for AIVR. The plots for the coarser in grain size and altered material (from Westcraigs and Tam's Loup 2), give values ranging from 11.7 to 15.7 for AIV and 37.2 to 44.4 for AIVR, with mean values of 13.7 and 40.6 for the AIV and AIVR respectively. The plots include flakiness values at 0, 25, 60 and 100%, since the 25% value is close to the commercial flakiness value of aggregate produced by a typically modern crushing plant.

The equations of the relationships between both AIV and AIVR and flakiness index value, together with their coefficients of correlation are given in Table 4.3.

The overall relationships between AIV and AIVR and I_F for dolerites from Caldercruix, Duntilland, Cairneyhill, Hillend Tam's Loup 1 and Boards quarry, were found to be:

$$AIV = 7.10 + 0.071\ I_F \qquad r = 0.984$$

$$AIVR = 55.21 - 0.14\ I_F \qquad r = 0.980$$

And the overall relationships between AIV and AIVR and I_F for dolerites from Westcraigs and Tam's Loup 2 quarry, were found to be:

$$AIV = 13.74 + 0.070\, I_F \qquad r = 0.985$$

$$AIVR = 40.61 - 0.19\, I_F \qquad r = 0.973$$

It is apparent that the residue value is more sensitive to any change in the proportion of flaky particles in the quartz dolerites aggregate than the impact value; as the curve between AIVR and I_F exhibits a steeper gradient. This demonstrates the usefulness of the residue fraction in evaluating the influence of aggregate shape on the aggregate impact value test (Dhir *et al.* 1971).

Ramsay *et al.* (1977) plotted the various impact parameters (AIV, AIVR, M) in a triangular diagram. The means of these parameters in the impact tests in this study are plotted in a triangular diagram (Fig. 4.6) which shows that as the flakiness index value increases aggregate strength decreases; which is correlated with increasing proximity to the AIV apex and increasing distance from the AIVR apex in the diagram.

Table 4.2. AIV test results of the quartz dolerites, (I_F = 25%).

Site		AIV^x	$AIVR^x$	$AIV^\wedge$	$AIVR^\wedge$
Caldercruix	mean	8.8	50.23		
	range	(8.0 - 9.7)	(48 - 53)		
	no. of samples	(4)			
Duntilland	mean	9.3	50.75		
	range	(7.9 - 10.7)	(46 - 54)		
	no. of samples	(5)			
Westcraigs	mean	17.1	30.83		
	range	(15.6 - 18.9)	(27 - 35)		
	no. of samples	(5)			
Cairneyhill	mean	7.90	53.45		
	range	(6.51 - 9)	(51 - 56)		
	no. of samples	(5)			
Hillend	mean	7.30	55.65	10	49
	range	(6.6 - 8.0)	(53 - 59)	(9.0 - 11)	(46 - 50)
	no. of samples	(5)		(3)	
Tam's Loup 1	mean	11.5	44.60		
	range	(9.6 - 13)	(41 - 48)		
	no. of samples	(5)			
Tam's Loup 2	mean	14.0	40.0	17	34
	range	(13.1 - 15)	(37 - 43)	(15.4-18.3)	(31-36)
	no. of samples	(4)		(3)	
Boards	mean	8.0	52.35		
	range	(7.0 - 9.2)	(47 - 54)		
	no. of samples	(5)			

^{x} using the apparatus with wooden block.

^ using the apparatus with concrete block.

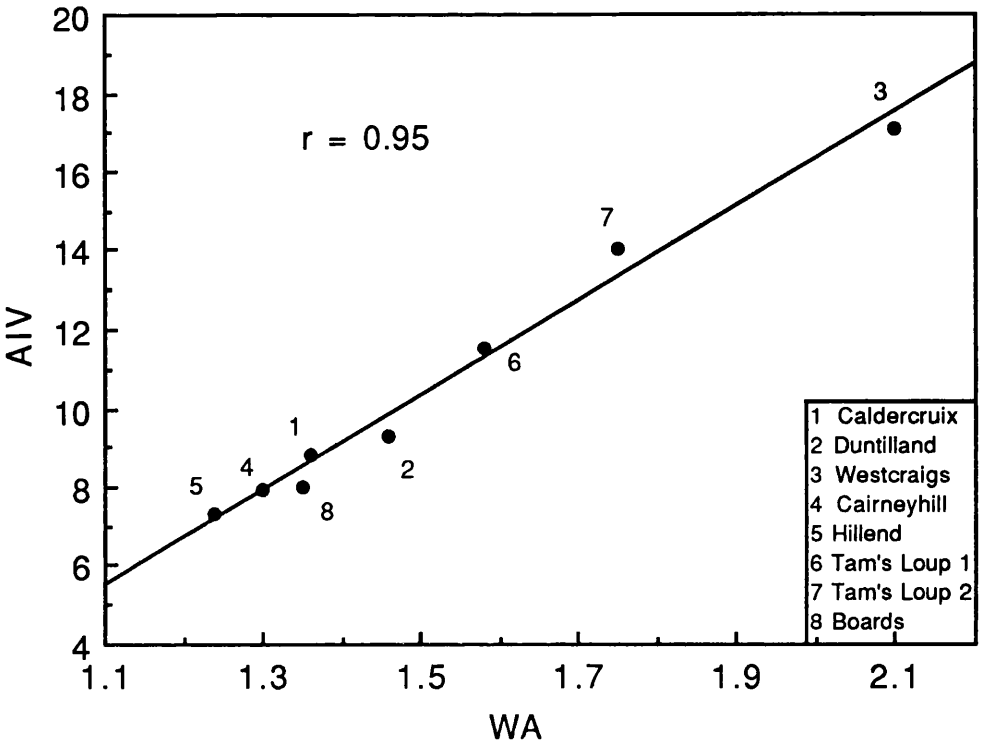


Fig. 4.2 Relationship between water absorption and impact value of the quartz dolerites.

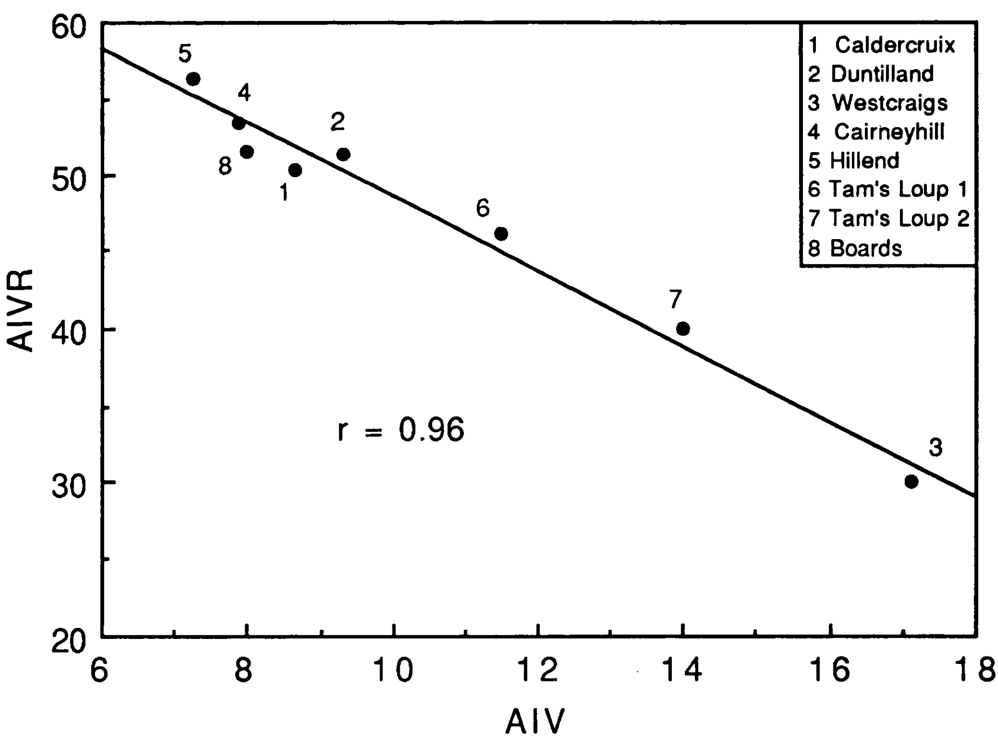


Fig. 4.3 Relationship between impact value and impact value residue of the quartz dolerites.

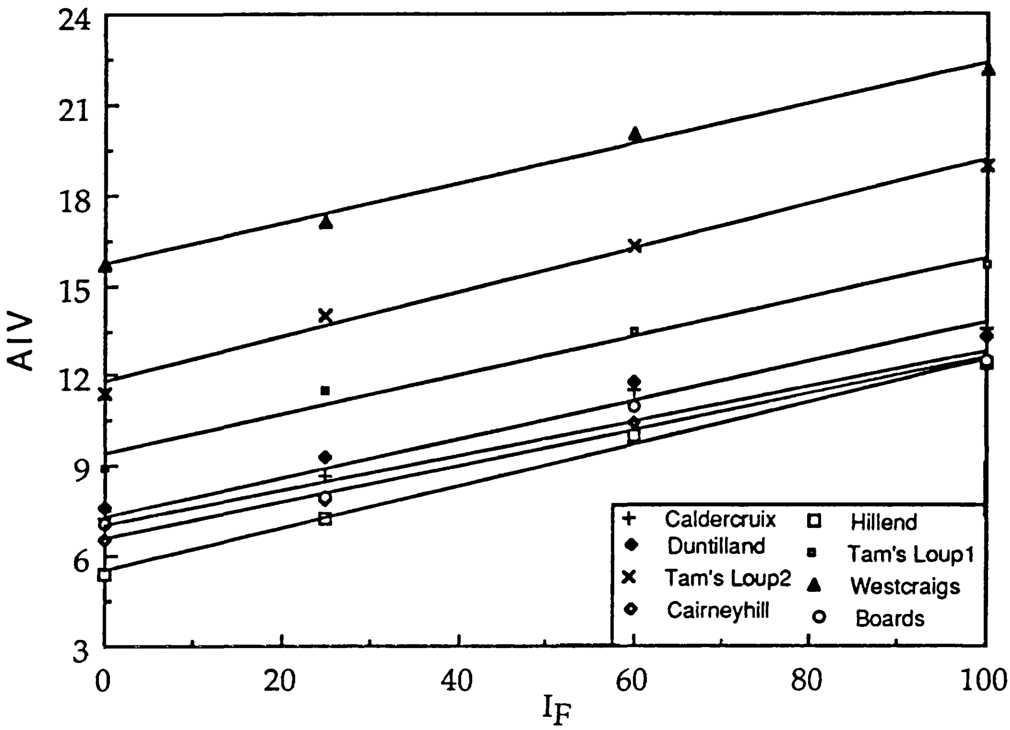


Fig. 4.4 Graph showing the relationship between flakiness index value and impact value of the quartz dolerites.

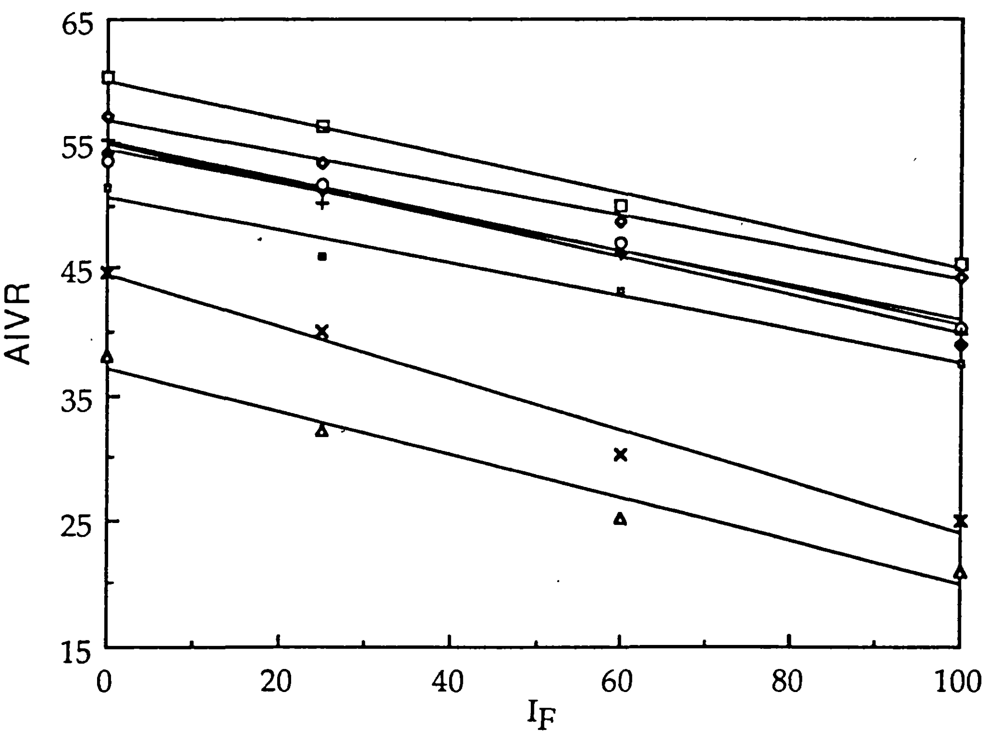


Fig. 4.5 Influence of flakiness index value on impact value residue of the quartz dolerites.

Table 4.3 Regression coefficients for the relationships between AIV, AIVR and the flakiness index value of the quartz dolerites.

Site	AIV			AIVR		
	c	n	r	c	n	r
Caldercruix	7.22	0.065	0.981	54.73	-0.15	0.993
Duntilland	7.39	0.067	0.970	55.15	-0.15	0.938
Cairneyhill	6.52	0.060	0.994	56.83	-0.13	0.996
Hillend	5.50	0.070	0.986	60.00	-0.15	0.992
Tam's Loup 1	9.14	0.065	0.978	50.62	-0.13	0.975
Boards	6.71	0.074	0.987	54.37	-0.14	0.981
Overall	7.10	0.071	0.984	55.21	-0.14	0.980
Tam's Loup 2	11.73	0.074	0.989	44.38	-0.17	0.971
Westcraigs	15.73	0.066	0.982	37.17	-0.18	0.974
Overall	13.74	0.070	0.985	40.61	-0.19	0.973

c = AIV and AIVR at 0 flaky, n = gradient, r = correlation coefficient

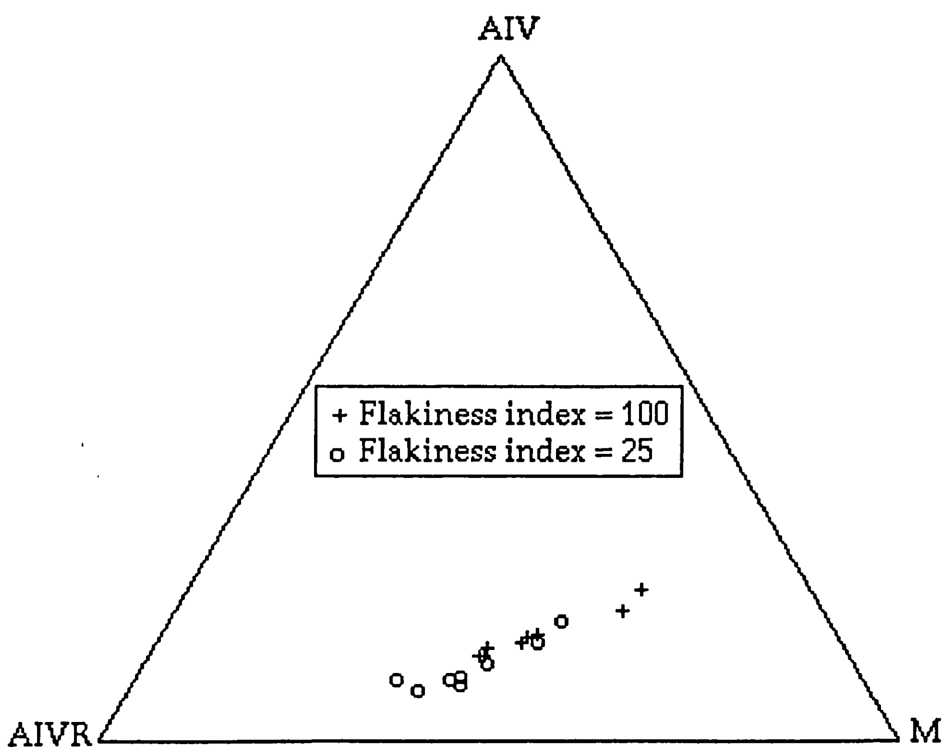


Fig. 4.6 Triangular diagram for aggregate impact tests of the quartz dolerites.

4.4 Aggregate crushing test.

4.4.1 *Introduction*

Most specifications for road aggregates require that the material is strong, in order to withstand road roller and traffic load. To measure the strength directly, relatively sophisticated testing techniques are necessary. A method that indicates the aggregate strength is the aggregate crushing value test (BS 812:1975). The aggregate crushing test gives a relative measure of the resistance of the aggregate to crushing under a gradually applied compressive load, which would be the case with a road carrying slowly moving traffic, or with aggregate being compressed by a road roller during road construction.

In this test a measured volume of material 14 - 10mm in size is placed in an open ended steel cylinder of nominal 150mm internal diameter, and 120mm internal depth, set on a removable base. A metal piston is placed in the cylinder on top of the aggregate ensuring that it is free to move without hindrance and in such a manner that it can transmit a uniformly applied load to the aggregate. The whole assembly is placed in a loading frame and loaded at a steady rate of 0.67kN per second until the maximum loading of 400kN is reached in 10 minutes. The compacted sample is extracted from the test cylinder and sieved, and the amount passing a 2.36mm sieve is expressed as a percentage of the initial sample. The result is the aggregate crushing value (ACV); and here again a lower value indicates a more resistant rock to degradation, i.e., high aggregate quality. The mass retained on the 10mm sieve after completion of the test is expressed as a percentage of the initial sample and is called the aggregate crushing value residue, ACVR (Dhir *et al.* 1971).

Values below 15 indicate a very strong aggregate, whereas exceptionally strong aggregates yield values of 10 or less. Aggregates with crushing values of 25 or less are generally considered to be strong enough for all road making purposes. Material with values as high as 35 can be used in road surfacing if special precautions are used (such as the use of a rubber-tyred roller), but such aggregates are usually lacking in abrasion resistance. Aggregates with crushing values greater than 35 would be regarded as too weak for any engineering use. The geological factors affecting the crushing value are the same as those for the impact test, and have the same effects (Dhir *et al.* 1971).

4.4.2 *Results and discussion*

The ACV tests on quartz dolerites in this study were carried out according to BS 812:1975. The mean and range of results for the aggregate crushing and residue values are given in Table 4.4. In this Table the results of the tests vary between 14.0 and 20. Hillend rocks have the lowest crushing value (14.0) and the highest crushing value residue of all (39), The highest crushing value (20.1) being from the rocks which are coarser in grain size and suffered some alteration from Westcraigs quarry which also have the lowest residue (19.5).

The relationship between the aggregate crushing value (ACV) and aggregate crushing value residue (ACVR) for each of the quartz dolerite sites is illustrated in Figure 4.8, which has a pattern similar to that obtained between AIV and AIVR; an increase in the ACV results in a decrease in ACVR.

The influence of varying flakiness index values, on both the aggregate crushing value (ACV) and the aggregate crushing value residue (ACVR) is shown in Figures 4.9 and 4.10. The pattern is similar to that obtained with impact test; the aggregate crushing value increases with increasing aggregate flakiness while the aggregate crushing value residue decreases. At 0% flakiness index value the plots for the finer and fresher rocks lie within a relatively narrow band; ranging from 12.9 to 14.2 for the ACV and 32.5 to 38.3 for the ACVR with mean values of 13.7 and 35.3 respectively. The plots for the relatively coarser in grain size and weathered material (Westcraigs and Tam's Loup 2), show a range from 16.7 to 18.3 for the ACV and 21.4 to 23.5 for the ACVR with mean values of 17.5 and 22.5 respectively. The overall relationships between ACV and ACVR and flakiness index value (I_F) for the finer grained size and fresher dolerites were found to be:

$$ACV = 13.73 + 0.05\ I_F \qquad r = 0.980$$

$$ACVR = 35.29 - 0.20\ I_F \qquad r = 0.973$$

and the relationships between ACV and ACVR and I_F for the coarser grain size and weathered dolerites were found to be:

$$ACV = 17.49 + 0.065\ I_F \qquad r = 0.970$$

$$ACVR = 22.46 - 0.12\ I_F \qquad r = 0.992$$

Once again it is apparent that the residue value is more sensitive to a change in the flaky particle proportion in the aggregate.

The results obtained from the AIV and ACV tests of the quartz dolerites were plotted against each other (Fig. 4.11). The relationship between the impact and crushing values can be said to be linear; the impact value increasing with aggregate crushing value. The results show that the aggregate crushing values are on average higher than the impact values by 4.4. Dhir *et al.* (1971) concluded that the aggregate crushing value, for practical purposes, is predictable from the impact value. The relationships between AIV and ACV, and AIVR and ACVR (Figs 4.12 and 4.13) are essentially the same.

4.5 10% fines test.

4.5.1 *Introduction*

In this test the gradually applied compressive load required to produce 10% fines from the aggregate is determined. A uniform loading rate is applied to cause a total penetration of the plunger of approximately 15mm (for rounded aggregates), 20mm (for normal crushed rock) and 24mm (for honeycombed aggregate) in 10 minutes. The ten per cent fines value is obtained by weighing the fraction passing the 2.36mm BS sieve and expressing it as a percentage of the initial test sample. Two or three tests have to be carried out on each sample, so that a graph can be constructed for per cent fines against loading and then extrapolated to get the loading value required to obtain exactly 10% fines. This test yields results which range from about 400kN for the strongest aggregates down to 10kN or less for weak material such as crushed chalk and bricks.

It should be noted that in this test, unlike the standard crushing value test, a higher numerical result denotes a higher aggregate strength. BS 882 : 1983 prescribes a minimum value of 100kN for aggregate to be used in concrete wearing surfaces, and 50kN when used in other concretes. 10% fine values show a good relationship when correlated with the standard crushing value.

One method of estimating the force required for the ten per cent fines test on aggregate is to divide the number 4000 by the aggregate impact value (BS 812:1975); the fines less than 2.36mm produced by this force should fall within 7.5 to 12.5 per cent of the initial weight. However, Bullas (1990) suggested that this method commonly produces too large a 10% fines value, and he suggested that a factor of 2800 would increase the probability of a valid estimate of the test load. It is interesting to note that Gribble (1991) found that the previous value of 4000 works quite well when testing aggregate from sand and gravel sources. He suggested that two values of K are needed; 2800 for aggregate from crushed rock material which is the one proposed by Bullas, and 4000 for aggregate from sand and gravel sources, which is the value previously used in this type of calculation; a suggestion with which Bullas (1991) agreed.

In the present study the two values (4000 and 2800) were tried in estimating the force required to produce the ten per cent fines. The value of 4000 gave too high forces, except in the case of weaker rocks where AIV is relatively high. On the other hand the factor of 2800 worked with most impact values and gave forces that would produce between 7.5 and 12.5 per cent fines of the initial sample weight.

4.5.2 *Results and discussion*

The 10% fines value test was carried out in accordance with the procedure outlined in the BS 812:1975. Only a few samples could be tested, since a large amount of material is needed for this test. The ten per cent fines tests were carried out on materials with a flakiness index value of 25%. The mean value and range of values for the 10% fines are given in Table 4.4. The results show that the strength of the quartz dolerite aggregate is higher (with a high 10% fines value) in Hillend rocks (282kN) due to a finer grained and fresher material. The lowest value (191) is given by the dolerite from Westcraigs quarry. Ten per cent fines values are plotted against water absorption, Figure 4.13; and high 10% fines values are associated with low water absorption values. In Figures 4.14 and 4.15 the 10% fines values are plotted against AIV and ACV and linear inverse relationships are shown to occur.

The influence of varying the flakiness index value on the 10% fines test, has been omitted from this discussion, because the test requires a large quantity of material and not enough material was available to perform this test at different flakiness index values. Flakiness index value, however, is thought to have the same influence as it has on the AIV and ACV tests; that is, aggregate with higher flakiness index values has lower 10% fines values.

Table 4.4 ACV and 10% fines tests results of the quartz dolerites, (I_F = 25%).

Site		ACV	ACVR	10% fines
Caldercruix	mean	15.1	37.1	260
	range	(13.6 - 16)	(33.1 - 40)	(256-265)
	no. of samples	(4)		(2)
Duntilland	mean	15.3	35.21	250
	range	(14.0-16.7)	(31.0-36.8)	(246-254)
	no. of samples	(5)		(3)
Westcraigs	mean	20.1	19.5	191
	range	(18 - 22)	(17.2 - 21.0)	(185-197)
	no. of samples	(5)		(2)
Cairneyhill	mean	14.5	39.7	
	range	(13.0 - 15.1)	(35.41 - 42.0)	
	no. of samples	(5)		
Hillend	mean	14.0	39.38	282
	range	(13.0 - 15.0)	(36.12 - 40.0)	(277-287)
	no. of samples	(5)		(2)
Tam's Loup 1	mean	15.8	35.31	250
	range	(14.68-16.0)	(29.1 - 35.0)	(245-256)
	no. of samples	(5)		(2)
Tam's Loup 2	mean	18.24	25.0	200
	range	(17.0 - 20.1)	(21.0 - 27.0)	(196-204)
	no. of samples	(4)		(2)
Boards	mean	15.0	38.8	256
	range	(13.76 - 16.1)	(29.0 - 35.0)	(251- 260)
	no. of samples	(5)		(2)

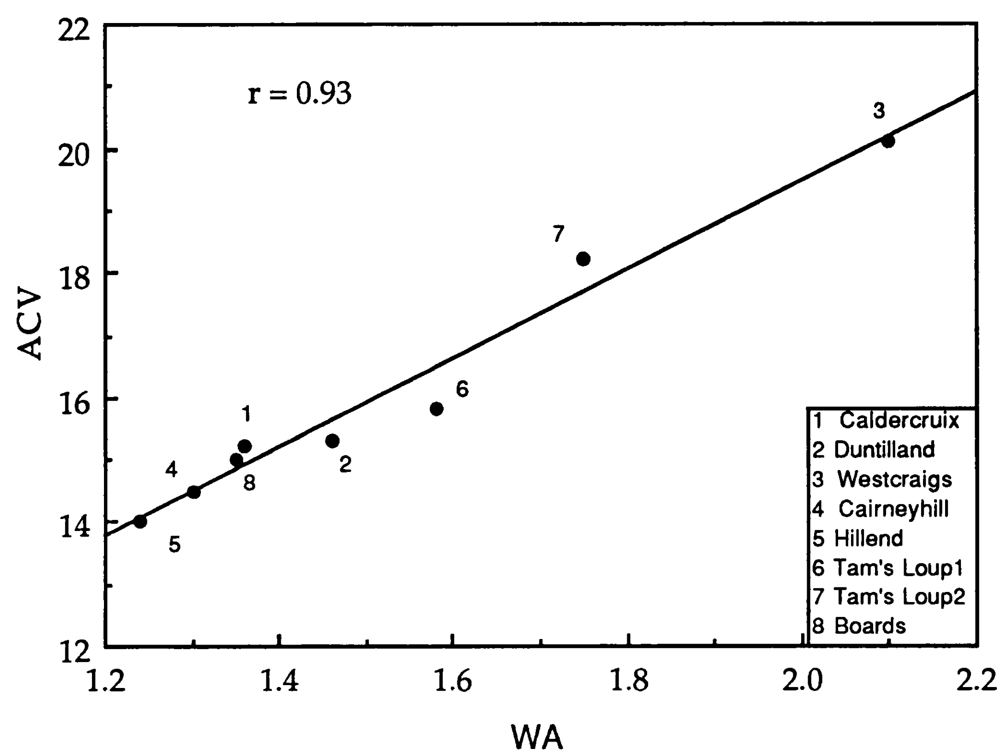


Fig. 4.7 Relationship between aggregate crushing value and water absorption of the quartz dolerites.

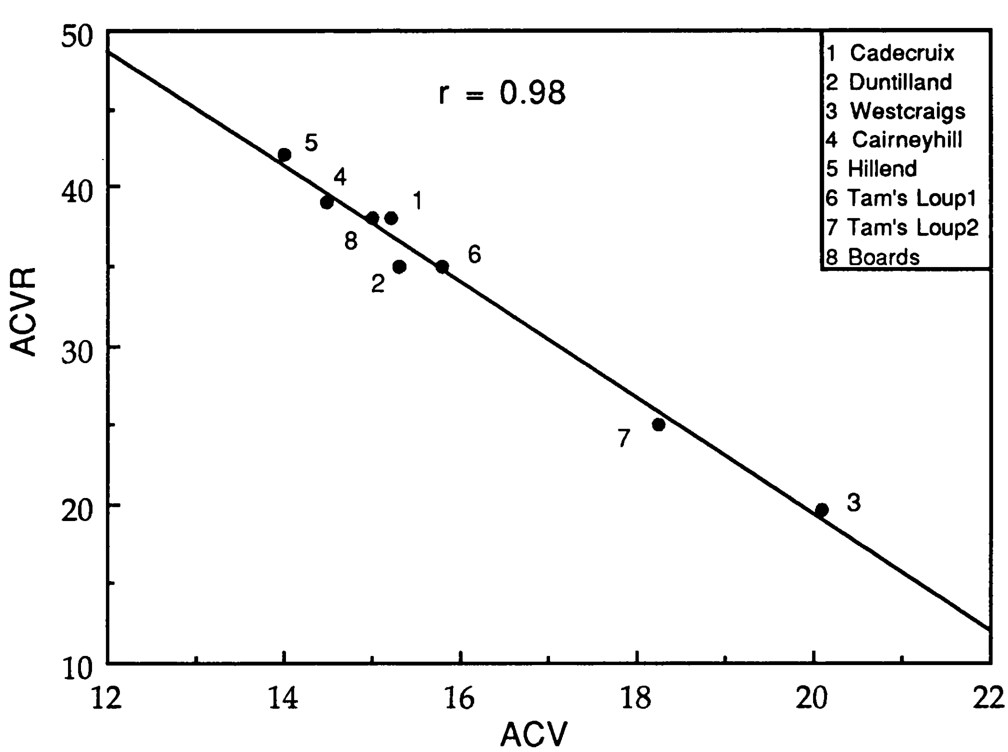


Fig. 4.8 Relationship between aggregate crushing and residue value of the quartz dolerites.

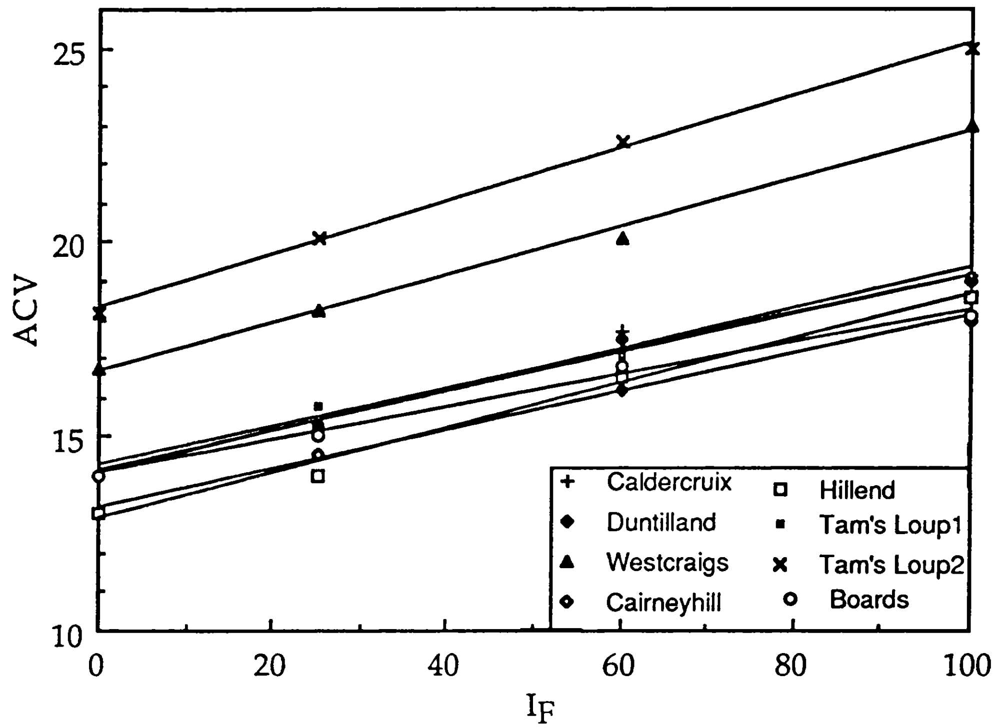


Fig. 4.9 The influence of the flakiness index value on the aggregate crushing value of the quartz dolerites.

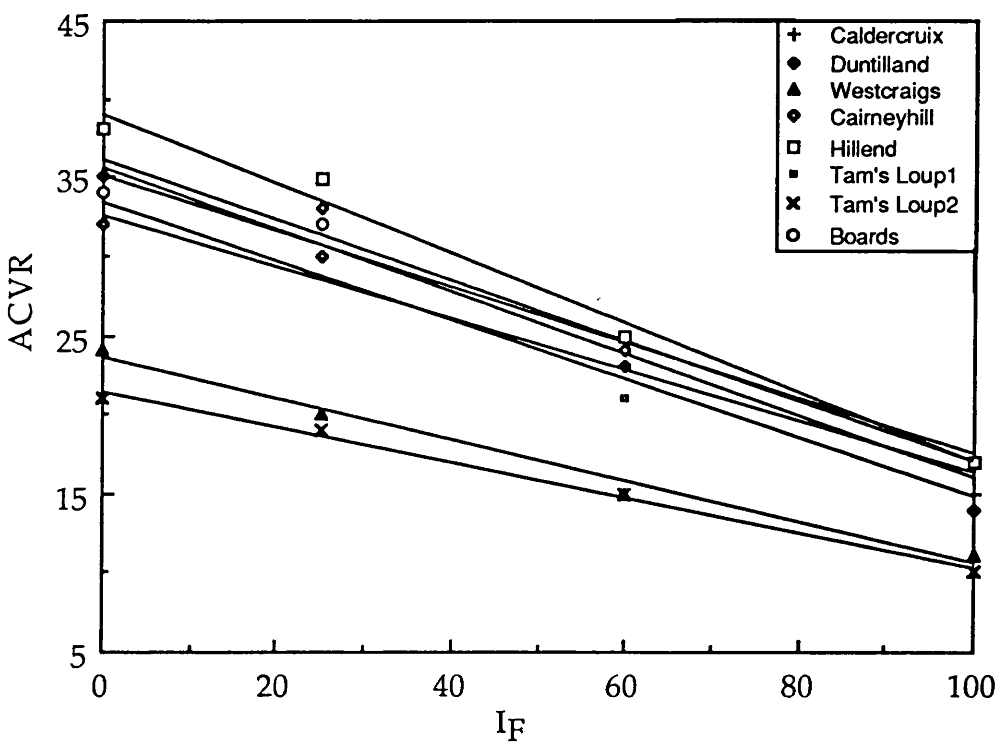


Fig. 4.10 Influence of the flakiness index value on the aggregate crushing value residue of the quartz dolerites.

Table 4.5 Regression coefficients for the relationships between ACV and ACVR and the flakiness index of the quartz dolerites.

Site	ACV			ACVR		
	c	n	r	c	n	r
Caldercruix	14.05	0.053	0.982	35.50	-0.195	0.970
Duntilland	14.10	0.051	0.988	33.31	-0.185	0.976
Cairneyhill	13.14	0.050	0.990	36.03	-0.189	0.979
Hillend	12.85	0.058	0.990	38.34	-0.223	0.947
Tam's Loup 1	14.23	0.050	0.986	32.47	-0.164	0.959
Boards	14.03	0.042	0.990	35.12	-0.176	0.981
Overall	13.73	0.051	0.980	35.29	-0.20	0.973
Westcraigs	18.32	0.068	0.945	21.40	-0.111	0.994
Tam's Loup 2	16.65	0.062	0.925	23.52	-0.130	0.989
Overall	17.49	0.065	0.970	22.46	-0.121	0.992

c = ACV and ACVR at 0 flaky, n = gradient, r = correlation coefficient

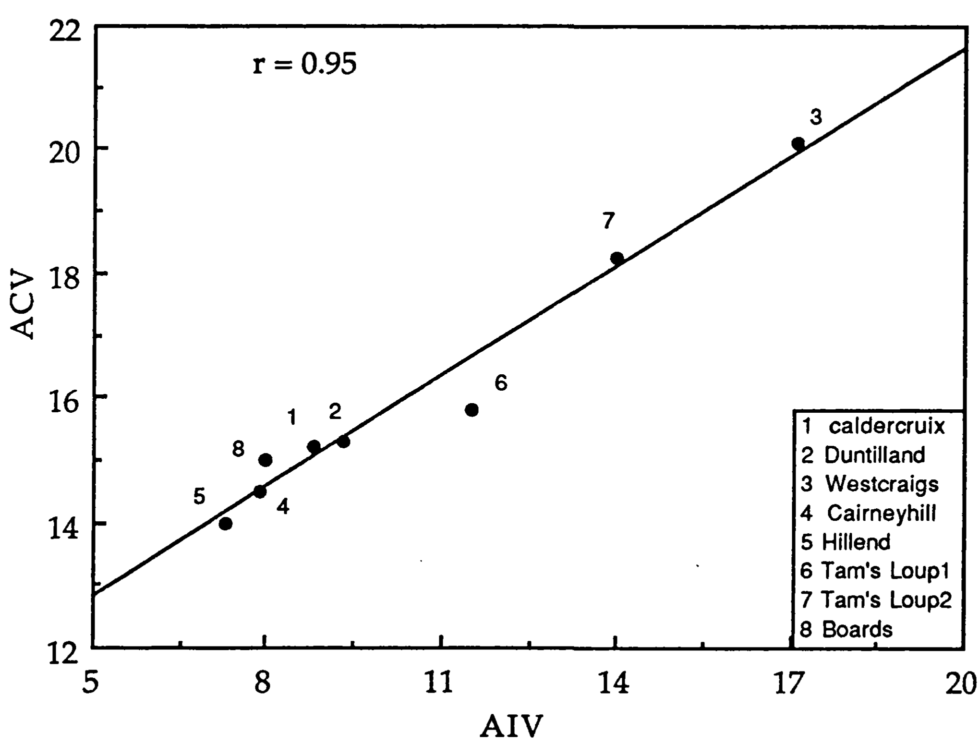


Fig. 4.11 Relationship between the aggregate impact and crushing value of the quartz dolerites.

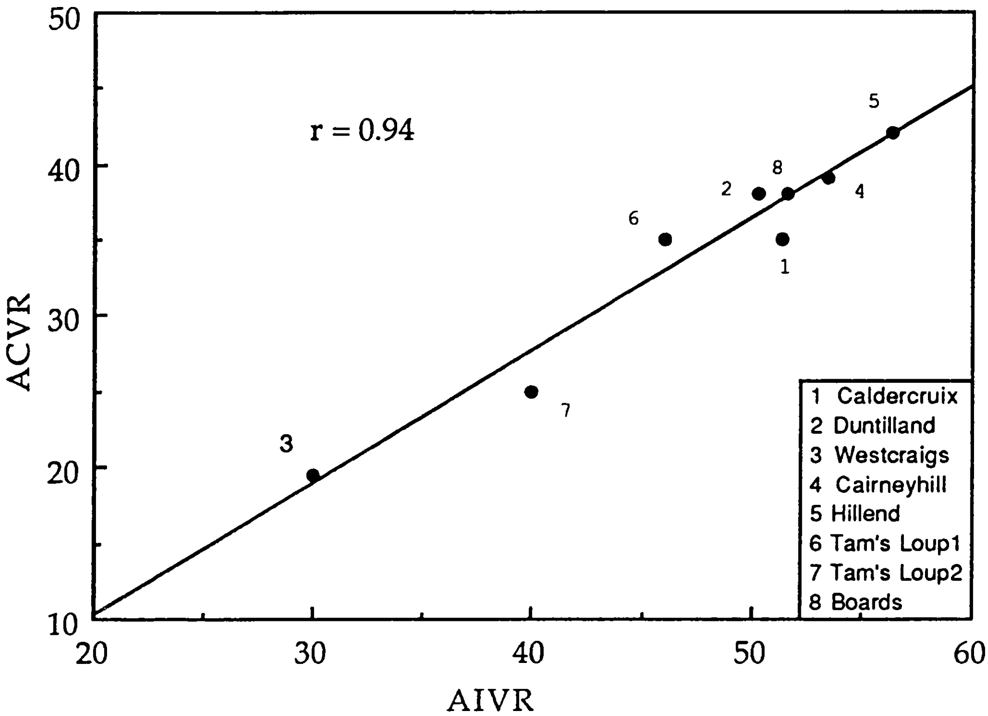


Fig. 4.12 Graph showing the relationship between aggregate impact value and crushing value residue.

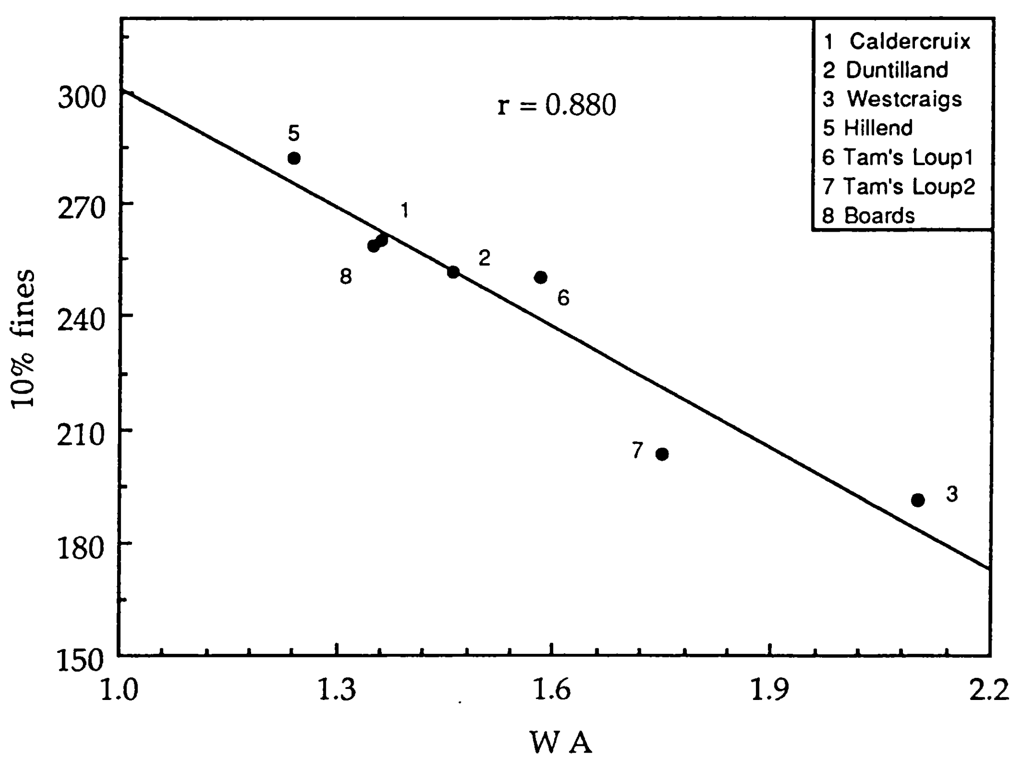


Fig. 4.13 Influence of water absorption on the 10% fines of the quartz dolerites.

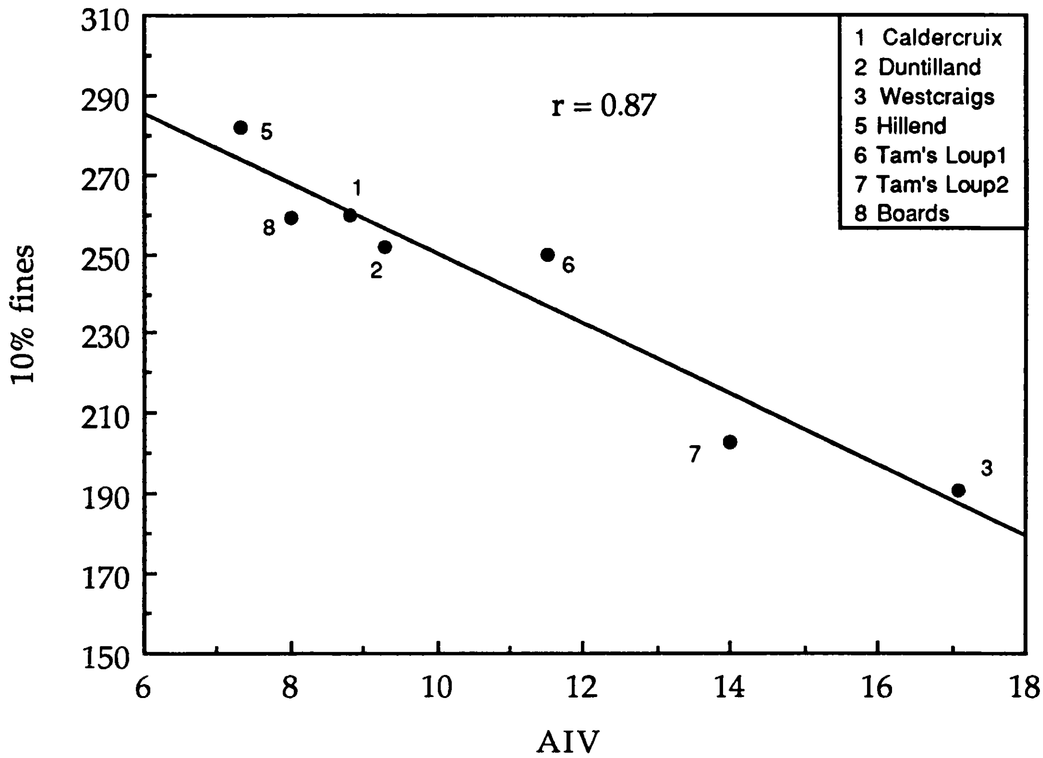


Fig. 4.14 Relationship between 10%fines and impact value of the quartz dolerite aggregate.

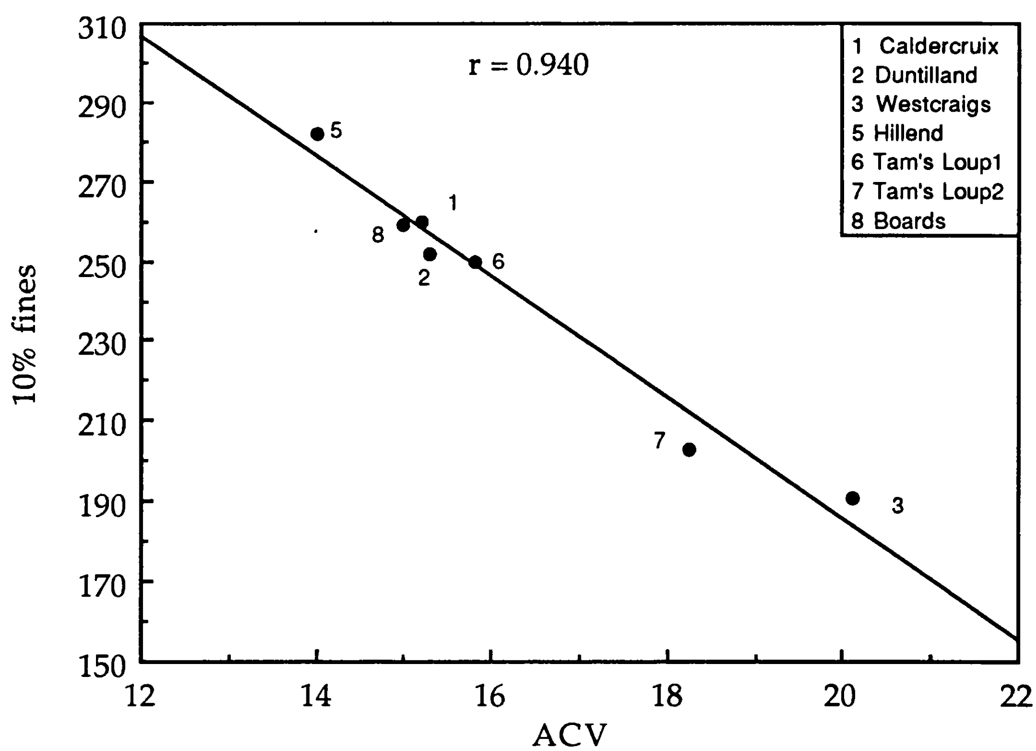


Fig. 4.15 Relationship between 10%fines and ACV of the quartz dolerite.

4.6 Aggregate abrasion test

4.6.1 *Introduction*

In addition to strength, hardness (or resistance to wear) is an important property of the aggregates used in roads or in floor surfaces subjected to wear. Several means have been devised for assessing the abrasion resistance of aggregates, the two common methods being the Dory abrasion test (BS 812:1975, part 3) and the Los Angeles abrasion test (ASTM C131 - 86).

The Dory aggregate abrasion test provides a measure of the resistance of the aggregate to surface wear by abrasion; the lower the value, the greater the resistance. The aggregate abrasion value is expressed as the average percentage loss of weight of two specimens each consisting of aggregate particles mounted on a flat plate which is subjected to abrasion by a standard quartz sand in a Dory abrasion machine consisting essentially of a machined flat circular steel grinding lap. Values range from 1 for very resistant materials (e.g., hard flints, quartzite) to over 16 for aggregate which would normally be considered too weak (e.g., limestones, weak sandstones) for use in road surfacing (Hawkes and Hosking 1972).

The degree of abrasion resistance required is governed by the type of road site and the amount of traffic. Inadequate abrasion resistance of road surfacing aggregates means an early loss of the texture depth that is required to maintain high-speed skid resistance. Maximum acceptable aggregate abrasion values and traffic loading for materials used in highway running surfaces in the United Kingdom are given in Table 4.6.

Table 4.6 Maximum aggregate abrasion values and traffic loading in the UK (Department of Transport, 1990).

Traffic[+]	*Under 250*	*Up to 1000*	*Up to 1750*	*Up to 2500*	*Up to 3250*	*Over 3250*
Maximum AAV for chippings	14	12	12	10	10	10
Maximum AAV for aggregate in coated macadam wearing course	16	16	14	14	12	12

[+]*in commercial vehicles per lane per day.*

The AAV value reflect the hardness and brittleness of the mineral constituents, the influence of surface texture, grain size, mineral cleavage and the strength of the intergranular bond. Acid igneous rocks with a high quartz content tend to be more resistant to abrasion than basic varieties with high ferromagnesian minerals and low quartz content. This is due to the hardness and lack of cleavage in quartz crystals. Another factor which generally results in increasing abrasion is chemical decomposition, which destroys the intergranular bond between the mineral grains, particularly in siliceous sedimentary rocks, where the resistance is entirely dependent upon the nature of such an intergranular bond.

There appears to be some correlation between the strength of an aggregate and its abrasion resistance value. Evidence has been presented which indicates that a high aggregate crushing value (ACV) can be tolerated if the material has a low aggregate abrasion value (see Table 4.7).

Table 4.7 Correlation between AAV, ACV and service life.
(after Wilson, 1966)

Rock type	AAV	ACV	Service life(Yrs)
Slag	7	26	>9
Limestone	17	28	2

The results in Table 4.7 indicate that the future degradation of an aggregate which survives the rigours of laying is more the result of abrasion forces than directly applied load.

4.6.2 *Results and discussion*

The aggregate abrasion value tests in this study were carried out in accordance with BS 812:1975. The aggregate abrasion values were calculated using the following formula: AAV = 3 (A-B) / d, where A is the mass of the specimen before the test; B is the mass of the specimen after the test and d is the density of the rock material on saturated and surface dried base.

The results given in Table 4.8 are the mean and range of aggregate abrasion values obtained from each site. The range of aggregate abrasion values determined for the quartz dolerites is between 4.7 and 8.0. The results show that the resistance to abrasion of the quartz dolerites is highly influenced by the state of weathering and grain size of the rock. Hillend rocks which are relatively fresh and finer in grain size possess a low aggregate abrasion value (4.7), whereas, Westcraigs quartz dolerite, which is altered and coarser in grain size, possesses a higher aggregate abrasion value

(8). It is clear that the presence of soft secondary minerals and coarser grain size give higher aggregate abrasion values.

4.7 Los Angeles Abrasion test

4.7.1 *Introduction*

This is a dry abrasion and impact test originally devised by the staff of the Los Angeles City Engineer's office in 1916 and, later, adopted by a number of state highway departments in America as an acceptance test for coarse aggregates in place of the tests formerly used. Experiments made in 1935 by Woolf and Runner resulted in the test being modified by the introduction of an abrasive charge of cast iron spheres in place of the cubical cast iron blocks originally used. In the test a sample of aggregate and a charge of steel balls are placed within a hollow steel cylinder, mounted horizontally. The cylinder is 508mm long and 711mm in diameter, closed at the ends, and has a single rigid steel shelf extending 89mm into the chamber. It is rotated at 30 to 33 revolutions per minute for 500 revolutions, and the percentage loss is obtained after 500 revolutions as the difference between the original sample weight and the amount of the final sample coarser than 1.70mm.

Although the Los Angeles abrasion test requires large quantities of aggregates of specified grading, it is extremely useful and widely used (USA and elsewhere) in determining the quality of aggregates.

4.7.2 *Results and discussion*

The Los Angeles abrasion tests were carried out in accordance with ASTM Designation C131 - 86 using grade B materials (sizes 20 - 14mm and 14 - 10mm), obtained either from crushing and processing plants operating in the area or by crushing large blocks in the Department laboratory. The results given in Table 4.8 are the mean and range of the Los Angeles abrasion tests carried out for each site. The highest LAAV (18.1) is attained by aggregates from Westcraigs, whilst the lowest value (12.0) is shown by Cairneyhill quartz dolerites. In general, it may be noticed that coarse grained and weathered rocks exhibit high Los Angeles abrasion values, whereas, the finer grained and fresher rocks are characterised by relatively low Los Angeles abrasion values.

The relationship between LAAV and LAAVR (Los Angeles abrasion value residue) is shown in Figure 4.16; as the LAAV increases the LAAVR decreases. In order to investigate the influence of the flakiness index value on the LAAV test, flakiness index values have been varied artificially between 0 and 100%. Figures 4.17 and 4.18 illustrate the relationship between the LAAV and LAAVR and flakiness index value. The Figures show that the Los Angeles abrasion value increases with increasing flakiness index value, whereas the LAAVR decreases. The Los Angeles abrasion test, however, appears to be less sensitive to the change in the flakiness index value than the aggregate impact and crushing value tests. This may be because in the aggregate impact and crushing tests the aggregates are subjected to direct force, whereas in the Los Angeles abrasion test the force is either tangential or direct. The overall relationships between the LAAV and LAAVR and the flakiness index value are as follows:

$$LAAV = 11.99 + 0.035\ I_F \qquad r = 0.805$$

$$LAAVR = 55.10 - 0.28\ I_F \qquad r = 0.962$$

For dolerites from Caldercruix, Duntilland, Cairneyhill, Hillend, Tam's Loup 1 and Boards quarries.

and for dolerites from Westcraigs and Tam's Loup 2 quarries are:

$$LAAV = 16.78 + 0.03\ I_F \qquad r = 0.776$$

$$LAAVR = 33.79 - 0.15\ I_F \qquad r = 0.874$$

The relationship between the Los Angeles abrasion value and the aggregate impact tests is demonstrated in Figure 4.19. This Figure reveals a relatively high degree of correlation; as the Los Angeles abrasion value increases the aggregate impact value increases as well. The same can be said about the relationship between LAAV and ACV (Fig. 4.20), and the relationships can be surmmarized as follows:

$$LAAV = 7.41 + 0.64\ AIV \qquad r = 0.92$$

$$LAAV = -3.35 + 1.1\ ACV \qquad r = 0.95$$

The relationship between LAAV and AIV demonstrated in this study is similar to that reported by Kazi and Al-Molki (1982) using dacite aggregates, whose equation of the relationship between Los Angeles abrasion value and aggregate impact value was as follows: LAAV = 8.7 +

0.23 AIV, with correlation coefficient of 0.95 is close to the equation obtained in this study. It can be concluded that the LAAV is predictable from the AIV. The best fit line of the relationship between LAAV and ACV has higher correlation coefficient (0.95) than that of LAAV and AIV.

The relationship between the Los Angeles abrasion value and aggregate abrasion value is shown in Figure 4.21 with the best fit line which has a correlation coefficient of 0.940

Table 4.8 Mean and range of the AAV, LAAV and LAAVR results of the quartz dolerites.

Site		AAV	LAAV[x]	LAAVR[x]
Caldercruix	mean	5.1	12.7	49.7
	range	(4.6 - 5.5)	(11.9 - 13)	(48.3-52)
	no of samples	(3)		
Duntilland	mean	5.4	13.3	45.0
	range	(4.8 - 6.7)	(12.2 - 15)	(39 - 48.3)
	no of samples	(5)		
Westcraigs	mean	8.0	18.13	23.0
	range	(7.1 - 9.0)	(17.4 - 19.1)	(18.4 - 25)
	no of samples	(5)		
Cairneyhill	mean	5.0	12.0	52.22
	range	(4 - 6)	(11.7 - 13)	(48 - 54.1)
	no of samples	(5)		
Hillend	mean	4.7	12.3	51.3
	range	(4.0 - 5.0)	(11.5 - 13.7)	(49.2 - 53)
	no of samples	(5)		
Tam's Loup 1	mean	5.6	14.2	43.1
	range	(4.7 - 6.2)	(12.8 - 15.1)	(37 - 50.2)
	no of samples	(5)		
Tam's Loup 2	mean	7.4	17.4	28.7
	range	(6.7 - 8.8)	(15.9 - 18.9)	(25.8 - 33)
	no of samples	(4)		
Boards	mean	5.0	13.0	48.4
	range	(4.3 - 5.8)	(11.9 - 14)	(45.3 - 51)
	no of samples	(5)		

[x] Test carried out on material with 25% flaky particles.

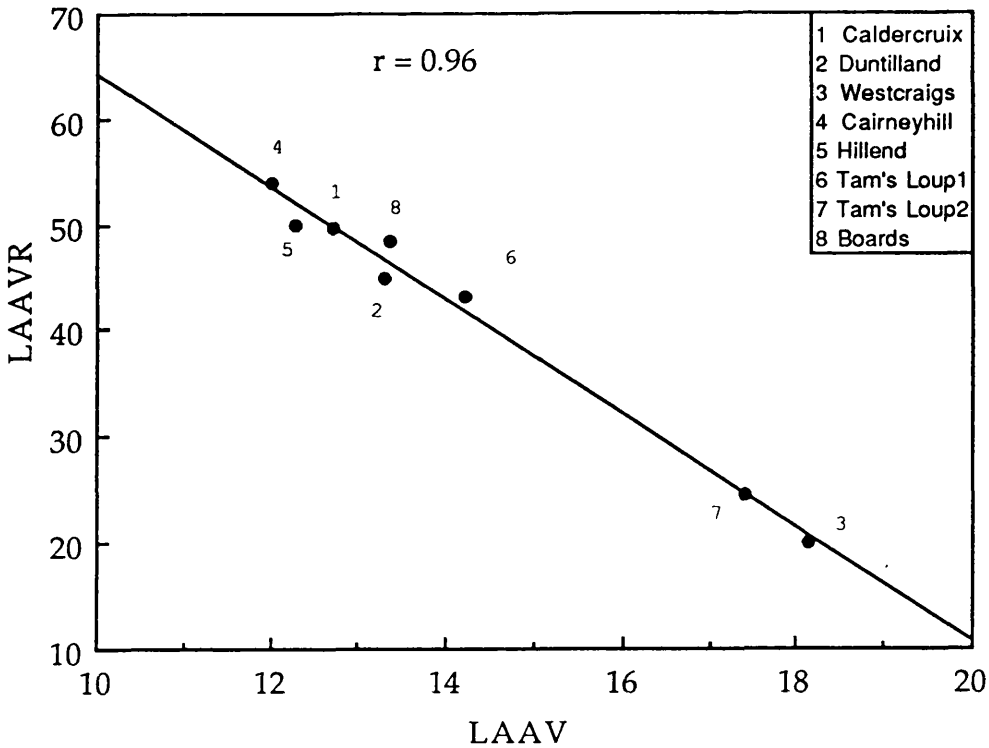


Fig. 4.16 Relationship between LAAV and LAAVR of the quartz dolerites.

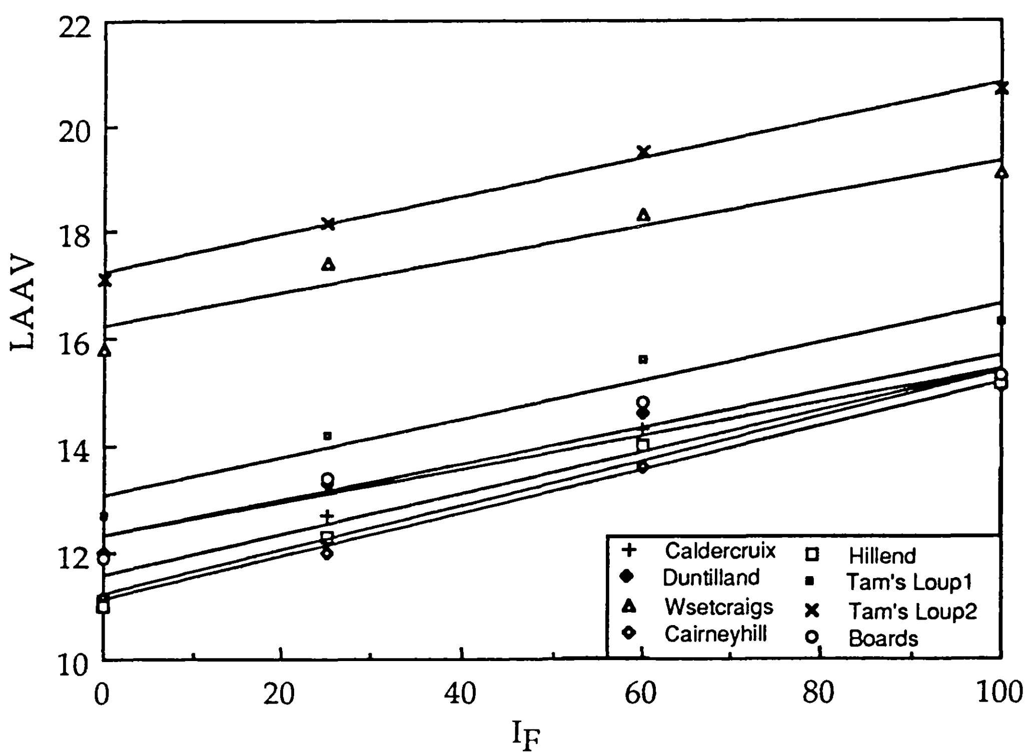


Fig. 4.17 Graph showing the relationship between flakiness index value and LAAV of the quartz dolerites.

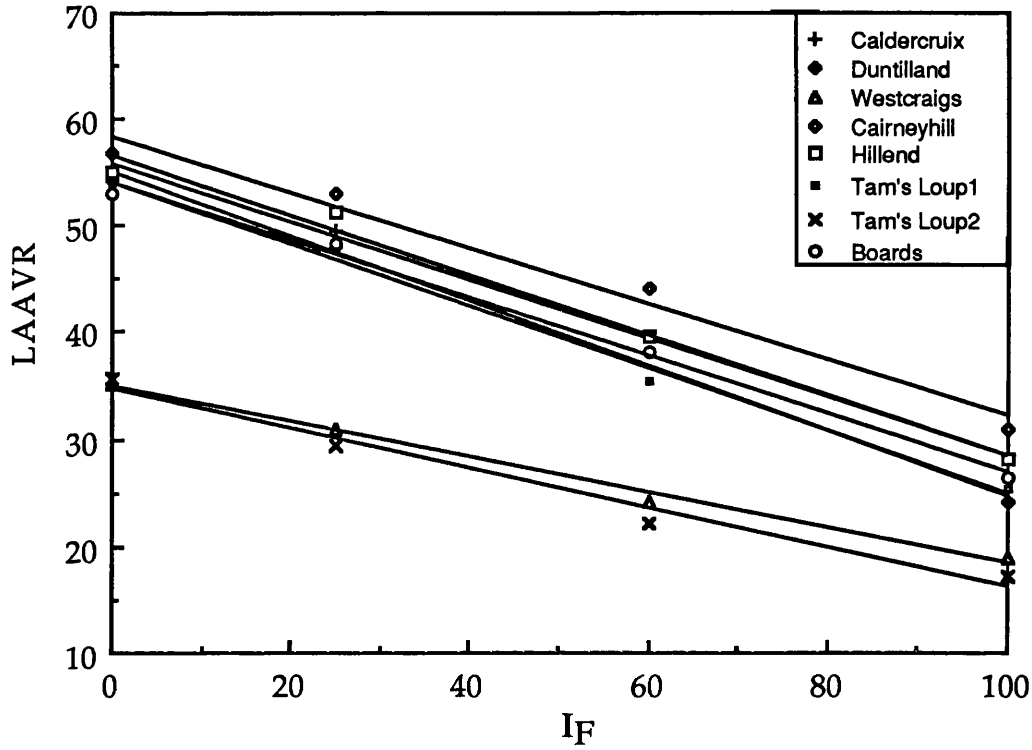


Fig. 4.18 The influence of flakiness index value on LAAVR of the quartz dolerites.

Table 4.9 Regression coefficients for the relationships between LAAV and LAAVR and the flakiness index value.

Site	LAAV			LAAVR		
	c	n	r	c	n	r
Caldercruix	11.53	0.038	0.950	55.76	-0.27	0.989
Duntilland	12.32	0.031	0.931	54.14	-0.30	0.995
Cairneyhill	11.10	0.038	0.990	58.38	-0.26	0.980
Hillend	11.19	0.042	0.981	56.48	-0.28	0.989
Tam's Loup 1	13.10	0.036	0.940	54.16	-0.29	0.994
Boards	12.29	0.034	0.918	54.10	-0.27	0.994
Overall	11.99	0.035	0.805	55.10	-0.28	0.962
Tam's Loup 2	16.20	0.031	0.927	34.89	-0.16	0.987
Westcraigs	17.20	0.034	0.990	34.31	-0.19	0.982
Overall	16.781	0.033	0.776	33.79	-0.15	0.874

c = LAAV and LAAVR at zero flaky particles, n = gradient and r = correlation coefficient

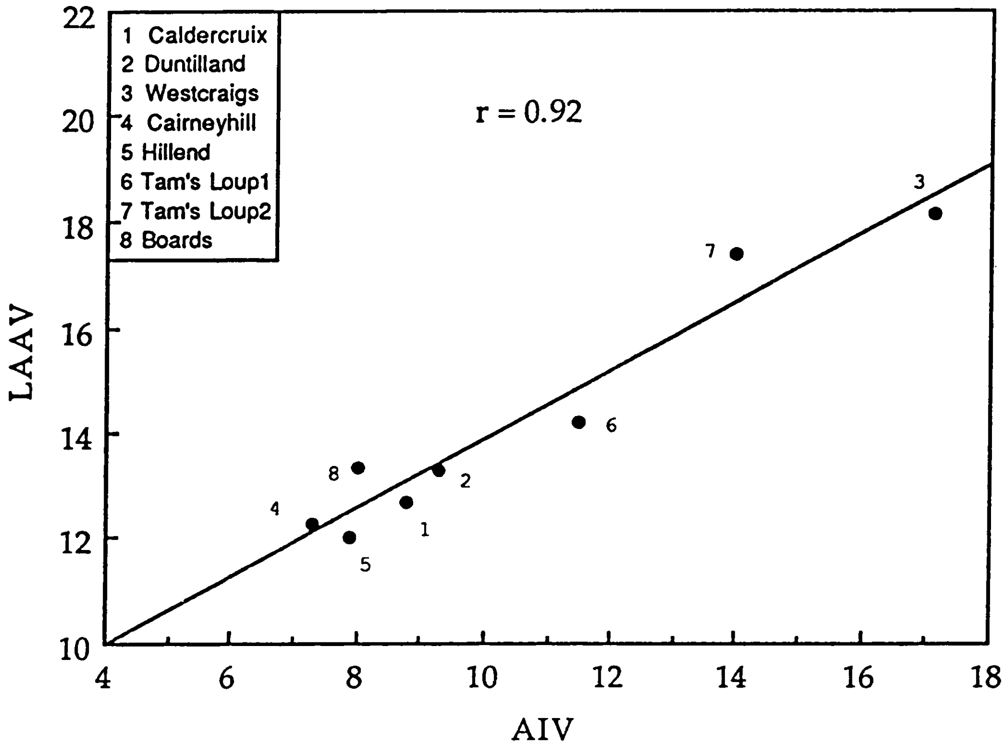


Fig. 4.19 Graph showing the relationship between LAAV and AIV.

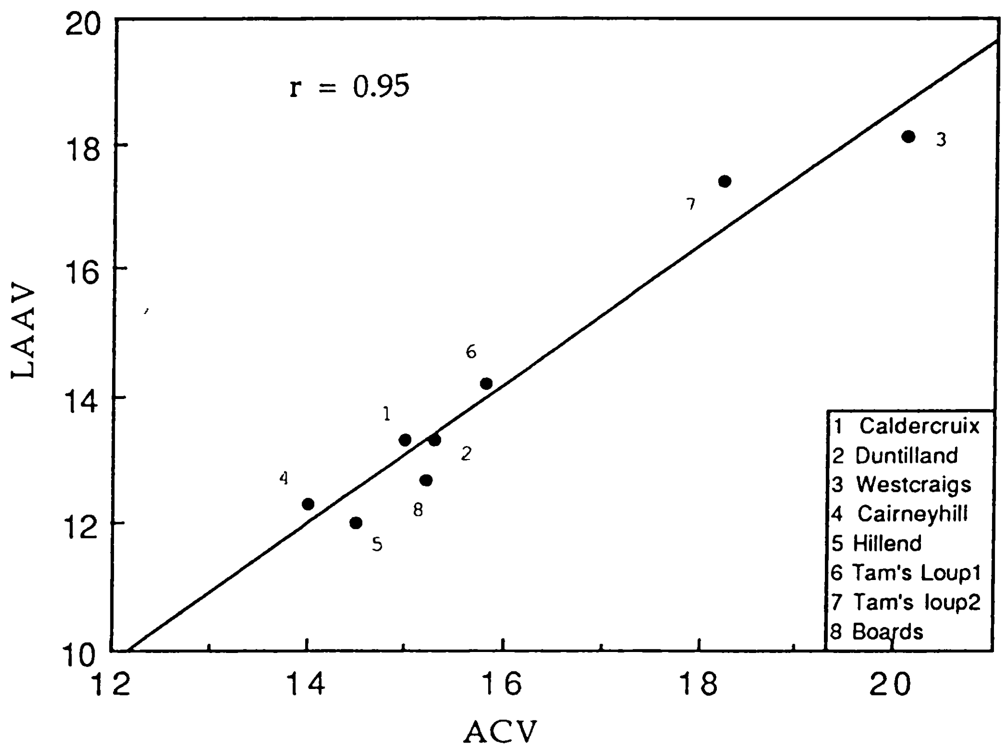


Fig. 4.20 Relationship between LAAV and ACV of the quartz dolerites.

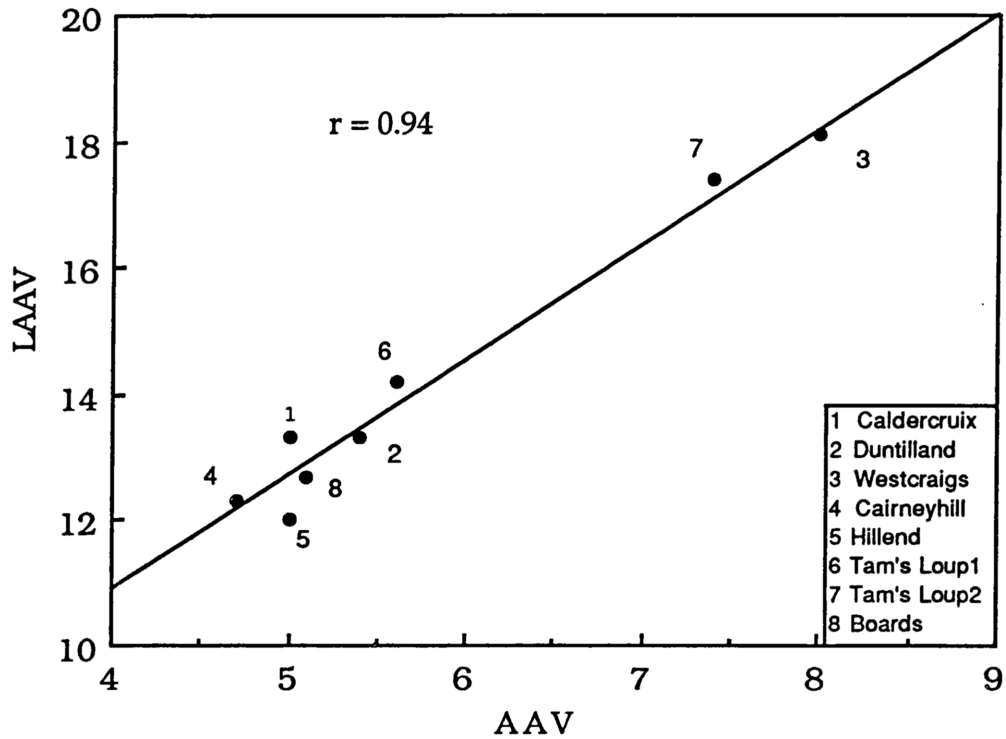


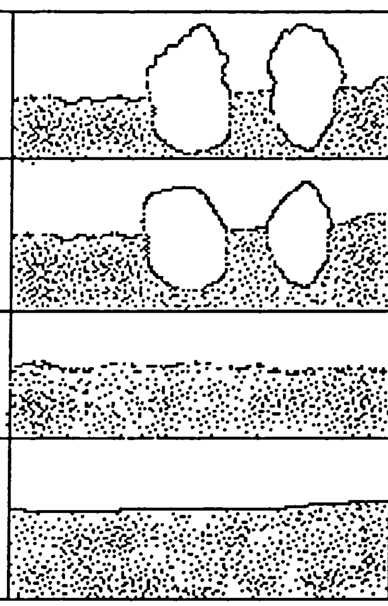
Fig. 4.21 Graph showing the relationship between LAAV and AAV of the quartz dolerites.

4.8 Polished stone value test

4.8.1 *Introduction*

Another type of wear in aggregates is polishing. This refers to the removal of the micro-texture (related to aggregate surface texture) and macro-texture (related to overall texture of the road, see Fig. 4.22) by the action of vehicle tyres, thus reducing the coefficient of friction between vehicle tyres and road surface. All aggregates used as roadstone need to have a high resistance to polishing by vehicle tyres. The polished stone value (PSV) gives a measure of the resistance of the aggregate to such road surface wearing action. It is one of the major determinants of road user safety, since a proper level of skid resistance improves safe driving conditions.

Some rocks such as moderately weathered dolerites are highly resistant to polishing because weathering processes transform existing rock forming minerals into an assemblage with some new mineral components with different hardness values, producing a harsher texture and thus increasing the resistance of the rock to polishing. The effect of amygdales in a rock depends upon their size and distribution throughout the rock and the relative hardness of the secondary infilling and the enclosing original minerals. Monomineralic limestones and those containing small insoluble residues tend to have a low resistance to polishing (Hartley 1974).



Surface	State of texture	
	Macro	Micro
A	rough	harsh
B	rough	polished
C	smooth	harsh
D	smooth	polished

Fig. 4.22 Texture scale used to describe an asphaltic pavement surface. (after Elsenar *et al.* 1976)

The British Standards Institution (BS 812:part 114:1989) recommends that the relative extent to which different aggregates will polish should be assessed by the laboratory determined polished stone value (PSV). In this test a sample of aggregate is subjected to accelerated polishing action achieved by attaching the prepared aggregate samples (four specimens from each rock sample) to a 400mm diameter wheel which is then made to bear against a 200mm diameter tyred wheel rotating at 315 - 325 revolution per minute. The abrasion action is accelerated by the introduction of a paste of water and corn emery and emery flour on to the aggregate specimen surface during the two test cycles, each of 3-hour duration. The former is fed during the first cycle and the latter during the second.

Polishing caused to the aggregate particles is measured in terms of the coefficient of friction between the specimen surface and a pendulum rubber slider. The pendulum is allowed to fall from a standard height and traverses the test specimen with its attached friction pad, both specimen and the rubber slider being wet. The greater the polish on the specimen, the less momentum is lost by the pendulum as it crosses the specimen and therefore the greater the polish imparted to the specimen the lower the value indicated on a calibration scale. The PSV is calculated using the following equation: $PSV = S + 52.5 - C$, where S is the mean value (on scale) of the four test specimens and C is the mean value of four control specimens.

High polished stone values tend to be associated with poor abrasion resistance and with low strength. Therefore it is difficult to find rocks with a combination of high strength and a high polished stone value for road surfacing. Work has been undertaken to investigate improvement of the PSV of artificial aggregate by heat treatment (Gutt and Hinkins 1972) and alteration of porosity by the addition of impurities (Hartley 1974); and a new DOE project will investigate the availability of high PSV natural aggregates in the UK.

The minimum acceptable value of polished stone value is dependent upon the situation in which the material is to be used and the amount of traffic (commercial vehicles per lane per day) using the road. Table 4.10 shows the minimum acceptable polished stone value in the UK for each site related to the amount of traffic using the road.

4.8.2 *Results and discussion*

The polished stone value tests were carried out in accordance with BS 812:1989 part 114. The mean and range of the PSV test results obtained are listed in Table 4.11. In this Table the highest polished stone value (63) is from Westcraigs quartz dolerite, with rocks which are coarser in grain size than the others and which have undergone some alteration and weathering causing changes in the rock constituents, by producing some new mineral products with different hardness values which give a harsher texture and consequently a higher resistance to polishing. Dahir and Meyer (1976) have shown that coarse grained rocks require a greater polishing effort than fine grained rocks of uniform composition. On the other hand the relatively fresh and finer grained quartz dolerite from Hillend possesses the lowest polished stone value (56) of the samples tested, although the other properties are better than all other quartz dolerites tested.

Table 4.10 Minimum polished stone values for material used in highway running surfaces in the UK. (Department of Transport 1990).

Site (see notes below)	Approximate percentage *of roads in England*	Traffic density^{x}	Min. PSV
A1	0.1	250	60
(difficult)		250 - 1000	65
		1000 - 1750	70
		1750	75
A2	4.0	1750	60
(difficult)		1750 - 2500	65
		2500 - 3250	70
		3250	75
B	15	1750	55
(average)		1750 - 4000	60
		4000	65
C (easy)	85	-	45

^{x} in number of commercial vehicles per lane per day.

A1 sites include:

i. Approaches to traffic signals on roads with 85 percentile speed of traffic greater than 64km/h.
ii. Approaches to traffic signals, pedestrian crossings and similar hazards on main roads.

A2 sites include:

i. Approaches to and across major priority junctions on roads carrying more than 250 commercial vehicles per day.
ii. Roundabouts and their approaches.
iii. Bends with radius less than 150m on roads with an 85 percentile speed of traffic greater than 64m/h.
iv. Gradients of 5 per cent or steeper, longer than 100m.

B sites include: Generally straight sections of and large radius curves on:

i. Motorways.
ii. Trunk and principle roads.
iii. other roads carrying more than 250 commercial vehicles per lane per day.

C sites include:

i. Generally straight sections of roads carrying less than 250 commercial vehicles per lane per day.
ii. Other roads where wet skidding accident are unlikely to be a problem.

NB The PSV of coarse aggregate in rolled asphalt and dense tar surfacing, having coated chippings applied to the surface, should not be less than 45 for sites in categories A1, A2 and B. There is no limit for sites in C.

This tends to confirm the previous statement that high PSV tends to be associated with poor abrasion resistance (low AAV) and with low strength (high AIV and ACV). For correlation purposes the aggregate abrasion and impact value results are shown also in Table 4.11. In Figure 4.23 polished stone values are plotted against the relevant aggregate abrasion values and show that high polished stone value results are associated with poor (high) abrasion values. In Figure 4.24, where PSV results are plotted against AIV results, the relationship has a correlation coefficient of 0.90, and here again high PSV results are associated with high (poor) AIV results. These two Figures illustrate the difficulty in finding rocks with a combination of high strength, abrasion resistance and high polished stone value.

Table 4.11 The mean and range of the PSV test results of the quartz dolerites.

Site	PSV			AAV	AIV
	no of tests	range	mean		
Caldercruix	4	54 - 60	58	5.1	8.65
Duntilland	5	56 - 63	59	5.4	9.31
Westcraigs	5	59 - 65	63	8.0	17.1
Cairneyhill	5	53 - 59	57	5.0	7.88
Hillend	5	54 - 57	56	4.7	7.26
Tam's Loup 1	4	56 - 61	59	5.6	11.5
Tam's Loup 2	4	60 - 64	61	7.4	14.0
Boards	5	55 - 60	58	5.0	8.00

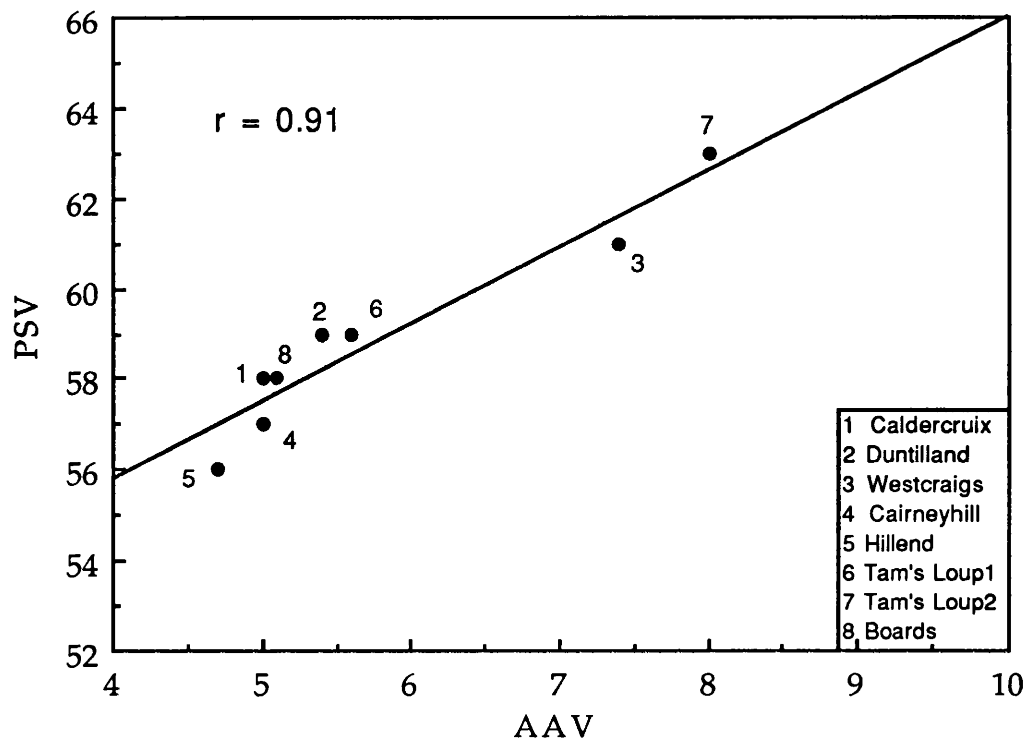


Fig. 4.23 Graph showing the relationship between PSV and AAV of the quartz dolerites.

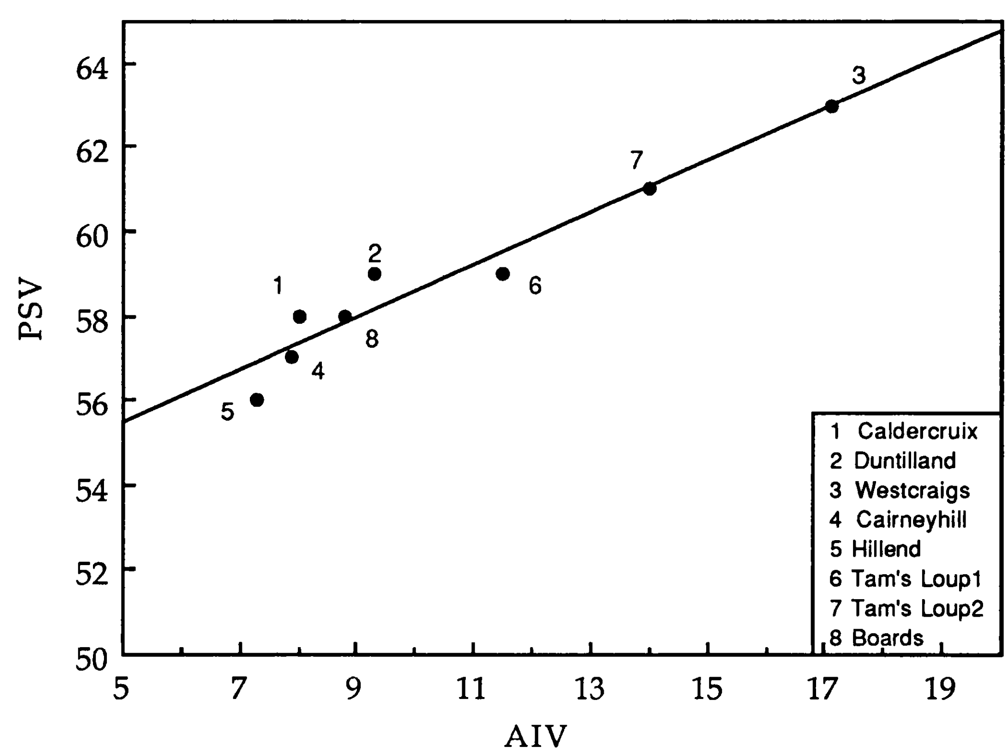


Fig. 4.24 Graph showing the relationship between PSV and AIV of the quartz dolerites.

4.9 Soundness test

4.9.1 *Introduction*

Soundness is a general term given to the ability of aggregate to resist excessive change in volume as a result of changes in physical conditions. Accelerated weathering experiments have been applied to certain engineering problems, particularly in relation to aggregates where knowledge of the weatherability of rock is highly desirable. Several cases of premature failure of aggregate have been documented (e.g., Wylde 1976).

Certain engineering tests have been developed which attempt to assess the resistance of rock to weathering, or to detect potentially unsound rock. Immersion cycles in sodium or magnesium sulphate solutions are the standard tests in Britain, United States and Australia. The slaked durability test has been developed to assess the durability of rock to wetting and drying cycles (Franklin & Chandra 1972). These tests usually attempt to simulate aspects of physical weathering.

In the soundness test samples of graded aggregate are subjected to alternate cycles of immersion in a saturated solution of magnesium sulphate followed by oven drying. Crystallisation of the magnesium sulphate salt in the pores and cavities of aggregates may set up bursting pressure and cause deterioration of the aggregate. The information obtained from this test is thought to be a qualitative indicator of the soundness of aggregates subjected to weathering action, freeze and thaw action etc. Hosking and Tubey (1969) employed the sulphate soundness test in conjunction with other standard and modified mechanical tests and suggested an arbitrary limit of 20% as the maximum acceptable loss in the weight of sample in soundness test.

4.9.2 *Factors affecting the test results*

a- *Aggregate size and shape*:- A decrease in size of the aggregate particles under test would increase the percentage of material lost in the test, because fine aggregate particles are easier to breakdown due to the pressure created by salt crystallising in the pore spaces of pieces of aggregate. Flaky aggregates are more prone to disintegration than cuboidal ones, thus aggregate shape is an important factor in this test.

b- *Aggregate water absorption (porosity)* :- Salts from the magnesium or sodium solution tend to precipitate in the pores and cavities of the aggregates and cause internal breakdown, especially in aggregates with higher water absorption values. Kazi and Al-Mansour (1980) have shown that the sulphate soundness results vary linearly with the water absorption values.

c- *Weathering*:- Weinert (1964) used sodium sulphate solution and established a link between the soundness of dolerite and the secondary minerals created by weathering or alteration. Fattohi (1973) mentioned that the percentage of aggregate loss increases in proportion to the degree of weathering of the rock, which can be measured by the amount of secondary minerals in the rock.

d- *Temperature of Solution*:- a change in the temperature of the solution leads to a change in the density of the solution, which eventually leads to the precipitation of dissolved salts and a reduction in the efficiency of the solution.

4.9.3 *Results and discussion*

The BS 812:1989 part 121 for the soundness test has been adopted using aggregate fractions of 20 - 14, 14 - 10 and 10 - 5mm sizes of each quartz dolerite investigated in this study. The aggregates are immersed in the saturated solution for 18 hours, and air dried for at least four hours, before drying in an oven at 105±5°C for 24h. At the end of each cycle the aggregate is cooled before being immersed again in the saturated solution. After five cycles the aggregate is washed thoroughly in warm water to ensure the removal of all the sulphate salts. At the end of the complete test the samples are dried, sieved and weighed, and the soundness value calculated from the following equation:

$$S = 100 \times M2 / M1$$

where S = soundness value

M1 = initial mass of the test specimen

M2 = mass of test specimen at the end of the test

The results given in Table 4.12 clearly show that the percentage of material lost from the tested aggregate increases as aggregate size decreases. The soundness values (S) varies between 92.5 - 99.1 for 20 - 14mm, 89.0 -

99.1 for 14 - 10mm and 85.0 - 96.3 for 10 - 5mm aggregate size. The results indicate that Westcraigs quartz dolerites is the least resistant to breakdown by saturated magnesium sulphate solution (*S* = 89.0 for 14 - 10mm size), whereas, Cairneyhill quartz dolerite is the most resistant material among all the quartz dolerites tested in this study (*S* = 99.1 for 14 - 10mm size).

The soundness test results showed some variations in the different quartz dolerites. These variations may be attributed to a difference in grain size and the state of weathering of each quartz dolerite rock. Coarse grained and weathered rocks (Westcraigs and Tam's Loup 2) have higher water absorption values (Table 4.1) which may indicate a higher porosity with internal cracks which would provide easy access for the magnesium sulphate solutions to enter and precipitate their salts. This will exert pressure on the surrounding grains (whose bonding is already weakened by weathering) and consequently cause disintegration of the rock resulting in a higher percentage loss and a lower soundness value.

Table 4.12 Mean and range values of soundness test results of the quartz dolerites, (I_F = 25%).

Site		20 - 14mm	14 - 10mm	10 - 5mm
Caldercruix	mean	99.1	98.7	96.0
	range	(98 - 99.8)	(95.1 - 99.3)	(92 - 98)
	no of samples	3		
Duntilland	mean	98.6	97.5	96.0
	range	(96 - 99.1)	(92 - 99.7)	(93 - 98)
	no of samples	4		
Westcraigs	mean	92.5	89.0	85.0
	range	(90 - 95)	(87 - 91)	(82 - 87)
	no of samples	3		
Cairneyhill	mean	99.6	99.1	96.3
	range	(97 - 100)	(96 - 100)	(96 - 99)
	no of samples	5		
Hillend	mean	99.1	97.0	96.1
	range	(98 - 100)	(97 - 99.7)	(96-98.9)
	no of samples	4		
Tam's Loup 1	mean	98.1	96.5	94.9
	range	(94 - 99)	(91 - 98)	(89- 96.1)
	no of samples	5		
Tam's Loup 2	mean	96.1	91.0	87.0
	range	(90 - 97)	(88 - 92)	(86 - 90)
	no of samples	4		
Boards	mean	99.1	98.5	96.3
	range	(97 - 100)	(96 - 99.3)	(95 - 98)
	no of samples	5		

The influence of aggregate shape on the soundness test results was investigated by testing samples with different flakiness index values. These investigations were carried out only on the 14 - 10mm aggregate size from Duntilland, Cairneyhill and Tam's Loup 2.

The results (Table 4.13) show that the soundness values (*S*) decrease, i.e., greater percentage loss, with increasing flakiness index values. Figure 4.25 shows the relationship between the flakiness index value and the soundness value. The general trend suggests that the higher the flakiness index value the lower the soundness value (lower resistance to disintegration). Thus it is clear that aggregate shape plays a major role in the resistance of an aggregate to the disintegration by magnesium sulphate solution.

Table 4.13 Soundness test results for different flakiness index value.

Site	Soundness (14 - 10mm size)		
	$I_F = 0$	$I_F = 25$	$I_F = 80$
Duntilland	98.0	97.5	92.6
Cairneyhill	99.6	99.1	96.3
Tam's Loup 2	96.1	91.0	87.0

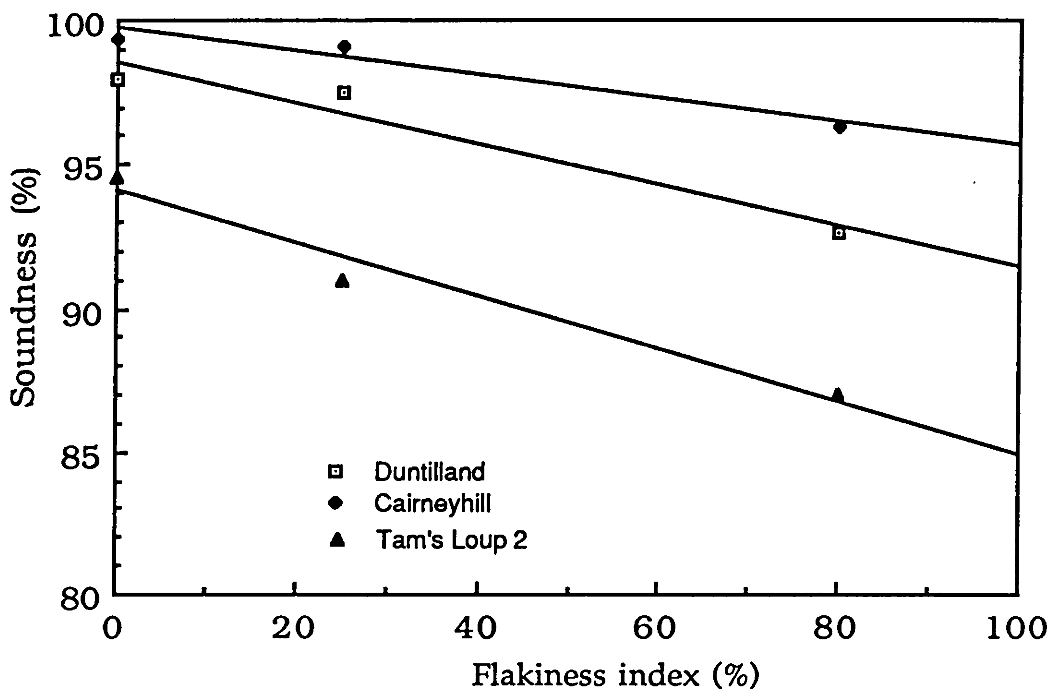


Fig. 4.25 Graph showing the relationship between soundness value and flakiness index value of the quartz dolerites.

4.10 Quartz dolerite aggregate used in concrete

4.10.1 *Introduction*

Concrete is a material which consists essentially of aggregate held in a cement and sand paste. Aggregate used in concrete may comprise more than 50% of the total volume of the concrete and is therefore the major constituent which largely affects the strength, durability and appearance of the concrete. In properly made concrete each particle of aggregate is completely coated with cement paste and all of the space between aggregate particles is completely filled with paste. The essential requirements for aggregates in concrete are that their characteristics do not adversely affect the performance of the concrete in either the fresh or hardened state. The aggregate specification must be relevant to the available resource and to the characteristics required of the concrete in which the aggregate will be used.

4.10.2 *Factors affecting concrete quality*

1. Aggregate

1.1 *Type and character* :- The type of aggregate used considerably influences the compressive strength of concrete. As a general rule uncrushed coarse aggregate (naturally reduced in size) makes a concrete with a lower strength than one with crushed coarse aggregate (Teychenne 1978). The aggregate characteristics have a major effect on the bond between aggregate and cement paste in hardened concrete.

1.2 *Size* :- The larger the aggregate particle the smaller the surface area to be wetted per unit weight. Thus extending the grading of aggregate to a larger maximum size lowers the water requirement of the mix, so that for a specified workability and richness, the water/cement ratio can be lowered with a consequent increase in strength. However, there have been indications (Neville 1983) that the improvement in the properties of concrete that an increase in the size of aggregate brings, does not extend beyond about 37.5mm. Perhaps because of the lower bond area and discontinuities introduced by the largest particles.

The largest size of aggregate permitted in BS 882 part 1201 for use in concrete in the United Kingdom is 40mm, whereas in Europe the maximum permitted size of coarse aggregates is 32mm, with the sand essentially composed of material of 4mm and less. However, the maximum size of aggregate usually employed in concrete is 20mm since consideration has to be given to the cover and bar spacing in reinforced concrete. Coarse aggregates with a maximum size greater than 40mm are sometimes specified for massive placements, the aim usually being to minimise the cement content and to reduce heat generation.

Smaller aggregate, usually of 10mm maximum size, may be required to be placed through congested reinforcement in concrete, in which case the cement content may have to be increased by 10 - 20% to get the same strength and workability as is produced with a 20mm maximum sized aggregate concrete, because the fine aggregate and water content normally have to be increased to produce a cohesive mix.

1.3 *Grading* :- The percentage by weight of aggregate passing various sieve sizes is called the grading of the aggregate. The use of appropriately proportioned coarse and fine aggregate gradings, each within their relevant envelopes, normally produces concretes with adequate properties in the fresh and hardened state. In general, using continuously graded aggregates tends to produce concretes which are relatively straightforward and easy to place, whereas concretes made with gap-graded aggregates usually require appreciably more expertise in mix design, production, handling and placing (Schaffler 1979).

1.4 *Shape and surface texture* :- In the case of crushed aggregates the particle shape depends on the nature of the parent material and on the type of crusher and its reduction ratio (the ratio of the size of material fed to the crusher to that of the finished product). The shape of coarse aggregate can have an adverse influence on the workability of concrete and may cause blockages in pump mixes due to particle interference. Experiments have shown that the workability of concrete as measured by the compacting factor is reduced by 10% when changing from a rounded to an angular shaped particle (Kaplan 1958). The requirements in the BS 882 part 1201 are that the flakiness index of the combined coarse aggregate shall not exceed 50 per cent for uncrushed gravel and 40 per cent for crushed rocks. An aggregate with a rough surface will have a better bond with the cement paste and the concrete made with it will be stronger. Surface texture is generally more important in relation to concrete flexural strengths, which are frequently found to decrease with increasing particle smoothness. The classification in BS 882 part 1201 is generalised and the specific exclusion of material of any particular surface texture is virtually unknown.

1.5 *Cleanness* :- Aggregate used in concrete should be clean and free from organic material, since aggregate containing such material makes poor concrete (BCA 1990). To achieve proper bonding of the material, aggregate should be free from a coating of dust or clay. Limits on the amount of clay, silt and dust are given in BS 882 part 1201:1983.

2. Water/cement ratio

The character of concrete is largely determined by the water/cement ratio; the lower the water the higher the concrete strength, but workability needs enough water, because the workability of concrete depends primarily on the water content of the mix. The water/cement ratio of a concrete mix is particularly important when a range of aggregates having different water absorptions are used. Research on crushed aggregates (Teychenne 1978), has shown that materials from different sources may require widely different water contents to produce concrete of the same workability. Such changes in water content change the water/cement ratio and hence the strength of the concrete.

3. Curing method.

Curing is the name given to procedures used for promoting the hydration of cement, and consists of the control both of temperature and of moisture movement from and into the concrete. Curing conditions largely affect the strength of concrete. The development of strength of concrete made with all types of Portland cement depends mainly on the cement and curing conditions (temperature and humidity conditions during curing).

Water cured concrete is stronger than air cured. Teychenne (1978) found that after 28 days the average strength of air stored concrete was 84 per cent of that of water stored concrete.

4. Type of cement

Different types of cement produce concrete having different rates of strength development. Although most concrete is made with ordinary or rapid hardening Portland cement there are other cements that will produce concretes with more specialised properties. The various types of cement and their properties are listed below.

4.1. *Ordinary Portland cement* :- This is the most used cement, it develops strength sufficiently rapidly for most concrete work. Its resistance to attack by sulphates is generally low.

4.2. *Rapid-hardening Portland cement* :- After setting it develops strength more rapidly than the ordinary Portland cement. Framework can therefore be struck earlier and this type of cement is useful in cold weather (BS 12:1978).

4.3. *High alumina cement* :- This type differs in its manufacture, chemical composition and characteristics from the Portland cements described above. Concrete made with this type of cement develops high strength very early, allowing it to be brought into early use, and this may be useful for temporary works. A substantial reduction in concrete strength may subsequently occur, associated with chemical conversion (Teychenne 1975).

4.4. *Sulphate-resisting Portland cement* :- This type of cement has a modified chemical composition compared with ordinary Portland cement so as to provide resistance to attack by sulphate solutions (BS 4027:1980). It is low-alkali cement which will be beneficial in situations where an alkali silica reaction may occur.

4.5. *Ultra-high early strength cement* :- This has a higher proportion of gypsum than ordinary Portland cement. The initial development of concrete strength is much more rapid than that of ordinary or rapid hardening Portland cements but there is little increase in strength after 28 days.

4.6. *Low heat Portland cement* :- This type of cement develops strength more slowly than ordinary Portland cement, particularly at low temperatures, but its final strength is not lower. It is intended for use when large volumes of concrete are required to reduce the temperature rise in the concrete and the possibility of subsequent cracking (BS 1370:1979).

4.10.3 Concrete tests

1. Compressive strength test

The crushing strength of concrete is probably the most important concrete parameter since it is the one most generally used to define the quality of concrete (BS 5328:1981). The strength of concrete is usually dependent upon the strength of the cement and sand paste, and on the bond between this paste and the aggregate, rather than on the strength of the aggregate alone. The cement aggregate bond is influenced by the cleanliness and the surface texture of the aggregate, and is frequently found to decrease with increasing particle smoothness. Generally, when the bond is good, a crushed concrete specimen should contain some aggregate particles broken right through, in addition to the more numerous ones pulled out from their sockets. However, an excess of fractured particles could suggest that the aggregate is too weak.

Results and discussion

Concrete compressive strength tests were carried out in accordance with BS 1881:1983. Three cubes of 100mm edge were prepared from each sample being tested. After 28 days curing in a water tank the cubes were compressed at an approximate rate of 0.3 N/mm^2 per second until total failure was reached and the load at failure of each cube was recorded. The compressive strength of each cube was calculated by dividing the load at failure by the cross section area. The results were expressed to the nearest 0.5 N/mm^2. The average of the results of the three cubes made from each sample was taken as the compressive strength of the concrete for that particular sample.

In order to examine the performance of concrete cubes made with aggregate from different quartz dolerite sites, all the factors in the mix design were kept constant for all the tested samples. The particular mix design used in the present study is given in Table 4.14, based on BRE (1988).

The mean and range of the compressive strength of concrete cubes made from aggregate samples from each site are given in Table 4.15, together with all other concrete properties investigated. The highest compressive strength was shown by cubes made with Hillend aggregate ($72N/mm^2$), whereas the lowest compressive strength of $43N/mm^2$ was shown by cubes using the Westcraigs quartz dolerite; and this difference can be attributed to grain size and the state of weathering of the Westcraigs quartz dolerite. It was observed that the amount of broken aggregate in the crushed cubes made from Westcraigs quartz dolerite was relatively high, which also indicates that these aggregates are weaker than the Hillend quartz dolerites.

The relationship between the physical and mechanical properties of the aggregate and the compressive strength of concrete made with the same aggregate is demonstrated in Figures 4.26 and 4.27. In Figure 4.26 the concrete compressive strength results are plotted against aggregate crushing value, and the graph shows that higher concrete compressive strengths are associated with lower crushing values. This relationship has a correlation coefficient of 0.90. Figure 4.27 shows the relationship between concrete compressive strength values and the 10% fines values, and this plot shows that aggregate with higher 10% fines value produced concrete cubes with higher strengths.

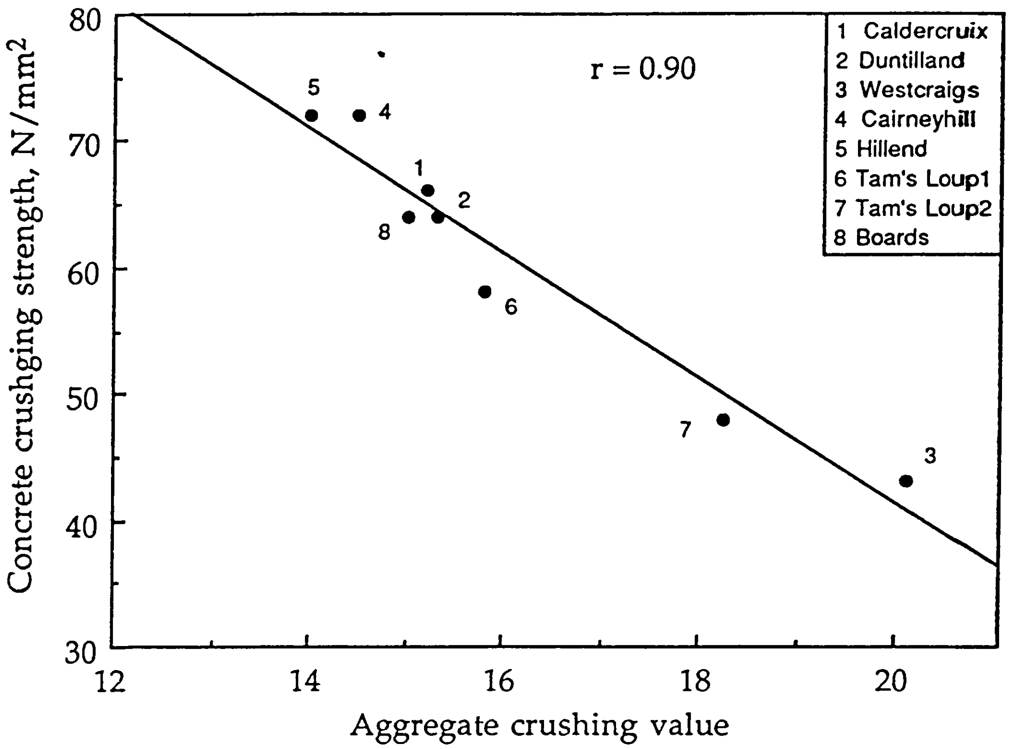


Fig. 4.26 Relationship between concrete compressive strength and aggregate crushing value of the quartz dolerites.

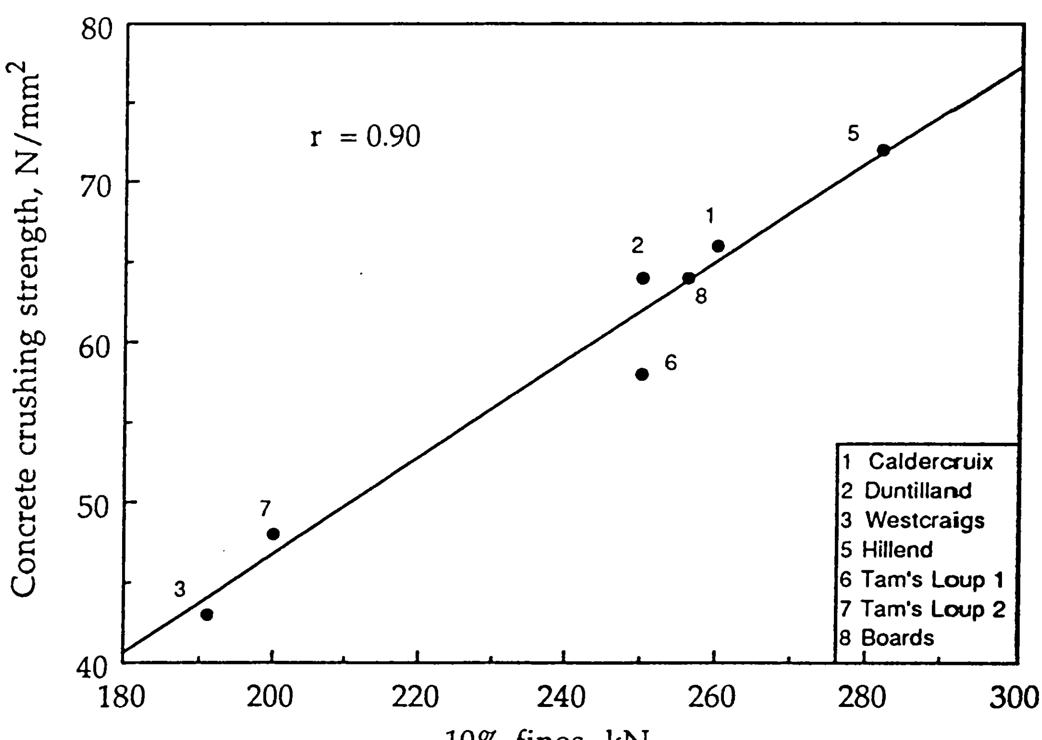


Fig. 4.27 Relationship between concrete compressive strength and 10% fines value of the quartz dolerites.

2. Drying shrinkage test.

Some aggregates change volume considerably from the wet to the dry state and this may affect the concrete in which they are incorporated. Rock types that exhibit excessive shrinkage characteristics are found to contain appreciable amounts of sheet silicates such as clays, chlorites, micas etc. (Collis and Fox 1985). Some aggregates from rocks such as weathered basalts, weathered dolerites and mudstones suffer from significant moisture movement; expanding when wet, and shrinking as they dry out. However, natural aggregates from fresh rocks normally do not shrink sufficiently for this to be considered as a pertinent factor in the behaviour of the concrete (Fookes 1980).

Rocks which shrink usually have a high water absorption, and this can be treated as a warning sign that the aggregate should be carefully investigated for its shrinkage properties. Shrinkage of aggregate in concrete, particularly the coarse aggregate, may lead to cracking or crazing of the concrete, thereby increasing the vulnerability of that concrete to deterioration by other agencies such as frost action. The drying shrinkage of a concrete containing such aggregate can be four times greater than that of a concrete made with non-shrinkable aggregates.

Excessive concrete shrinkage has caused severe problems in South Africa (Roper 1960), Scotland (Snowdon and Edwards 1962), Australia (Cole *et al.* 1981), and Italy (Barsione 1984). Cole (1979) has found that undesirable rocks with very large drying shrinkages may pass the standard, BS, tests for aggregates. Therefore drying shrinkage was investigated as a criterion to define stable rocks. Edwards (1967, 1970) showed a relationship between drying shrinkage and the water absorption of the aggregate but the graph had a poor correlation coefficient as the points showed considerable scatter.

Roper (1960) mentioned that aggregates which give rise to excessive shrinkage in concrete contain clay minerals. Clay minerals occur in three forms:

a) as matrix materials of syngenetic origin in the case of sandstones, conglomerates and argillaceous sediments;
b) as alteration products due to the weathering in the vadoze zone of all types of rocks; and.
c) as minerals formed under hydrothermal conditions in the case of basic igneous rocks.

In all the above occurrences the clays form an integral part of the aggregate as opposed to surface coatings or clay balls found in gravel deposits. These coatings and balls may also increase shrinkage considerably, since the clay functions like a gel instead of as a restraining body. In the presence of dissolved calcium hydroxide clay particles are likely to be strongly flocculated and distributed as floes through the paste. Then, after the paste has hardened and drying shrinkage has started the clay floes will shrink more than the cement gel.

However, Rae (1971) mentioned that some aggregates did not contain montmorillonite or any other similar swelling clay, yet showed dimensional instability with variation in relative humidity. This suggested that an expansive crystal lattice is not prerequisite for dimensional instability, but that forces outside the crystal lattice may also be significant.

Carlson (1935) made an attempt to reduce shrinkage by adding small amounts of ground fresh mica to concrete mixes, and he did obtain substantial reductions in shrinkage values. However, the addition of mica may reduce the tensile strength of the mortar.

To determine the drying shrinkage of the aggregate under test, a concrete mix is prepared and cast into prisms of specified dimensions (200mm x 50mm x 50mm). The prisms are subjected to wetting followed by drying at 105°C and the change in length from the wet to the dry state determined. The drying shrinkage of the aggregate is then calculated as the average change in length of the prisms as a percentage of their final dry length.

The BS 812 part 120:1989 gives categories of use of aggregate with different shrinkage values:

Category A includes aggregates with drying shrinkage value up to and including 0.075 per cent, which can be used for all concrete purposes.

Category B includes aggregates with drying shrinkage values exceeding 0.075 per cent, which can be used in positions where complete drying out never occurs; namely mass concrete surfaced with air entrained concrete, heavily reinforced concrete, none of which should be exposed to the weather.

The same British standard also recommends the minimum testing frequency; aggregates with shrinkage value from 0 to 0.05 per cent should be tested once every 5 years, and aggregates with shrinkage values greater than 0.05 per cent should be tested annually.

Results and discussion

In the present study BS 812 part 120:1989 was adopted to assess the drying shrinkage of quartz dolerite aggregates in concrete. The fine aggregate was natural sand from a particular source which has a low shrinkage. This sand will therefore have a minimal affect upon the shrinkage value of the concrete made from the aggregate from the quartz dolerites.

Three prisms were prepared from each sample being tested for drying shrinkage. Ball-bearings were cemented on to the ends of the prisms after compaction was completed (in order to obtain accurate measurements). 48hrs after this, the prisms were immersed in water at a temperature of 20±1°C for 5 days. The prisms were then removed from the water and the length (w) of each prism measured. The prisms were heated at 105±5°C for 3 days, removed from the oven, and allowed to cool down to 20±1°C, and the length (d) of each prism measured. The length (l) of each prism alone also measured (length without measuring ball-bearings).

The drying shrinkage (Sh) of each prism was calculated as a percentage from the following equation: $Sh = (w - d / l) \times 100$. The drying shrinkage of each sample was taken as the average shrinkage value of the three prisms prepared from that particular sample.

The ranges and means of the drying shrinkage values obtained in this study are given in Table 4.15. The drying shrinkage values range from 0.049% to 0.065%. The results show that weathered samples (Westcraigs) have high shrinkage values whereas fresher samples from the Caldercruix site have the lowest value. It is possible that secondary minerals in the aggregate are the cause of high shrinkage values in the more weathered rocks.

In Figure 4.28 the mean values of the drying shrinkage results obtained at each site are plotted against aggregate water absorption and the relationship has a correlation coefficient of 0.86. The plot suggests that high shrinkage values are associated with high water absorption values.

The present study reveals that altered and weathered samples give higher concrete shrinkage values. This may be because the normal rock forming minerals become unstable during weathering conditions and alter to sheet silicates with possibly expansive crystal lattices. Feldspars in the altered and weathered samples were largely sericitised, and the ferromagnesian minerals were partly or completely altered to chlorite or serpentine. The development of secondary sheet silicates probably increases the porosity of the rock, enabling water to inter the rock and eventually into the sheet silicates causing their expansion, and shrinkage when they dry out. Thus, the relationship between water absorption and shrinkage of aggregate is proportional; the higher the water absorption the higher the shrinkage.

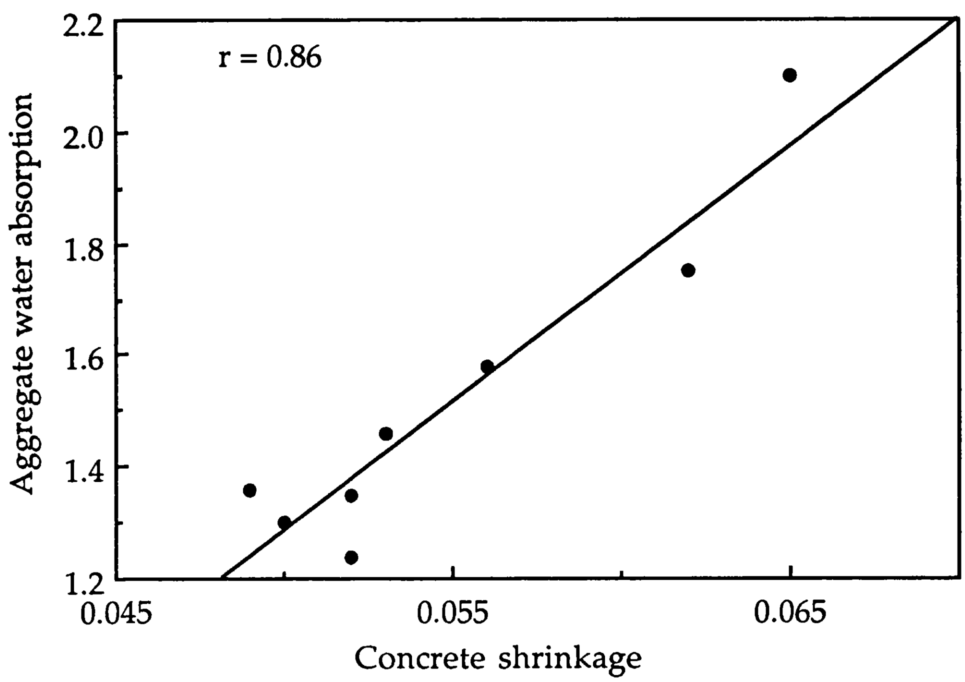


Fig. 4.28 Relationship between concrete shrinkage and water absorption of the quartz dolerites.

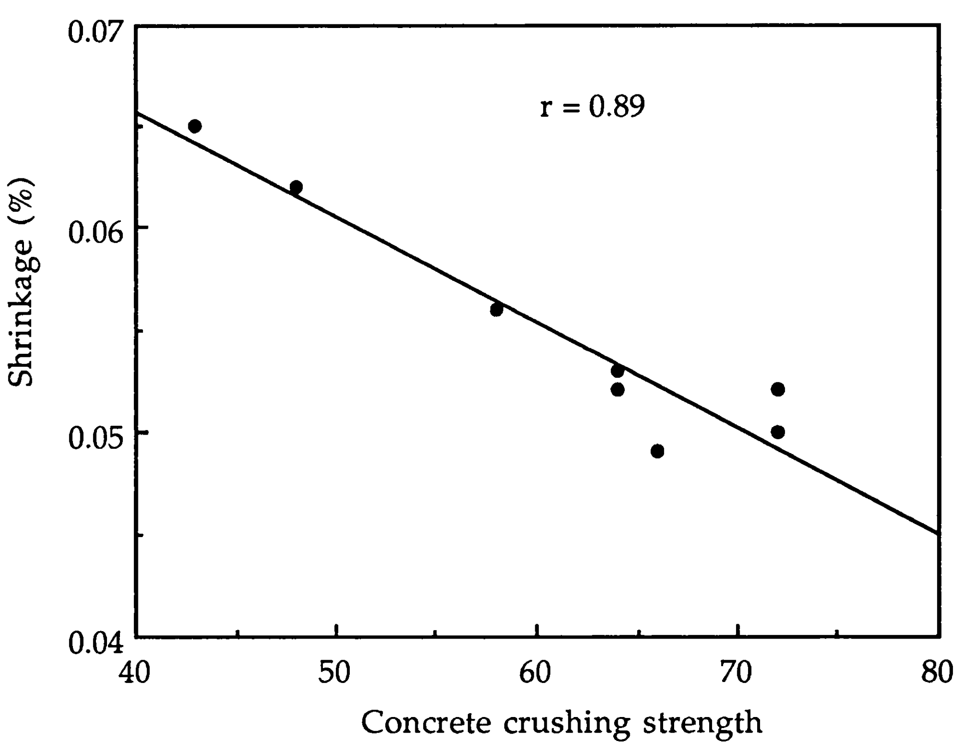


Fig. 4.29 Plot of concrete crushing strength against concrete shrinkage of the quartz dolerites.

Table 4.14 Concrete mix design used for the present study (based on BRE, 1988)

Stage	Item	Reference or calculation	Values
1	1.1 Characteristic strength	Specified	60 N/mm² at 28 days; Proportion defective ____ %
	1.2 Standard deviation	Fig 3	____ N/mm² or no data ____ N/mm²
	1.3 Margin	C1 or Specified	(k = ____) ____ × ____ = ____ N/mm²; ____ N/mm²
	1.4 Target mean strength	C2	____ + ____ = 60 N/mm²
	1.5 Cement type	Specified	OPC/~~SRPC~~/~~RHPC~~
	1.6 Aggregate type: coarse Aggregate type: fine		Crushed/~~uncrushed~~ ~~Crushed~~/uncrushed
	1.7 Free-water/cement ratio	Table 2, Fig 4	0.43
	1.8 *Maximum free-water/cement ratio*	*Specified*	____ Use the lower value $\boxed{0.43}$
2	2.1 Slump or Vebe time	Specified	Slump 10–30 mm or Vebe time ____ s
	2.2 Maximum aggregate size	Specified	20 mm
	2.3 Free-water content	Table 3	2/3 (160) + 1/3 (190) $\boxed{170}$ kg/m³
3	3.1 Cement content	C3	170 ÷ 0.43 = 395.5 kg/m³
	3.2 *Maximum cement content*	*Specified*	/ kg/m³
	3.3 *Minimum cement content*	*Specified*	/ kg/m³
			use 3.1 if ≤ 3.2 use 3.3 if > 3.1 $\boxed{395.5}$ kg/m³
	3.4 Modified free-water/cement ratio		$\boxed{/}$
4	4.1 Relative density of aggregate (SSD)		2.90 known/~~assumed~~
	4.2 Concrete density	Fig 5	2575 kg/m³
	4.3 Total aggregate content	C4	2575 − 395.5 − 170 = 2009.5 kg/m³
5	5.1 Grading of fine aggregate	Percentage passing 600 μm sieve	40 %
	5.2 Proportion of fine aggregate	Fig 6	35 %
	5.3 Fine aggregate content	C5	2009.5 × 0.35 = $\boxed{703.33}$ kg/m³
	5.4 Coarse aggregate content	C5	2009.5 − 703.3 = $\boxed{1306.2}$ kg/m³

Quantities	Cement (kg)	Water (kg or L)	Fine aggregate (kg)	Coarse aggregate (kg) 10 mm	20 mm	40 mm
per m³ (to nearest 5 kg)	395	170	705	435	870	
per trial mix of 0.003 m³	1.2	0.51	2.1	1.3	2.6	

Items in italics are optional limiting values that may be specified (see Section 7)

1 N/mm² = 1 MN/m² = 1 MPa (see footnote to Section 3).

OPC = ordinary Portland cement; SRPC = sulphate-resisting Portland cement; RHPC = rapid-hardening Portland cement.

Table 4.15 Mean and range values of compressive strength and shrinkage results of concrete made with the quartz dolerites.

Site		Compressive strength (N/mm2)	Shrinkage (%)
Caldercruix	mean	66	0.049
	range	(64 - 70)	(0.047 - 0.052)
	no. of samples	(4)	(3)
Duntilland	mean	64	0.053
	range	(61 - 66)	(0.051 - 0.057)
	no. of samples	(4)	(5)
Westcraigs	mean	43	0.065
	range	(41 - 46)	(0.061 - 0.068)
	no. of samples	(4)	(4)
Cairneyhill	mean	72	0.050
	range	(69 - 74)	(0.049 - 0.052)
	no. of samples	(3)	(4)
Hillend	mean	72	0.052
	range	(71 - 75)	(0.049 - 0.053)
	no. of samples	(4)	(4)
Tam's Loup 1	mean	58	0.056
	range	(55 - 61)	(0.053 - 0.060)
	no. of samples	(5)	(4)
Tam's Loup 2	mean	48	0.062
	range	(45 - 50)	(0.060 - 0.063)
	no. of samples	(4)	(3)
Boards	mean	64	0.052
	range	(60 - 66)	(0.050 - 0.054)
	no. of samples	(5)	(4)

3. Alkali aggregate reactions (AAR).

The alkali aggregate reaction is an expansion reaction which takes place between alkalis in the cement and certain rocks and minerals sometimes used as aggregates. The main external evidence of deterioration of concrete due to alkali aggregate reaction is cracking. It is important to remember that this form of cracking can also result from other disruptive forces within concrete, such as shrinkage. However, shrinkage cracking usually appears early in the life of a structure, probably in the first year. The earliest time at which cracking due to alkali aggregate reaction has been diagnosed in the UK is about five years (BRE 1982). If the reaction causes expansion (French and Poole 1976) damage can occur, and the presence of as little as 0.5 per cent of a defective aggregate may be sufficient to cause considerable damage. The maximum expansion is frequently found to be produced when reactive aggregate comprises up to 4 per cent of the total used aggregate.

The actual reactivity of aggregate is affected by the alkali present and its amount in the cement, aggregate particle size and porosity as these influence the area over which the reaction can take place, the availability of enough moisture in the paste and the permeability of the paste (Neville 1983). Gillott (1975) suggested that other factors such as temperature and the presence or absence of pozzolanic material admixture (such as pyrex glass or fly ash) may also be important, and the addition of 20 - 30% of suitable pozzolan may reduce or prevent expansion due to the alkali-silica reaction. Pozzolanic material reacts readily with the calcium hydroxide in the cement and water mix and reduces the alkali available for reaction with coarse aggregate. However, this may lead to an increase in the amount of water required for the mix. It is evident that the various physical and chemical factors make the alkali-aggregate reaction a highly complex one, and difficult to resolve.

Alkali-aggregate reactions can be classified into three groups; (I) the alkali-silica reaction, (II) the alkali-carbonate reaction and (III) alkali-silicate reaction.

I. *Alkali-silica reaction* :- In the alkali-silica reaction, aggregate expansion results from the formation of a silica gel (Gillott 1975) that continues to increase in volume as long as water is available. Crypto-crystalline silica types such as chalcedony, opal and chert, and acid and intermediate volcanic glasses are some of the expansive components commonly in the aggregate which can be considered under this category. Gogte (1973) pointed out that aggregates containing strained quartz (characterised by undulatory extinction) are more reactive towards cement alkalies than aggregates containing unstrained quartz, due to the presence of a large number of dislocated zones in crystalline silica formed during mechanical deformation of rocks.

II. *Alkali-carbonate reaction* :- Carbonate aggregates are not immune from alkali reaction problems. Commonly a reaction takes place between coarse argilliceous dolomitic limestone aggregates and cement alkalis in the presence of moisture. This type of alkali-aggregate reaction is similar to some extent to the first type. Similarities include the influence of the alkali content, the importance of moisture and the abnormal expansion of the concrete leading to map-cracking in unstrained concrete or to a crack pattern modified by the reinforcement in reinforced concrete. Important differences from the alkali-silica reaction are the absence of significant quantities of gel, the much greater expansion and the less certain effect of pozzolanic addition in controlling the reaction (BRE 1982).

III. *Alkali-silicate reaction* :- Rocks prone to this reaction are phyllites, some greywackes and argillites (Duncan *et al.* 1973; Gillott & Swenson 1973). These rocks commonly contain quartz, feldspar, rock fragments and clay minerals as essential constituents. The reaction mechanism is complicated and difficult to characterise, but it is frequently expansive (Gillott & Swenson 1973). Expandable phyllosilicates such as montmorillonite are sometimes present in the clay fraction of those rocks, and are at least partly responsible for expansion in alkali-aggregate reaction (Gillott 1975). In dealing with basaltic rocks Gogte (1973) concluded that basalts containing chalcedony or palagonite with some interstitial glass as common mineral constituents, may indicate deleterious reactivity in the alkali Portland cement environment.

Testing for alkali-aggregate reaction

Three tests commonly used to diagnose the potential alkali reactivity of the aggregate were developed in the United States. They are ASTM C227-86 (Mortar-bar Method), ASTM C289-86 (Chemical Method) and ASTM C586-86 (Rock Cylinder Method).

The aggregate in the ASTM C227-86 (Mortar-Bar) test in which the increase or decrease of mortar bar containing combination of cement and aggregate during storage under prescribed conditions. If the mortar bar expands more than 0.05 per cent after 6 months and more than 0.1 per cent after one year the aggregate is considered harmful. If only the 6 months limit is exceeded the aggregate is not considered to produce a harmful expansion (Neville 1983). This method has shown that no UK aggregates in known use are deleterious, including aggregates of the types involved in reported cases of alkali-silica reaction. This indicates that UK aggregates cannot be effectively classified by this method (Concrete Society 1987).

The ASTM C289-86 (Chemical Method) covers chemical determination of the potential reactivity of aggregates with alkali in Portland cement concrete as indicated by the amount of reaction during 24h at 80°C between sodium hydroxide solution and the aggregates. This test is usually classify aggregates containing cherts or flint as potentially deleterious. It is widely regarded as being too pessimistic, especially since the method does not assess the effects of varying the combination of coarse and fine aggregates (CS 1987).

The ASTM C586-86 (Rock Cylinder Method) covers the determination of the expansive characteristics of carbonate rocks. In this test a rock cylinder is immersed in a solution of sodium hydroxide (NaOH) at room temperature. The change in length occurring during such immersion indicates the general level of reactivity of rocks.

Although the rocks under examination in this study are unlikely to be reactive (CS 1987 & BCA 1988), some tests were carried out using the ASTM C227 - 86 Mortar-bar method. 600g of cement and 1350g of graded aggregate were used to make four specimens, the amount of water used was sufficient to produce a mix of a flow of 105% to 120%. The mix was then poured into moulds which were placed immediately in a moist room for 24 hours. The specimens were then removed from the moulds and measured for initial length (to the nearest 0.002mm). After this the specimens were put on end, over, but not in contact with, water in storage container for 12 days in a cabinet maintained at a temperature of 37±1°C. Before opening the container and making length measurements, the container and contents were stored at 23±1°C for at least 16h.

The difference between the initial length of the specimens of the concrete containing the quartz dolerites examined in this study after 3 months is zero, and no further measurements were made. Petrographic examinations were carried out on thin sections prepared from the specimens examined in this study, to see whether alkali-silica gel, microcracks and reactive aggregate are present or not. The results show no presence of the three characteristics of the alkali reactivity. However, some microcracks were seen in some thin sections particularly those prepared from weathered materials. It is thought that they were either primary in the aggregates or secondary caused by aggregate shrinkage. Thus the reactivity of concrete made with the dolerites in this study in three months is nil, and the dolerites can be used in all applications for concrete with a shrinkage value with range of 0.045 - 0.065.

4.11 Adherence to bitumen

4.11.1 *Introduction*

Aggregates used in road construction in general should be durable, strongn and resistant to the polishing action of traffic. A further property required especially for surface dressing and for macadam type mixes is that they should show good adhesion with bituminous binders (i.e., resistance to stripping). In the service life of a road-stone the greatest stress appears to be applied during the laying process when the coated surface dressing is rolled into the surface of the asphalt to provide the required skid-resistance to the wearing course. Thus, aggregate should have high strength to withstand the stress during the laying process and along its service life.

The resistance to stripping is dependent upon mineral composition, particle shape and surface texture, and cleanlliness of the aggregate. The influence of detailed mineralogy has been presented by Hughes *et al.* (1960) who showed that the state of detachment tends to occur most readily from aggregates containing high proportions of quartz and feldspar but less readily, or not at all, from aggregate containing high proportions of more basic components, particularly those containing iron, e.g., augite and olivine.

The traditional aversion to flaky and elongate particles in aggregates for bituminous construction materials arises chiefly out of the greater degradation they exhibit both in conventional aggregate tests and in service behaviour as surface dressing and open texture macadams. Maupan (1970) reported that a significally shorter life occurs for mixtures containing high proportions of flaky aggregate. However, flaky or low crushing strength aggregates can be used in dense bituminous mixes, because of the support afforded by the bituminous mortar and by the increased impermeability to water and air.

Dust inhibits adhesion, and the BS specifications limit the amount of material passing a 75μm sieve to 15 per cent in bituminous mixes. Insufficient drying and non removal of dust before coating are the most common causes of stripping when surface dressing comes under the action of traffic. The influence of the binder type is another factor, it follows that bituminous binders (which are slightly negatively charged) adhere better to positively charged rocks such as basic igneous, metamorphic rocks and limestones (Collis and Fox 1985).

Breakage occurs readily in high void content or single layer mixes because the aggregate particles are in a specially vulnerable situation, with little or no protection from the development of high stresses. When the mix is compacted or layed on the road it gives a carpet with a relatively high voids content. Under dry conditions there is no problem of adhesion between the aggregate and the binder in bitumen macadam but, because of the high void content, water may enter and remain in the carpet on the road and this can cause a modification in the state of adhesion which may lead to a loss of mechanical cohesion in the material. Rocks which tend to absorb water in preference to bitumen, with resulting stripping, are generally silicious, such as granite, quartzite and chert, whereas those which preferentially absorb bitumen are limestone, dolomite, basalt and dolerite.

This demonstrates that aggregate suitable for one function is not necessarily suitable for others. For example, quartzite and granite with their low shrinkage and high strength, though eminently less suitable as concrete aggregate, are obviously unsuitable as road aggregate. By using or pre-treating the aggregate with certain metal salts, e.g., zinc chloride, the detachment can be delayed considerably (Hughes *et al.* 1960). Depending upon the use of the mix, a wide variety of sizes and gradations of an aggregate can be used. In general the selection of aggregate for bituminous mixes is controlled by many factors, such as:

1- the type of usage (e.g., highways, airfields)
2- the bituminous material (e.g., surface dressing, rolled asphalt)
3- the intensity of traffic
4- the location of pavement (e.g., base course, wearing course)

4.11.2 *Testing*

Collis and Fox (1985) stated that in spite of all the work that has been done on adhesion testing and all the variations of test methods that exist, no completely satisfactory method appears to have been developed. Because of limited time for this study this test could not be carried out, but the widespread use of quartz dolerite aggregates as coated material in Central Scotland, including use in the wearing course layer, suggest that their adherence to bitumen is acceptable to the regional authority using them.

CHAPTER FIVE

CORRELATION OF RESULTS

CORRELATION of RESULTS

5.1 Introduction

The preceding chapters have investigated the different properties of the quartz dolerites of the Midland Valley of Scotland. These properties included petrography and geochemistry (Chapter 2), and engineering properties from both intact rock and aggregate (Chapters 3&4).

In this chapter the results of the examinations carried out in the previous chapters are correlated with each other in an attempt to determine the types of relationships that occur.

5.2 Micropetrographic index

5.2.1 *Introduction*

The petrographic features which can be expected to influence the engineering behaviour of a rock material are; mineral composition and alteration products; grain size; textural types and amount of microfractures; types of microfracture fillings; and anisotropy, if present.

Several authors such as Sabine *et al.* (1954), Lumb (1962) and Mendes *et al.* (1966) have attempted to specify some form of quantitative weathering index. For example, Lumb (1962) defined a quantitative index (X_d) related to the weight ratio of quartz and feldspar in residually decomposed granite of

Hong Kong. For fresh rock $X_d = 0$, and for completely decomposed rock $X_d = 1$. Weinert (1964) dealt mainly with basic igneous rocks, and classified stages of decomposition in terms of the percentage of secondary minerals; fresh rock had less than 15% secondary minerals, decomposed rock had 15% to 30%, and badly decomposed rock had more than 30%, and there was no lower limit for defining residual soil.

Mendes *et al.* (1966) defined an overall quality index which takes into account the mineral components of a rock, their alteration products, and other peculiarities with a marked influence on the mechanical behaviour of the rock. Good correlations were obtained between the micropetrographic modal analysis and the modules of elasticity. The authors considered that the method would prove to be useful to characterizing rock masses at selected points by means of a qualitative index.

Onadera *et al.* (1974) used the number and width of microcracks as an index of physical weathering of a granite and found a linear relationship between effective porosity (ρ_{cr}) and density of microcracks defined as:

$$\rho_{cr} = (\text{total of width of cracks / length of measuring line}) \times 100$$

They found that porosity increases as the density of the microcracks increases. The mechanical strength of granite decreases as ρ_{cr} increases from about 1.5 to 4 per cent.

5.2.2 *Results and discussion*

In the present study a micropetrographic index *Ip* was calculated from the results of the detailed micropetrographic modal analyses, using the point count technique, (Chapter 2). The mean value of volume percentage of sound and unsound constituents were determined and presented in Table 5.1, which also contains the mean values of the determination of *Ip*. Alteration products are, however, frequently very difficult to identify in thin sections. The micropetrographic index was calculated as follows:

$$Ip = \text{sound components} \,/\, \text{unsound components}.$$

where sound components are the primary minerals such as feldspar, pyroxene, ores and quartz and accessory minerals such as apatite, pyrite, and unsound components include secondary minerals such as sericite, chlorite, serpentine and carbonates, plus any cracks or voids in the rock. Table 5.1 shows that Westcraigs quartz dolerite has the lowest *Ip* value (2.97) which probably reflects the degree of weathering the rock has undergone. Westcraigs has also the highest amount of secondary minerals and cracks (representing the unsound component in equation 5.4), whereas Hillend quartz dolerite has the highest *Ip* (11.1) since it contains the lowest amount of unsound components (8.3%) and the highest amount of sound components (91.7%) of all the sites investigated in this study.

Table 5.1 Sound components, unsound components and micropetrographic indices for the quartz dolerites.

Site	SC(%)	UC(%)	*Ip*
Caldercruix	87.3	12.7	6.87
Duntilland	85.5	14.5	5.90
Westcraigs	74.8	25.2	2.97
Cairneyhill	90.6	9.40	9.64
Hillend	91.7	8.30	11.1
Tam's Loup 1	85.0	15.0	5.70
Tam's Loup 2	77.7	22.3	3.48
Boards	89.9	10.1	8.90

SC = sound components, UC = unsound components and *Ip* = micropetrographic index.

5.3 Correlations

The micropetrographic indices obtained in the previous section can be correlated with the results of tests carried out on the quartz dolerites, to see whether the amount of sound or unsound components have any influence upon the engineering properties of the intact rock or aggregate.

The *Ip* values have been plotted against mechanical properties of the quartz dolerites as intact rock and aggregate in a series of graphs (Figs. 5.1-5.22). The relationships show that the mechanical properties of the quartz dolerites are almost the same in the rocks with *Ip* between 5 and 11, i.e those rocks with secondary minerals of less than 16%. In this part of the graph the engineering quality of the rock is very good and a small increase in the amount of secondary minerals does not cause any major decline in the quality of the rock as the *Ip* decreases. The change in slope of the regression line appears to correspond with an *Ip* of about 5 (which would indicate an amount of secondary mineral of about 16%) below which value of *Ip* and engineering properties of the rock become seriously reduced.

The same can be said about the correlation between *Ip* and concrete properties (Figs. 5.7 and 5.8). The above minimum value of 5 for *Ip* is also confirmed in Figure 5.8 where the shrinkage values of concrete made with dolerite fit into the range of 0.045 - 0.057, but the two most weathered types fall outwith this range. However, all samples from the different quarries produced concrete cubes with crushing strengths of more than 37N/mm^2, and shrinkage values of less than 0.075%. According to the new BS 812:120 drying shrinkage test and the limits of the allowed shrinkage value in concrete construction, all of the dolerites tested produced concrete with shrinkage values below the upper limit (0.075) of the BS, with the dolerites from the Westcraigs and Tam's Loup 2 quarries having the highest (i.e., poorest) shrinkage values. Figure 5.9 shows the relationship between concrete strength and concrete shrinkage; high concrete crushing strength values are associated with low shrinkage values. The best fit line of this relationship has a correlation coefficient of 0.89.

The relationship between some of the engineering properties and the chemistry of the quartz dolerites is shown in Figures 5.10-5.13. In these plots the $CaO + Na_2O$ and Fe_2O_3/FeO are plotted against AIV and 10% fines value. It should be noted that weathering is accompanied by an increase in the oxidation index of the rock, given by the Fe_2O_3/FeO ratio. In fresh rocks it is less than 1 but when the value increases to over 1 then the engineering properties of the rock are considerably reduced, and a value of $Fe_2O_3/FeO \geq 1$ gives an indication of considerable weathering in the quartz dolerites.

Engineering properties of the Midland Valley quartz dolerites have been plotted against each other in order to provide further information on the inter-relationships of the tests and to see whether it is possible to predict the value of one property from that of another property so that a simple and quick test may be all that is required to determine engineering quality of an aggregate. Correlation coefficient index properties have been obtained by computer.

The effects of water absorption on Los Angeles abrasion value are given in Figure 5.14. It can be seen that the Los Angeles abrasion value increases (gets poorer) with increasing absorption (volume of pore space). The same effects were observed on aggregate impact value and aggregate crushing value; and the 10% fines value decreases as water absorption increases (Figs. 4.2, 4.7 and 4.13).

Since an increase in water absorption value represents an increase in weathering in the quartz dolerites, then the physical and mechanical properties of the dolerites all decrease systematically with an increase in weathering. The differences in test values for the wet and dry states are greater in weathered material than they are in fresh material (graph 5.15, 5.16, 5.17 and 5.21 show this well).

When the uniaxial compressive strength is plotted against the aggregate impact value (Fig. 5.15) an approximate linear correlation is observed with a relationship as follows:

$$S_c d = 265 - 10\ AIV \qquad r = 0.93$$

$$S_c s = 262 - 11\ AIV \qquad r = 0.92$$

A similar linear relationship is observed for the uniaxial compressive strength and aggregate crushing value (Fig. 5.16). The mathematical relationship is:

$$S_c d = 432 - 17\ ACV \qquad r = 0.95$$

$$S_c s = 442 - 18\ ACV \qquad r = 0.96$$

A greater degree of correlation occurs between uniaxial compressive strength and aggregate crushing value than between uniaxial compressive strength and aggregate impact value and this results from the fact that the uniaxial compressive strength and aggregate crushing value are both static strength tests while the aggregate impact value is a measurement of the rock's resistance to dynamic impact loading. Hawkins and McConnell (1991) found similar relationships between sandstones of different ages. However Dhir *et al.* (1971) considered that the correlation of aggregate impact and crushing values with uniaxial compressive strength is poor.

There is also a linear relationship between point load strength index and sonic velocity. The best fit line in Figure 5.17 gives a very high correlation coefficient as follows:

$$Vpd = 1159 + 500\ I_s d \qquad r = 0.96$$

$$Vp\,s = 2858 + 341\,I_s s \qquad r = 0.94$$

A linear relationship exists between Schmidt hammer rebound numbers and their related uniaxial compressive strength values (Fig. 5.18). The rebound number can help to differentiate between rocks, and provide a means of estimating rock strength by correlating the rebound numbers with either unconfined compressive strength or point load indices (Fig. 3.15).

$$S_c d = -45 + 3.95\,R_n(\text{lab}) \qquad r = 0.90$$

The point load strength index has been widely used to estimate the uniaxial compressive strength of rocks, in the field and laboratory. The linear relationships for dry and saturated specimens are given in Figure 5.19, and the following equation applies:

$$S_c d = 22\,I_s d \qquad r = 0.96$$

Point load strength index I_s is plotted against modulus of elasticity E_s in Figure 5.20, and the best fit line of the relationship between these two parameters has a correlation coefficient of 0.95. The value of E_s may be estimated from the following formula:

$$E_s d = -12.83 + 6.15\,I_s d$$

The relationship between P-wave velocity and uniaxial compressive strength is obtained in Figure 5.21 with correlation coefficients of 0.92 and 0.93 for saturated and dry samples respectively; high velocities are associated with high strength values. As the values of the P-wave velocity and the uniaxial compressive strength increase, the difference between the results of dry and saturated samples decreases. This shows that rocks with high P-wave velocity also possess high strengths. Such a graph can be used to predict one property if the other is known, which may be particularly useful since the sonic velocity test is non destructive and may be carried out on non standard sized specimens.

$$S_c d = -39.2 + 0.04\ Vpd \qquad r = 0.93$$

$$S_c s = -134 + 0.05\ Vps \qquad r = 0.92$$

The correlation between the Los Angeles abrasion test results and Schmidt rebound number is illustrated in Figure 5.22. This Figure shows a very good correlation between these two parameters and the best fit line has correlation coefficient of 0.91. This shows that the Schmidt rebound number could be used to predict the Los Angeles abrasion test results. The usefulness of the relationship lies in the simplicity and rapidity of performing the Schmidt hammer test.

Some other correlations between the different physical and engineering properties of the quartz dolerites of the Midland Valley were already carried out in the Chapters 3&4 (e.g., Fig. 3.9 where S_c is plotted against E_d, Fig. 3.12 where E_s is plotted against E_d, Fig. 4.15 where ACV is plotted against 10% fines value, Fig. 4.19 where AIV is plotted against LAAV, and Fig. 4.20 where LAAV is plotted against ACV etc.). The good correlations between engineering properties of the quartz dolerites in this study suggest that, provided one or more BS engineering test properties is known, other properties can be predicted with reasonable accuracy using the graphs provided. Furthermore, the *Ip* index has shown that values of less than 5 indicate an altered rock which should be treated with caution, as the engineering properties are considerably reduced below this *Ip* value. Beside secondary minerals another factor which appears to affect the engineering properties of the quartz dolerites, is grain size. For example quartz dolerites of Duntilland and Tam's Loup 1 quarries have very similar *Ip* values, but their grain sizes are different with dolerite from Duntilland quarry being stronger than that from Tam's Loup 1 quarry, due to its finer grain size (Table 2.1).

Fig. 5.1 Relationship between micropetrographic index and water absorption.

Fig. 5.2 Relationship between micropetrographic index and aggregate impact value.

Fig. 5.3 Relationship between micropetrographic index and aggregate crushing value.

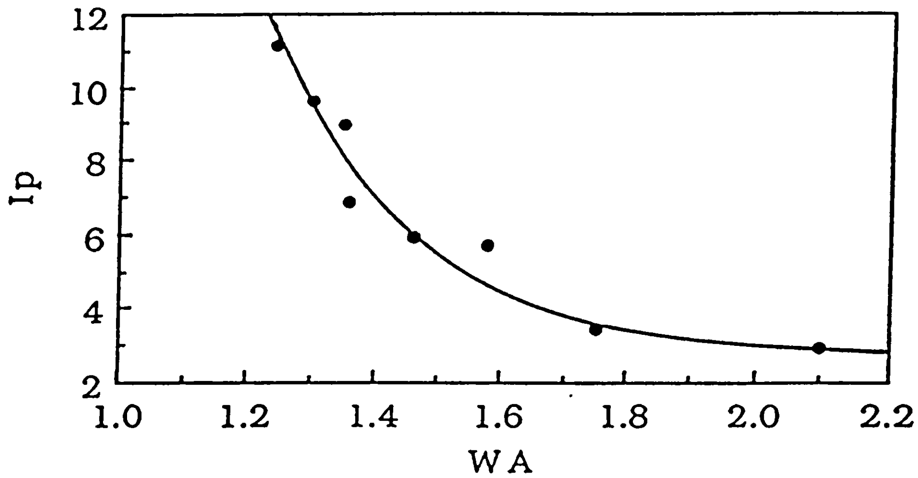


Fig. 5.1

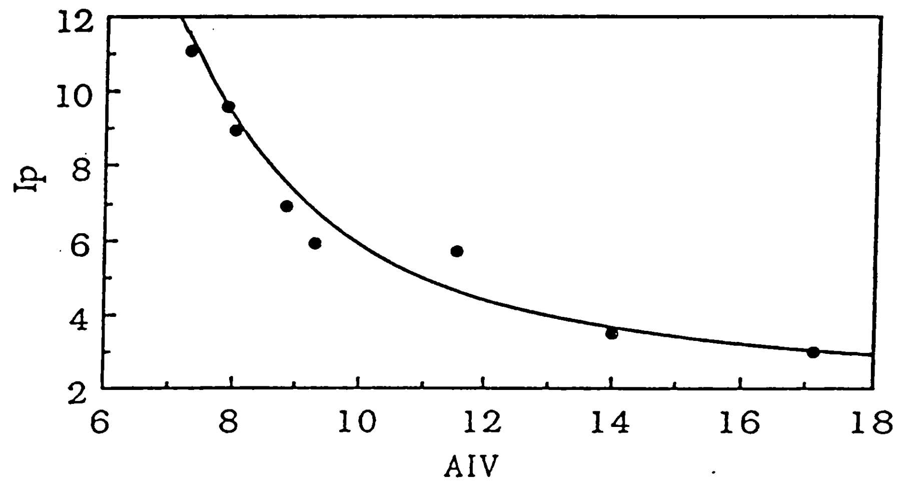


Fig. 5.2

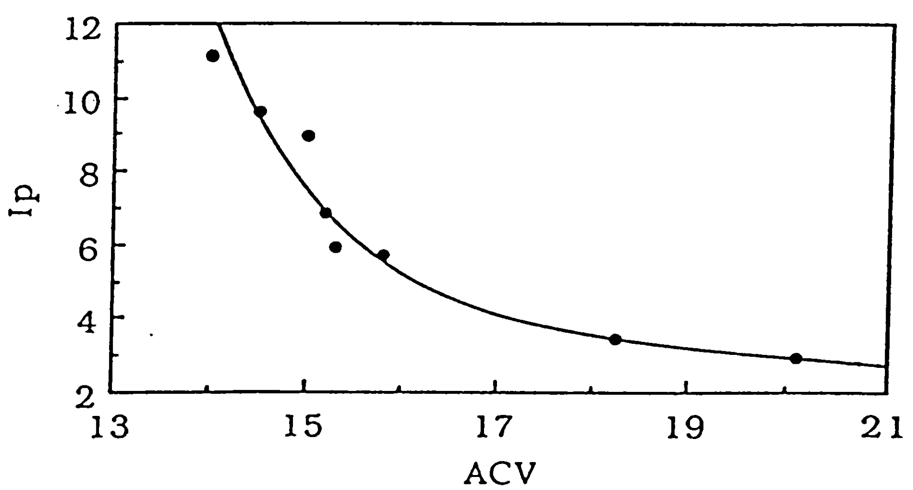


Fig. 5.3

Fig. 5.4 Relationship between micropetrographic index and uniaxial compressive strength.

Fig. 5.5 Relationship between micropetrographic index and P-wave velocity.

Fig. 5.6 Relationship between micropetrographic index and Schmidt hammer rebound number.

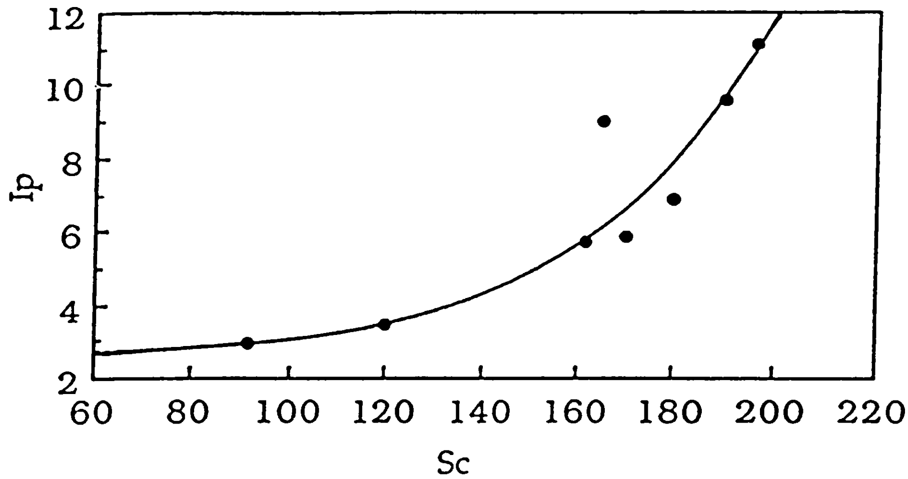


Fig. 5.4

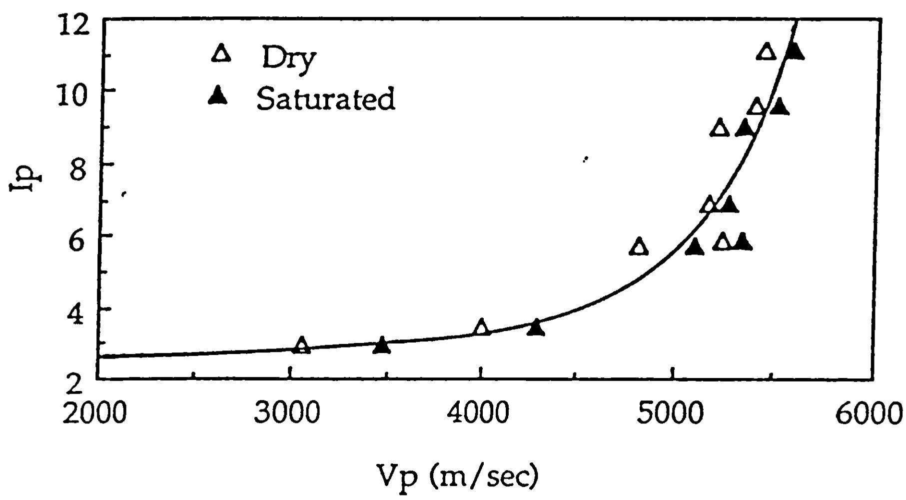


Fig. 5.5

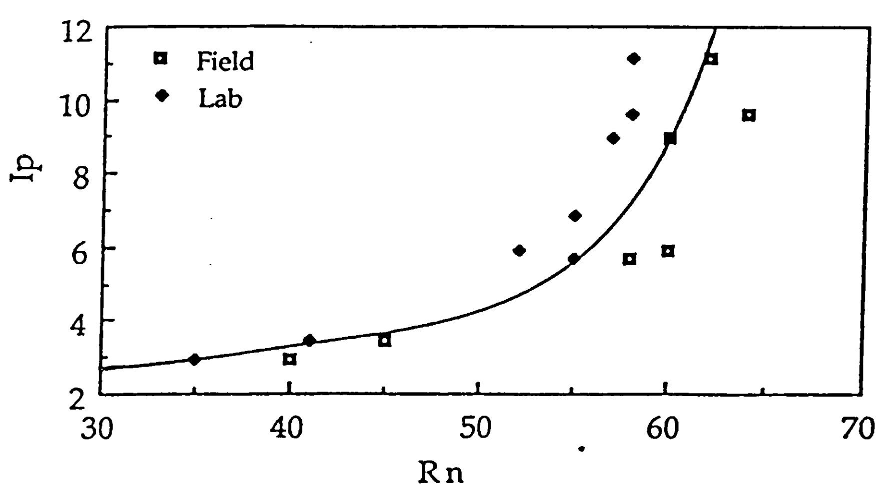


Fig. 5.6

Fig. 5.7 Relationship between micropetrographic index and concrete crushing strength.

Fig. 5.8 Relationship between micropetrographic index and concrete shrinkage.

Fig. 5.9 Relationship between concrete crushing strength and concrete shrinkage

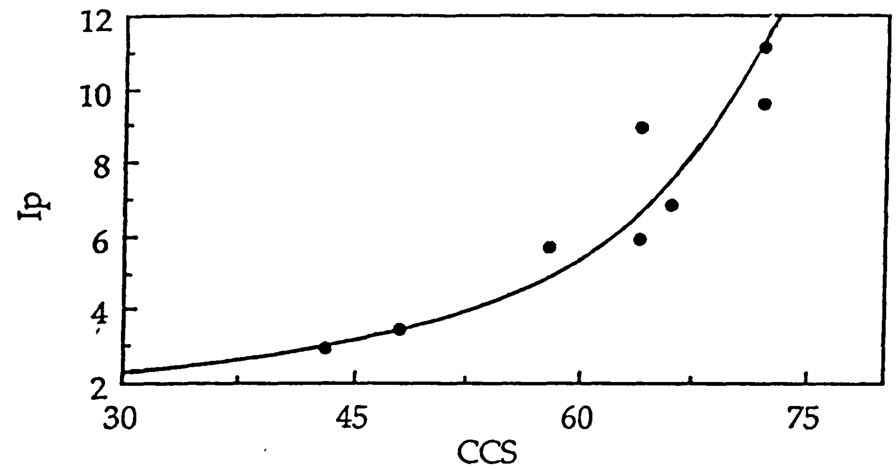


Fig. 5.7

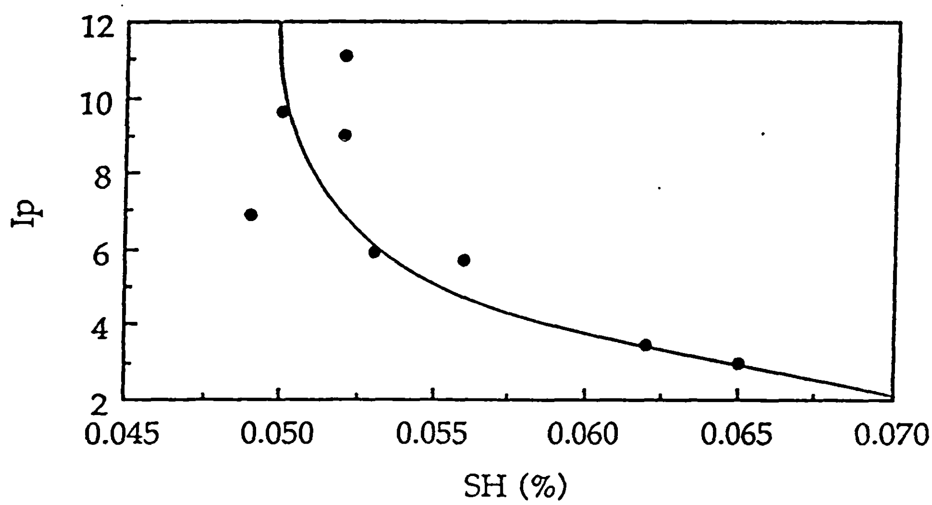


Fig. 5.8

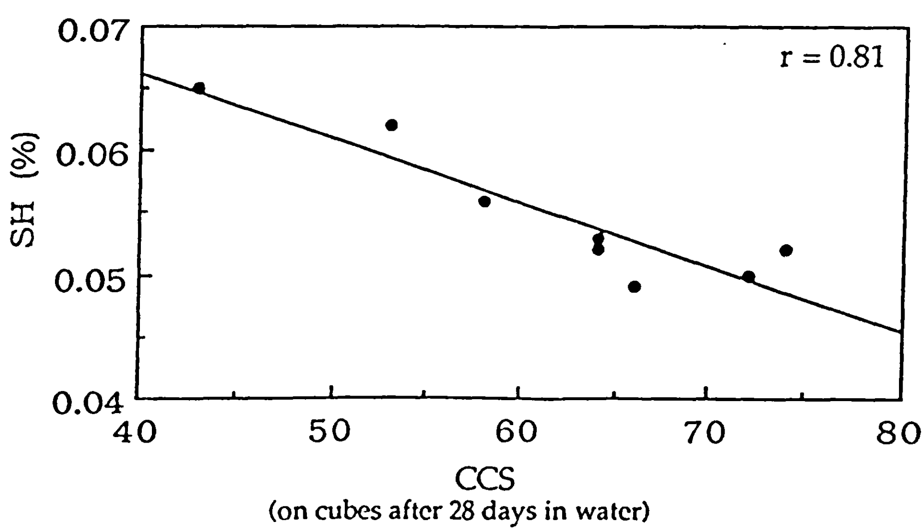


Fig. 5.9

Fig. 5.10 Relationship between Fe_2O_3/FeO and aggregate impact value.

Fig. 5.11 Relationship between $CaO+Na_2O$ and aggregate impact value.

Fig. 5.12 Relationship between Fe_2O_3/FeO and 10% fines value.

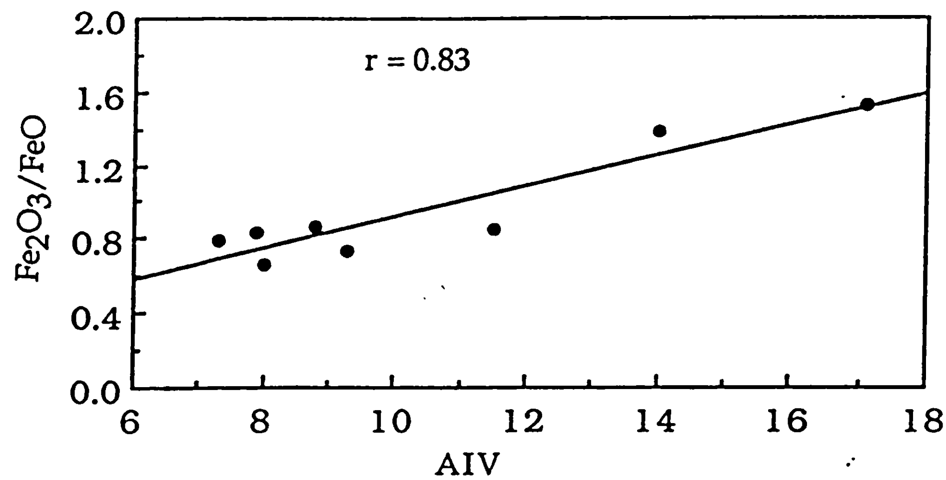


Fig. 5.10

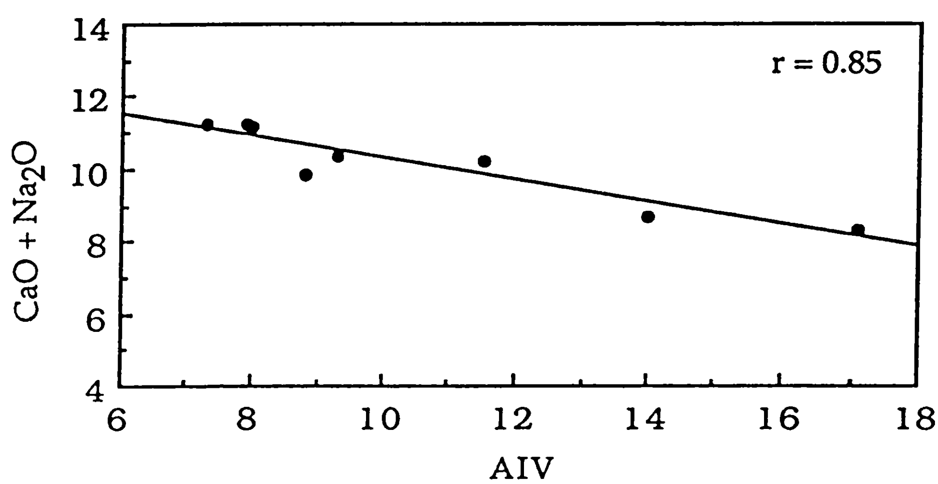


Fig. 5.11

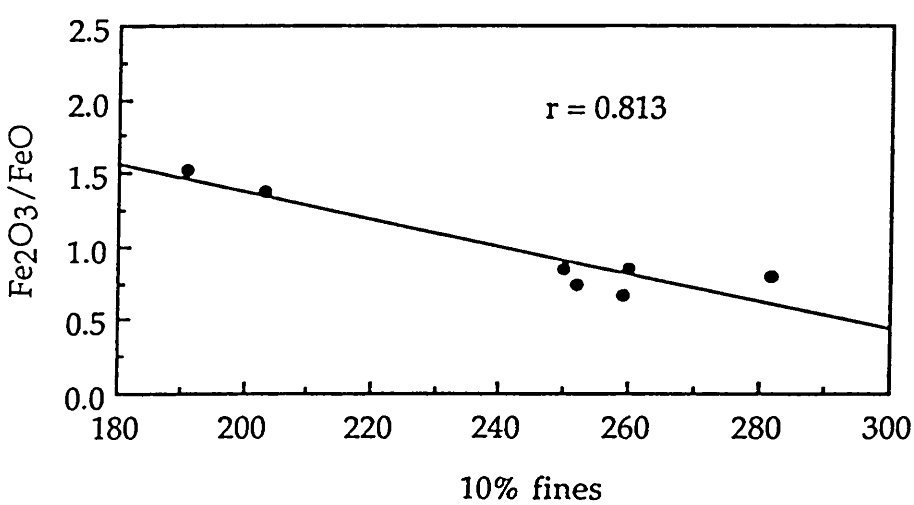


Fig. 5.12

Fig. 5.13 Relationship between $CaO+Na_2O$ and 10% fines value.

Fig. 5.14 Relationship between Los Angeles abrasion value and water absorption.

Fig. 5.15 Relationship between uniaxial compressive strength and aggregate impact value.

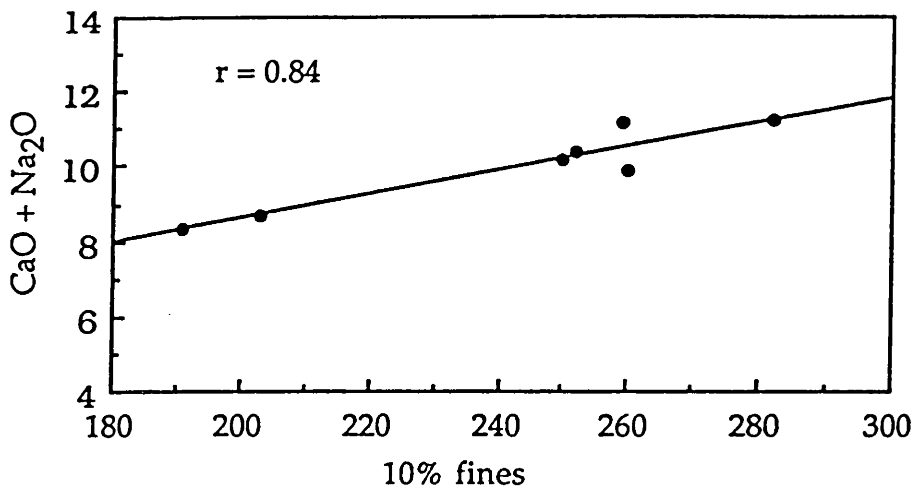


Fig. 5.13

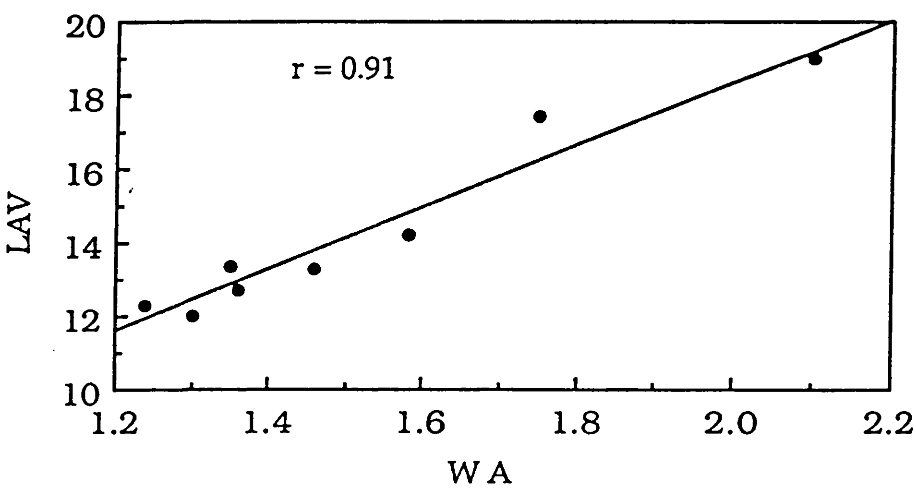


Fig. 5.14

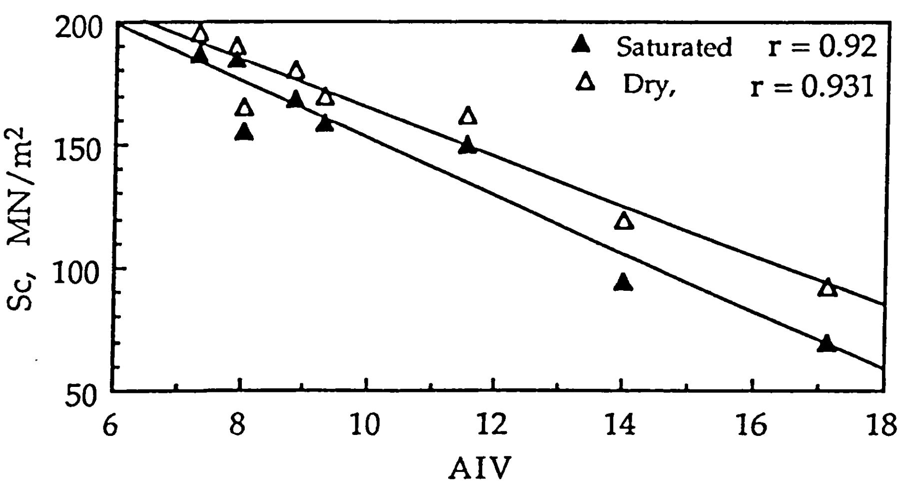


Fig. 5.15

Fig. 5.16 Relationship between uniaxial compressive strength and aggregate crushing value.

Fig. 5.17 Relationship between P-wave velocity and point load index.

Fig. 5.18 Relationship between uniaxial compressive strength and Schmidt hammer rebound number.

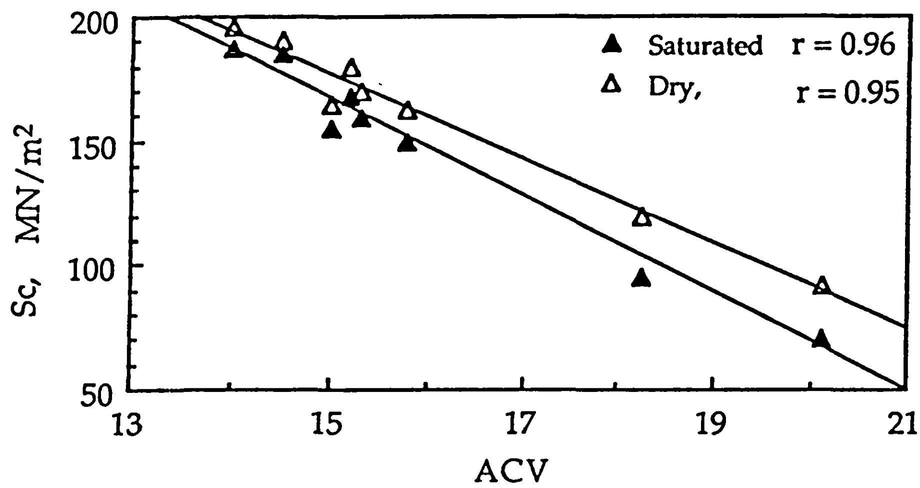


Fig. 5.16

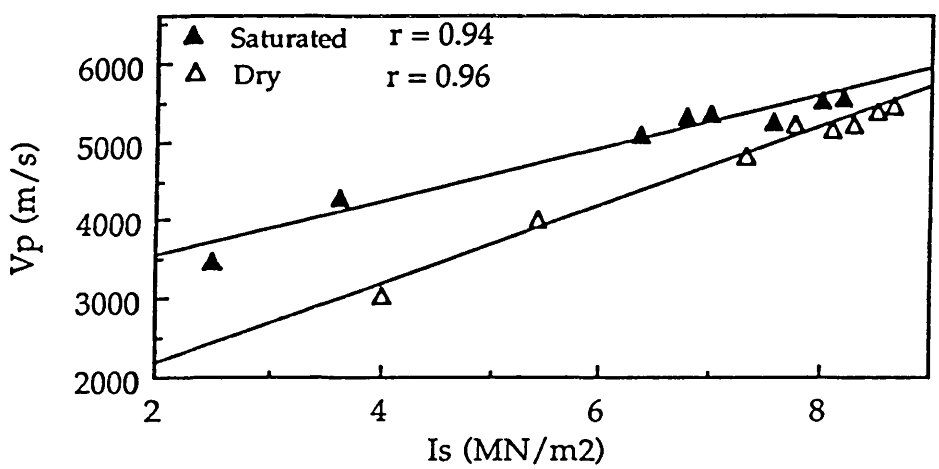


Fig. 5.17

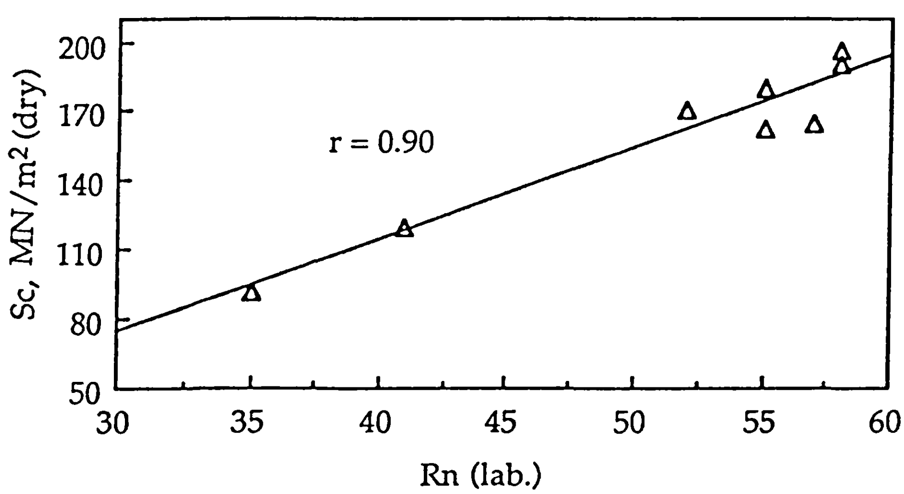


Fig. 5.18

Fig. 5.19 Relationship between uniaxial compressive strength and point load index.

Fig. 5.20 Relationship between static modulus of elasticity and point load index.

Fig. 5.21 Relationship between uniaxial compressive strength and point load index.

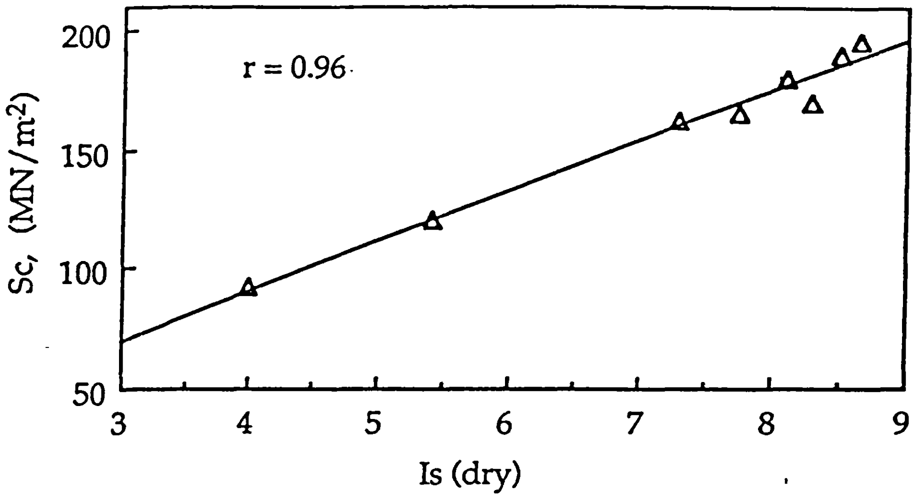


Fig. 5.19

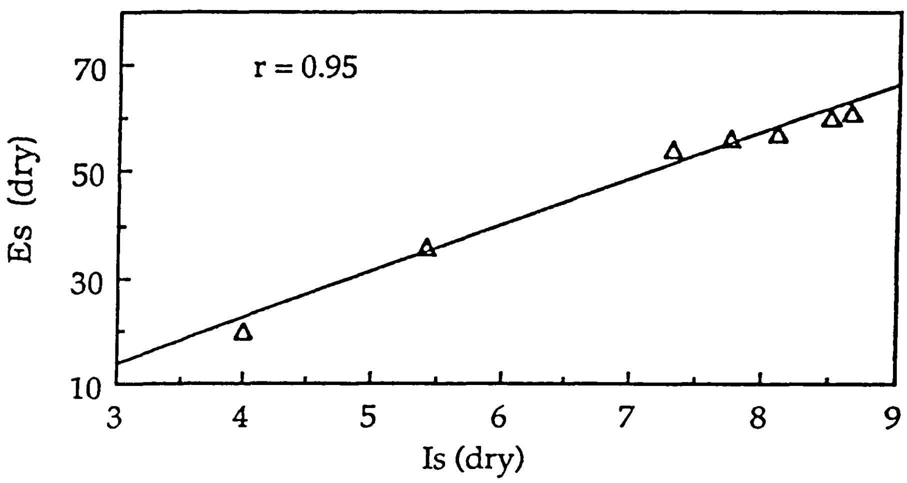


Fig. 5.20

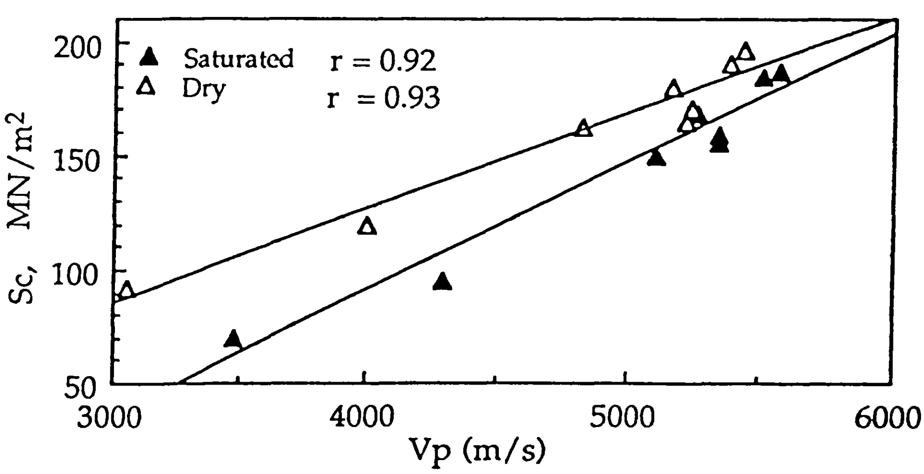


Fig. 5.21

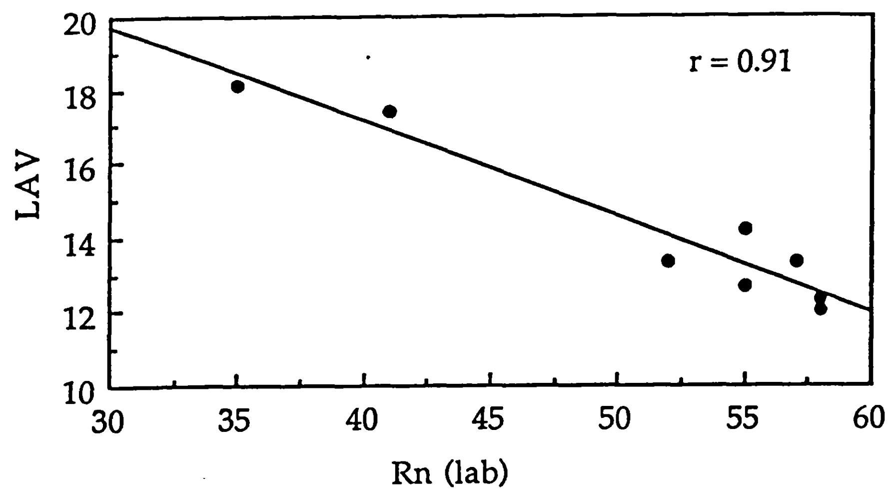


Fig. 5.22 Relationship between Los Angeles abrasion value and Schmidt hammer rebound number.

CHAPTER SIX

CONCLUSIONS

CONCLUSIONS

Quartz dolerite intrusions provide most of the rock material used for construction purposes in the Midland Valley area, and, since a detailed understanding of the different engineering properties of this rock is important, the present investigation has concentrated on the following studies:-

1. Petrographical and geochemical studies of the quartz dolerite.

2. Examination of the engineering properties of the intact rock and also its aggregate.

3. Correlation between the different engineering properties to establish any correlations and inter-relationships upon which the likely behaviour of one property can be predicted by study of the other.

The following points are the main conclusions of the present study:

1. The Midland Valley quartz dolerites consist essentially of plagioclase laths, sub-ophitic augite, iron oxides and an intersertal mesostasis of quartz and alkali feldspar. Apatite and pyrite are usual accessories. Alteration products include sericite, chlorite, calcite and serpentinous material.

2. Aplite veins are a minor rock type occurring within the dolerite sequences

and are common throughout the Midland Valley dolerites. The aplite may represent the final acidic magmatic liquid injected as a vein complex into the consolidated quartz dolerite sheets. However, their presence probably has a negligible effect on the quality of the dolerites.

3. The top part of the central zone (between top and bottom chilled marginal zones) in thick sills (>60m) is coarser in grain size and more prone to alteration.

4. Alteration in the altered quartz dolerite rocks has transformed minerals or part of them to softer material such as calcite which in some cases would be carried away by chemical agents leaving voids and spaces which weakened bonds between minerals and subsequently the rock.

5. Chemical analyses of the quartz dolerites show the remarkable chemical similarity of all relatively fresh dolerites at the same site or from different sites.

6. The chemistry of fresh dolerites and altered dolerites shows some differences. Altered quartz dolerites show a decrease in SiO_2, FeO, CaO and Na_2O, and a relative increase in MgO, Fe_2O_3, K_2O, Al_2O_3 and H_2O. The degree of chemical alteration that the quartz dolerite may have undergone is reflected more by its mineral composition than by its chemical analysis.

7. The Fe_2O_3/FeO ratio is a good measure of the amount of oxidation that the quartz dolerites have undergone during weathering. The Fe_2O_3/FeO ratio has increased considerably from 0.79 in fresh rock to 1.5 in the most

altered rock, but is still always less than the 150% of the original value, suggested by Moore and Gribble (1980) as indicating aggregate too weathered to be used as roadstone.

8. Various engineering tests were applied to assess the quality of the quartz dolerites examined in this study both as intact rock and aggregate. The following are the main conclusions:

(i) As the alteration in the minerals of the quartz dolerite increases the amount of water able to be absorbed by the rock also increases. Similarly the apparent density of the quartz dolerites decreases as the alteration of a rock increases. The void ratio also increases, probably due the partial dissolution of its components.

(ii) Differences in grain size, porosity and degree of alteration, in the quartz dolerites usually produce differences in physical and mechanical properties, even in rocks which are otherwise similar.

(iii) The fine-grained and relatively fresh dolerites from Hillend and Cairneyhill quarries give the best engineering test results and are much stronger than the coarse-grained and altered quartz dolerites from Westcraigs and Tam's Loup 2 quarries. However, the PSV properties of the dolerites have been enhanced by the alteration of the primary minerals in the rocks from Westcraigs and Tam's Loup 2 quarries.

(iv) Graphing different test results shows that the quartz dolerites exhibit a consistency in their behaviour under most tests; dolerites which show good results in one test show the same relative result in another test. The exception is the PSV test in which rocks of low quality under other tests showed good results under this test (high PSV). Plotting different tests results against each other also suggests that some properties of the quartz dolerite can be predicted from other properties, e.g., uniaxial compressive strength may be predicted by using P-wave velocity results or the point load strength index, and the Los Angeles abrasion value may be predicted by using the Schmidt hammer rebound number. Furthermore, the dynamic modulus of elasticity (determined from P-wave velocity and density results) is shown to be as reliable as the more expensive static test. The usefulness of these relationships lies in the simplicity, ease and rapidity of performing some tests, like the Schmidt hammer test for example. Thus for routine work it is not necessary to carry out all tests, merely those tests that will give a good and accurate indication of the quality of the rock.

(v) The results show that the quality of the quartz dolerite throughout a vertical section in a sill is virtually the same, unless there is a weathered zone or alternatively the sill is thick enough to show a vertical variation in grain size.

9. The micropetrographic index (*Ip*) derived from a ratio of the volume of sound constituents / volume of unsound components (including voids, microcracks etc.) indicates the degree of change from fresh quartz dolerite to altered quartz dolerite extremely well, and this ratio can be used as an index to plot against engineering properties.

(i) In the micropetrographic index (*Ip*) the only diagnosis that has to be made under the microscope is between fresh "sound" constituents and altered "unsound" constituents. Voids are included with the unsound constituents. In highly weathered rock the distinction between fine grained alteration products and voids does not necessarily have to be made.

(ii) The *Ip* is a reasonably good indicator of the quality of the rock. But other tests have to be carried out in addition to the *Ip* to give more precise information about the performance of the quartz dolerites under particular conditions.

(iii) The results obtained using the *Ip* values to set a quality index characteristic of each rock look promising. In general the quantity of secondary minerals may itself be of some immediate significance; it could indicate a direct relationship between the content of soft secondary minerals and some physical and mechanical properties. Correlations between the quality index *Ip* and the engineering properties were produced to determine how successful these would be in assessing the quality of rock for use as construction material, and in other engineering applications.

(iv) It has been shown that when the *Ip* is greater than 5 (i.e., percentage of secondary minerals is less than 16%) the rock material can be considered sound (suitable) for roadstone aggregate under all conditions.

(v) This study shows that a detailed knowledge of the petrography, geochemistry and engineering properties of a suite of quartz dolerites has indicated that a forecast of the quality of the rock can be accomplished by a certain indices. In particular *Ip* is an important index for determining the amount of alteration in a rock. Engineering properties can be successfully plotted against *Ip* to determine the minimum *Ip* value for rock with acceptable engineering properties. Thus for similar rocks the value of *Ip* will correlate directly with engineering properties.

10. The suitability of the dolerites as construction material is shown in test values in Table 6.1 and can be summarised as follows:

(i) Tests results show that the quartz dolerites examined in this study can be divided into three groups. Group 1 includes quartz dolerites from Hillend and Cairneyhill quarries, while dolerites from Caldercruix, Duntilland, Tam's Loup 1 and Boards are in group 2, and group 3 includes dolerites from Tam's Loup 2 and Westcraigs quarries. Test results from group 1 and 2 are similar with results from the former being better than those from the latter.

(ii) P-wave velocity in the group 1 is between 5390m/s and 5593m/s, while the altered and weakened dolerites from group 3 give the lowest results between 3050m/s and 4290m/s. Uniaxial compressive strength results for group 1 are between 185MN/m^2 and 196MN/m^2, for dolerites from group 2 they are between 155MN/m^2 and 180MN/m^2 and for group 3 they are between 70MN/m^2 and 120MN/m^2. Table 6.1 shows also that dolerites from group 1 possess the highest point load strength indices and rebound numbers, being between 8MN/m^2 and 9MN/m^2 and 58 and 60 for the point load strength index and rebound number respectively. Quartz dolerites from group 3 give the lowest point load strength indices (2.5MN/m^2 to 5.4MN/m^2) and lowest rebound numbers between 35 and 41.

(iii) Aggregate impact values for the quartz dolerites in the three groups are between 7 and 17 and aggregate crushing values are between 14 and 20, with group 1 having the lowest values of AIV and ACV (i.e., the best results), and group 3 having the highest (worst) results. The two aggregate properties which are specified for road-works are aggregate abrasion value and polished stone value. AAV results showed that all the tested dolerites have aggregate abrasion values between 4.7 and 8 which are within the aggregate abrasion value specified for chippings in very dense traffic roads (i.e., AAV = 10 for traffic density of 3250 commercial vehicles per lane per day) and also within the AAV specified for aggregate in coated macadam wearing course for similar traffic density (AAV = 12, see Table 4.6). This means that aggregates from all quartz dolerites in this study are suitable as chippings and macadam wearing coarse even in high traffic density roads.

(iv) Results of the PSV test range between 56 and 63 and the best results (high values) were obtained from quartz dolerites from group 2 (Tam's Loup 2 and Westcraigs quarries). However, these rocks possess lower strength and higher aggregate abrasion value. This shows the difficulty in finding rocks with a good PSV and at the same time strong (high strength) and abrasion resistant (low AAV). All of the quartz dolerites tested in this study can be used in C sites (see Table 4.10, page 122) which are easy sites and the minimum allowed PSV is 45. They can also be used in most roads in B sites (average) with traffic densities between 1750 and 4000 commercial vehicle per lane per day. Quartz dolerites with PSV of 60 or more (from Westcraigs and Tam's Loup 2 quarries) can be used in A1 and A2 sites which are the most difficult sites where the minimum allowed PSV is 60 for low traffic flow.

(v) Most of the quartz dolerite aggregates from different sites produce concrete with similar properties, with each sample giving concrete with a crushing strength higher than $37MN/m^2$ (which is the lowest value specified for structural concrete used in road-works). The highest crushing strength is obtained from cubes made with aggregates from group 1. The amount of broken aggregate in the crushed cubes made from quartz dolerites from group 3 was relatively high, which indicates a weaker aggregate from this group.

(vi) The drying shrinkage test results range between 0.049% and 0.065% with rocks from group 3 having the highest (worst) results. This could be related to the amount of secondary minerals in these rocks as shown in Fig 5.8. However according to the new BS (which defines the upper limit of shrinkage value in most of concrete works as 0.075%), all the aggregates can be used in most concrete structures.

11. This study could provide a basis for future investigations into geotechnical properties of different dolerite types from other sites and of more altered material; to see the extent to which the amount of secondary minerals can be tolerated in quartz dolerites being used as construction material. This study also provides information on engineering properties of the quartz dolerites of the Midland Valley of Scotland which could be incorporated into subsequent engineering geology maps of central Scotland.

Table 6.1 Summary of the results and classification of the studied quartz dolerites.

GR	WS	Vp	S_c	I_s	R_n	AIV	ACV	AAV	PSV	CCS	SH	SM	*Ip*
1	F	5390 - 5593	185 - 196	8 - 9	58 - 60	7 - 8	14 - 15	4.5 - 5.0	57 - 58	69 - 75	0.049 - 0.053	8 - 9	9.6 – 11.1
2	SW	5166 - 5347	155 - 180	6.8 - 8.1	52 - 57	8 - 12	15 - 16	5.1 - 5.6	58 - 59	55 - 70	0.047 - 0.060	9.6 - 14	5.7 - 8.8
3	WW	3050 - 4290	70 - 120	2.5 - 5.4	35 - 41	14 - 17	18 - 20	7.4 - 8.0	61 - 63	41 - 50	0.060 - 0.068	21 - 24	2.97 - 3.47

GR = group, WS = weathering state, F = fresh, SW = slightly weathered, WW = weathered and weakened, Vp = P-wave velocity (m/s), S_c = uniaxial compressive strength (MN/m^2), I_s = point load strength index (MN/m^2), R_n = Schmidt hammer rebound number, AIV = aggregate impact value, ACV = aggregate crushing value, AAV = aggregate abrasion value, PSV = polished stone value, CCS = concrete crushing strength (N/mm^2), SH = concrete shrinkage (%), SM = secondary minerals and *Ip* = petrographic index.

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APPENDICES

APPENDIX 1

Petrography

Plagioclase

Cal.	Dunt.	Wes.	Cai.	Hil.	Tams.1	Tams.2	Boa.
45.0	45.7	35.0	45.1	46.0	49.6	41.0	50.0
48.0	46.0	39.0	48.0	48.5	47.1	40.0	44.2
45.1	45.0	37.0	47.3	47.0	45.0	41.0	47.1
44	42.0	38.0	46.6	50.0	44.9	39.0	46.1
-	44.0	39.0	52.0	52.0	44.0	36.0	48.1

Clinopyroxene

Cal.	Dunt.	Wes.	Cai.	Hil.	Tam.1	Tams.2	Boa.
23.5	21.0	14.0	24.0	23.0	22.4	14.4	27.0
23.5	24.0	19.0	26.0	26.0	19.7	17.0	26.0
25.0	23.0	18.0	25.9	24.0	17.0	20.0	24.0
28.0	23.6	16.0	24.9	25.1	18.7	19.0	25.0
-	25.0	17.0	27.1	25.1	20.7	18.0	22.0

Iron oxides

Cal.	Dunt.	Wes.	Cai.	Hil.	Tam.1	Tams.2	Boa.
6.6	5.0	7.0	6.0	7.5	6.5	7.0	6.0
5.9	6.3	6.9	6.5	8.7	7.7	7.9	7.0
5.2	6.0	7.3	6.7	8.5	7.3	8.0	7.2
7.9	6.2	7.1	6.3	8.6	8.0	7.7	7.3
-	6.0	8.1	7.0	8.7	7.5	8.2	6.8

Groundmass

Cal.	Dunt.	Wes.	Cai.	Hil.	Tam.1	Tams.2	Boa.
7.0	7.7	8.1	6.9	6.1	8.8	9.1	7.6
7.4	7.8	9.9	7.5	6.9	7.4	11.2	8.0
6.1	7.5	10.2	7.5	7.0	7.8	9.9	8.4
8.3	8.2	9.7	8.0	6.8	7.9	10.1	8.9
-	7.0	10.2	7.5	8.0	8.0	9.7	8.2

Quartz

Cal.	Dunt.	Wes.	Cai.	Hil.	Tam.1	Tams.2	Boa.
3.3	2.4	2.3	3.2	2.0	39	2.3	4.0
3.0	3.5	4.0	2.6	3.0	2.9	3.5	3.0
3.3	3.9	3.8	2.6	2.7	2.8	2.8	2.0
1.6	3.7	3.2	2.6	2.6	2.7	2.7	3.0
-	2.7	3.6	2.0	2.6	2.0	2.6	3.0

Secondary minerals

Cal.	Dunt.	Wes.	Cai.	Hil.	Tam.1	Tams.2	Boa.
12.1	15.2	21.0	7.2	7.0	16.0	19.1	10.6
9.8	13	25.0	9.0	9.2	12.0	23.0	9.4
13.0	12	23.0	9.4	8.0	14.1	22.0	8.0
12.2	14.1	24.3	10.0	7.8	13.5	20.5	10.0
-	15	26.0	9.3	7.9	14.5	21.5	9.6

Appendix 2

Intact rock tests results

1. Ultrasonic pulse velocity

	Caldercruix		Duntilland		Westcraigs	
no.	sat.	dry	sat.	dry	sat.	dry
1	5120	5000	5230	5130	3290	2967
2	5300	5200	5308	5205	3573	3064
3	5286	5185	5370	5267	3413	2904
4	5350	5280	5496	5370	3523	3014
5	-	-	5330	5233	3600	3300

	Cairneyhill		Hillend		Tams Loup 1	
no.	sat.	dry	sat.	dry	sat.	dry
1	5471	5299	5486	5390	5250	5100
2	5509	5392	5585	5450	5100	4805
3	5490	5400	5566	5425	5060	4773
4	5600	5460	5594	5440	5000	4650
5	5500	5397	5640	5500	5080	4900

	Tam's Loup 2		Boards	
no.	sat	dry	sat	dry
1	3290	2870	5460	5290
2	4050	3898	5300	5250
3	3475	3050	5390	5240
4	3505	3022	5275	5200
5	3600	3300	5270	5100

2. Uniaxial compressive strength.

no.	Caldercruix		Duntilland		Westcraigs	
	sat.	dry	sat.	dry	sat.	dry
1	167	181	152	165	63	86
2	160	172	158	164	73	94
3	173	184	167	175	75	96
4	-	183	161	171	67	90
5	-	180	154	177	90	93

no.	Cairneyhill		Hillend		Tams Loup 1	
	sat.	dry	sat.	dry	sat.	dry
1	175	185	182	189	150	164
2	188	189	180	192	159	166
3	189	197	192	199	146	161
4	188	191	189	198	149	163
5	-	190	-	201	-	158

	Tam's Loup 2		Boards	
no.	sat.	dry	sat.	dry
1	88	100	150	167
2	97	120	150	162
3	93	110	159	165
4	99	122	150	160
5	91	118	-	17

3. Static modulus of elasticity.

no.	Caldercruix	Westcraigs
1	56	18
2	56	20.6
3	59	22

no.	Cairneyhill	Hillend	Tams Loup 1
1	57	58	52
2	61.3	62	53
3	62	63	57

no.	Tam's Loup 2	Boards
1	35	54
2	35.6	55
3	38	59

4. Dynamic modulus of elasticity.

no.	Caldercruix dry	Duntilland dry	Westcraigs dry
1	79	78	23
2	78	82	25
3	80	81	24.6
4	77	79	24
5	-	82	28

no.	Cairneyhill dry	Hillend dry	Tams Loup 1 dry
1	84	85	63
2	85	88	70
3	86	85	66.5
4	85	86	68.6
5	87	90	70

no.	Tam's Loup 2 dry	Boards dry
1	45	77
2	48	78
3	49	80
4	47.5	79
5	50	83

5. Point load test.

	Caldercruix		Duntilland		Westcraigs	
no.	sat.	dry	sat.	dry	sat.	dry
1	7.0	7.7	6.7	7.9	2.0	3.25
2	8.11	8.5	7.0	8.4	2.3	4.13
3	7.9	8.1	7.2	8.5	2.7	4.20
4	7.4	-	6.8	8.4	2.5	4.00
5	-	-	7.3	-	2.9	4.36

	Cairneyhill		Hillend		Tams Loup 1	
no.	sat.	dry	sat.	dry	sat.	dry
1	7.7	8.3	7.8	8.32	6.0	6.9
2	9.4	-	8.4	9.0	6.25	7.6
3	7.6	8.2	8.4	8.7	6.25	7.0
4	7.6	-	8.2	8.9	-	7.6
5	8.0	9.0	8.3	8.3	6.3	7.3

	Tam's Loup 2		Boards	
no.	sat.	dry	sat.	dry
1	3.0	4.9	7.00	7.8
2	3.5	5.5	6.95	-
3	3.9	5.6	7.00	7.8
4	4.0	5.7	5.95	7.75
5	-	-	7.0	8.0

6. Schmidt hammer rebound number.

	Caldercruix		Duntilland		Westcraigs	
no.	field	lab.	field	lab.	field	lab.
1	-	53	55	50	36	32
2	-	54	57	51	44	37
3	-	55	63	53	40	35
4	-	58	60	52	39	34
5	-	-	64	55	41	36

	Cairneyhill		Hillend		Tams Loup 1	
no.	field	lab.	field	lab.	field	lab.
1	59	55	62	58	55	52
2	54	58	58	54	62	57
3	66	59	63	59	58	55
4	66	60	61	57	59	56
5	65	58	65	60	58	55

	Tam's Loup 2		Boards	
no.	field	lab.	field	lab.
1	47	43	55	52
2	44	42	62	59
3	46	42	58	58
4	42	37	61	57
5	-	-	64	59

Appendix 3

Aggregate tests results

1. Relative density and water absorption.

	Caldercruix				Duntilland			
	OD	SD	AD	W A	OD	SD	AD	W A
1	2.87	2.90	2.92	1.5	2.89	2.91	2.93	1.4
2	2.88	2.91	2.93	1.4	2.86	2.89	2.91	1.6
3	2.90	2.94	2.95	1.2	2.90	2.92	2.93	1.5
4	2.89	2.92	2.93	1.3	2.90	2.92	2.94	1.4
5	-	-	-	-	2.89	2.91	2.93	1.4

	Westcraigs				Cairneyhill			
	OD	SD	AD	W A	OD	SD	AD	W A
1	2.70	2.77	2.79	1.8	2.91	2.93	2.95	1.3
2	2.74	2.79	2.81	2.5	2.89	2.91	2.94	1.4
3	2.75	2.81	2.83	2.0	2.92	2.94	2.98	1.4
4	2.72	2.79	2.80	2.0	2.92	2.94	2.98	1.2
5	2.76	2.80	2.84	2.2	2.92	2.94	2.99	1.1

	Hillend				Tam's Loup 1			
	OD	SD	AD	W A	OD	SD	AD	W A
1	2.89	2.91	2.95	1.3	2.85	2.89	2.93	1.5
2	2.90	2.93	2.96	1.3	2.83	2.88	2.91	1.7
3	2.91	2.93	2.94	1.2	2.85	2.90	2.93	1.4
4	2.92	2.94	2.97	0.9	2.85	2.90	2.93	1.5
5	2.93	2.96	3.00	1.0	2.86	2.91	2.94	1.5

	Tam's Loup 2				Boards			
	OD	SD	AD	W A	OD	SD	AD	W A
1	2.72	2.75	2.80	2.5	2.81	2.85	2.89	1.4
2	2.75	2.79	2.84	2.2	2.83	2.87	2.92	1.4
3	2.77	2.82	2.86	1.9	2.84	2.87	2.91	1.1
4	2.74	2.79	2.85	2.0	2.82	2.86	2.92	1.3
5	-	-	-	-	2.85	2.89	2.94	1.4

2. Aggregate impact value test.

	Caldercruix			Duntilland		
	AIV	AIVR	M	AIV	AIVR	M
1	9.7	47.8	42.5	8.9	51.6	39.5
2	8.6	50.6	40.8	10.7	46.1	42.2
3	8.0	53.0	39.0	8.7	52.8	38.9
4	8.9	49.5	41.6	7.9	54.0	38.1
5	-	-	-	10.1	49.0	39.9

	Westcraigs			Cairneyhill		
	AIV	AIVR	M	AIV	AIVR	M
1	15.6	35.0	49.4	7.8	53.4	38.8
2	18.9	27.0	54.1	9.0	51.1	39.9
3	17.5	29.1	52.4	8.3	52.0	39.7
4	16.4	32.2	50.4	7.66	53.2	39.1
5	17.1	30.8	52.1	6.51	56.0	37.5

	Hillend			Tam's Loup 1		
	AIV	AIVR	M	AIV	AIVR	M
1	7.3	54.2	38.5	10.4	46.5	43.1
2	7.8	53.3	38.9	13.0	41.1	45.9
3	6.8	57.1	39.9	9.6	48.3	42.1
4	6.6	59.0	34.4	11.5	45.1	43.7
5	5.6	59.1	35.8	13.0	42.0	45.0

	Tam's Loup 2			Boards		
	AIV	AIVR	M	AIV	AIVR	M
1	15.0	43.3	41.7	9.2	47.0	43.8
2	13.9	39.5	46.6	8.5	52.2	39.3
3	13.1	37.2	52.7	7.30	54.0	38.7
4	14.0	41.1	44.9	8.0	53.0	39.0
5	-	-	-	7.0	55.6	37.4

3. Aggregate crushing value test.

	Caldercruix			Duntilland		
	ACV	ACVR	M	ACV	ACVR	M
1	14.9	37.1	48.0	15.7	32.7	52.0
2	13.6	40.0	46.4	14.9	35.1	50.0
3	15.8	37.2	47.0	16.7	28.0	55.3
4	16.0	33.1	50.9	15.1	31.3	53.6
5	-	-	-	14.0	36.2	49.8

	Westcraigs			Cairneyhill		
	ACV	ACVR	M	ACV	ACVR	M
1	18.0	21.0	60.9	15.1	36.4	48.5
2	19.2	20.0	60.8	14.3	40.1	45.6
3	22.0	17.2	60.8	15.0	39.0	46.0
4	20.2	19.6	60.2	15.3	38.0	47.0
5	21.1	19.7	59.2	13.0	42.0	55.0

	Hillend			Tam's Loup 1		
	ACV	ACVR	M	ACV	ACVR	M
1	13.0	40.2	46.8	14.7	35.0	48.3
2	14.7	38.0	47.3	16.0	29.3	54.7
3	15.0	37.3	47.7	15.3	31.0	53.3
4	13.3	40.0	46.7	15.5	30.9	53.6
5	14.0	39.1	46.9	17.0	29.1	53.9

	Tam's Loup 2			Boards		
	ACV	ACVR	M	ACV	ACVR	M
1	17.0	27.7	55.3	15.4	31.1	53.5
2	17.3	27.0	55.7	13.8	35.0	51.2
3	18.6	25.0	56.4	14.5	33.1	52.4
4	20.1	21.0	58.9	16.1	29.0	54.9
5	-	-	-	15.2	30.6	54.2

4. Aggregate abrasion value test.

	Caldercruix	Duntilland	Westcraigs	Cairneyhill
1	4.6	6.7	8.2	4.0
2	5.5	5.0	7.8	5.3
3	5.2	5.3	7.7	6.0
4	-	4.8	9.0	4.7
5	-	5.2	7.1	5.0

	Hillend	Tam's Loup 1	Tam's Loup 2	Boards
1	5.0	4.7	8.8	4.3
2	4.0	5.9	7.2	4.9
3	4.9	6.2	6.9	5.8
4	4.7	5.5	6.7	5.0
5	4.8	5.7	-	4.8

5. Los Angeles abrasion value tests.

	Caldercruix			Duntilland		
	LAV	LAVR	M	LAV	LAVR	M
1	11.9	52.5	35.6	15.0	39.0	46.0
2	13.2	48.8	38.0	13.5	43.0	43.5
3	13.0	48.3	38.7	12.7	48.0	39.5
4	-	-	-	12.2	50.0	37.8
5	-	-	-	13.1	44.0	42.9

	Westcraigs			Cairneyhill		
	LAV	LAVR	M	LAV	LAVR	M
1	18.1	22.1	59.8	11.7	53.5	34.8
2	17.4	28.3	54.3	11.7	54.0	34.3
3	18.0	22.9	58.1	11.9	51.6	36.5
4	18.0	23.0	59	13.0	49.8	37.2
5	19.1	20.0	60.9	11.8	51.0	37.2

	Hillend			Tam's Loup 1		
	LAV	LAVR	M	LAV	LAVR	M
1	13.7	49.0	37.3	15.1	37.1	32.7
2	12.3	51.8	36.5	12.8	50.0	37.2
3	11.5	53.1	35.4	14.7	42.1	43.2
4	11.9	53.0	35.1	14.9	41.0	40.1
5	12.1	52.0	35.9	13.7	44.6	41.7

	Tam's Loup 2			Boards		
	LAV	LAVR	M	LAV	LAVR	M
1	18.9	22.1	59.0	11.9	51.0	37.1
2	17.0	26.0	56.6	13.1	49.2	37.7
3	17.9	24.0	58.1	14.0	45.0	41.0
4	15.9	35.0	49.1	13.5	49.5	36.9
5		-	-	12.6	50.6	36.8

6. Polished stone value test.

	Caldercruix	Duntilland	Westcraigs	Cairneyhill
1	60	59	60	58
2	54	63	59	53
3	58	58	65	58
4	59	56	64	59
5		59	65	57

	Hillend	Tam's Loup 1	Tam's Loup 2	Boards
1	56	56	60	55
2	54	60	64	59
3	57	59	62	58
4	56	61	61	58
5	56	-	-	57