AN INVESTIGATION INTO THE PHYSICAL-MECHANICAL INDICES OF IGNEOUS ROCKS AND THEIR AGGREGATES

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in the department of geology and applied geology
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by

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Dedicated to the memory of my Mother, to my Father and all members of my Family

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Declaration

The material presented in this thesis is the result of my own independent research undertaken between December 1989 and May 1994 in the department of geology and applied geology, the University of Glasgow. Any published or unpublished research has been given full acknowledgement in the text.

A. Boumezbeur Prof. D. M Ramsay

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Abstract

The Present investigation is a study of the physical and mechanical properties of a suite of igneous rocks and aggregates derived from them, with special emphasis on their geological nature and the post emplacement processes which affects them. The study concentrated on:

i- Petrographical characteristics.

ii- Weathering effects and characterisation.

iii- Index properties for intact and aggregate rock strength.

iv- Los Angeles Abrasion Value, methodological and geological factors affecting values

v- Correlation between index properties for intact rock and aggregate.

The rock suite ranges from volcanic to plutonic, basic to acid in composition and fresh to completely weathered. This provided an opportunity for testing and evaluating various textural and weathering variables.

Weathering which systematically affects the engineering properties of rocks, can be quantified by secondary mineral content or alternatively the well established micropetrographic index (Ip). It can also be quantified by physical indices such as specific gravity, porosity and water absorption or mechanical indices such as Rebound Number and Point Load Strength.

Weathering and other geological variables such as grain size and texture are prominent factors affecting the strength of both intact and aggregate strength values.

An investigation into the Los Angeles Abrasion Value (LAAV) and the factors which affect it established that test results are as systematic and rational as the other recognised strength tests, Aggregate Impact (AIV) and Crushing Values (ACV).

In the Los Angeles Abrasion Test It was demonstrated that the mechanism

of comminution is dominantly one of impact loading (80 %) with a minor component of attrition (20 %). The LAAV is consistently affected by geological variables such as grain size, texture, clast shape and degree of weathering in a manner similar to that established for AIV and ACV.

Intact and aggregate strength indices are related in a simple manner and provided geological and methodological variables are known and evaluated, aggregate strength indices can be predictable from intact rock properties.

INTRODUCTION

Knowledge of the physical and mechanical properties of rocks and causes of their variation is essential for evaluating or predicting performance in the wide range of possible geotechnical environments.

In the geotechnical regime rock has three possible states in different structures and their characteristics must be so evaluated. i.e.

- Intact rock
- Rock mass
- Rock aggregate

Part of the present study was to examine and establish diagnostic properties, suitable index tests and explore causes of variation within and between rock types. Having categorised and predicted intact rock behaviour the study was extended to crushed rock aggregate, to see:

1- If intact properties had any predictive value in quality assessment.

2- or whether a different suite of indices was necessary.

In the course of the study special attention was paid to:

1- LAAV test. To widen the data base and identify the influence of any variables or causes of variation in value. The test itself was examined in depth to establish how rational the results are when geological constraints were held tight.

2- An investigation of weathering in both intact rock and aggregate to establish the nature of rock weathering and patterns of change, and to explore index properties with their predictive implications.

In many spheres of activity rock mass characteristics like anisotropy and fracturing may dominate over intact properties. However, there are situations where intact rock properties are more relevant, i.e. blasting; rock cutting/drilling; design of openings in high residual stress fields; the durability of cut surfaces especially subaerial; strength and durability of block stone; performance and durability of rock aggregate.

A significant geotechnical input area in the study of rock materials is the intrinsic or external factors which can effect the response of material at any one time or over a period of time. This metastable material lacks consistency and quality control may becomes an important field for monitoring as well as evaluation.

CHAPTER II

GEOLOGY

2 1- THE MIDLAND VALLEY OF SCOTLAND

The Midland Valley of Scotland has the structure of an ancient rift valley with the parallel Highland Boundary, and the Southern Upland Faults forming the limits of the area. By analogy with modern rift valley systems such as the East African and the North Atlantic, Mac Donald (1965) stated that the Midland Valley may be a part of a larger upper Palaeozoic rift system which has been either buried by Devonian and Carboniferous sediment or obliterated by later tectonic activity.

The presence of a pre-Palaeozoic basement has been revealed by seismic methods and from the samples of gneissose rocks brought to higher level as xenoliths in Carboniferous and Permian volcanic vents. Beneath the Midland Valley the seismic profile suggests the nature of the basement is one of high grade metamorphic rocks at a depth of 7 to 9 km.

The major part of the sedimentary rocks in the Midland Valley are of Devonian and Carboniferous age, underlying about 36 and 38 per cent of the area. Igneous rocks of this age constitute about 21 per cent.

The oldest rocks exposed within the Midland Valley are of

Ordovician and Silurian age and are mainly sandstone, mudstone and conglomerates. They constitute a considerable area in the south-western part of the valley in the Girvan and Ballantrae districts and in the Pentland Hills. In these rocks the oldest beds are of marine origin, but as the succession is traced upwards the sediments change to fluviatile.

In lower Devonian times a considerable thickness of sediments was deposited in an internal basin between the mountains of the Caledonian orogene and the Southern Uplands. These sediments are mainly red and grey sandstone and conglomerates of continental facies with thick piles of basaltic and andesitic lava. These rocks are well seen on the coast, from the Tay estuary to Stonehaven, and in the lavas of the Ochil and Sidlaw Hills.

Post Lower Devonian times were a period of intense faulting, gentle folding, uplift and erosion with important movement along the Highland Boundary Fault. At that time the Midland Valley was raised to an upland area which received no sediment throughout the middle Devonian (Craig 1983).

The Upper Old Red Sandstone, which is mainly a fine to medium grained red, yellow or buff fluvial sandstone with darker red siltstone and mudstone, rests with marked unconformity on older rocks. In general they are finer grained and more mature than the lower Devonian. These sediments crop out in Ayrshire, Edinburgh and in the Pentland Hills and appear to occupy a strip to the North and West of the Clyde Plateau lavas.

Carboniferous times are characterised by a major change of climate and depositional environment. These changes are reflected in a shift from fluviatile and lacustrine to fluvio-deltaic and shallow marine sedimentation. The Carboniferous sediments are mainly sandstone, siltstone and mudstone with thin beds of marine limestone and calcareous mudstone which has been attributed to flooding by the sea. On the surfaces of emerging deltas luxuriant forests grew which ultimately became coal seams. The Northern Highlands and the Southern Upland received no sediments during the Carboniferous which suggests that they remained above the level of deposition.

From an economic point of view, the Carboniferous has played a vital role in the industrial development and the prosperity of the region. The coal, ironstone and the oilshales led to the industrialisation of the area during the nineteenth and the beginning of the twentieth century.

During early to middle Visean times (Calciferous Sandstone time) thick and widespread sequences of alkali olivine-basalt and related lavas were outpoured both in the East and in the West of Scotland. Relatively short lived, local centres of more pyroclastic activity and alkali olivine-basalt lavas erupted in many places in the Midland Valley throughout and up to the Lower Permian. The later volcanic activity was characterised by the intrusion of thick and widespread sill complexes of various alkali olivine-dolerite types, and ended with widespread quartz dolerite sills, during the Stephanian and the late Westphalian. The latter have no extrusive equivalent.

Differential movement along the inherited fractures from the closure of the Iapetus ocean has strongly influenced the thickness of the strata in different parts of the area. Furthermore, these fractures have controlled the location and mode of occurrence of the igneous rocks within the Midland Valley.

The youngest sedimentary rocks in the Midland Valley are of Permian to Jurassic age, consisting mainly of red sandstone and mudstone intercalated with basalt lava, and known as the New Red Sandstone. The sandstone is brick-red and characterised by the presence of wind rounded grains with cross-bedding of dune type. These characteristics show a climatic change from a fluvio-deltaic environment in the Upper Coal Measures to an arid environment, where aeolian desert sandstone has been deposited in the late Permian. In the Midland Valley the New Red Sandstone has restricted outcrop, in Mauchline Basin, Arran and offshore in the Firths of Clyde and Forth.

The most recent solid rocks in the Midland Valley are Tertiary dykes and sills. These dolerites are of both tholeiitic and alkali basalt affinities. The dykes are the continuation of a regional swarm trending NW - SE and centred upon Mull. Sills outcrop in the Mauchline Basin and are known as the sill complex.

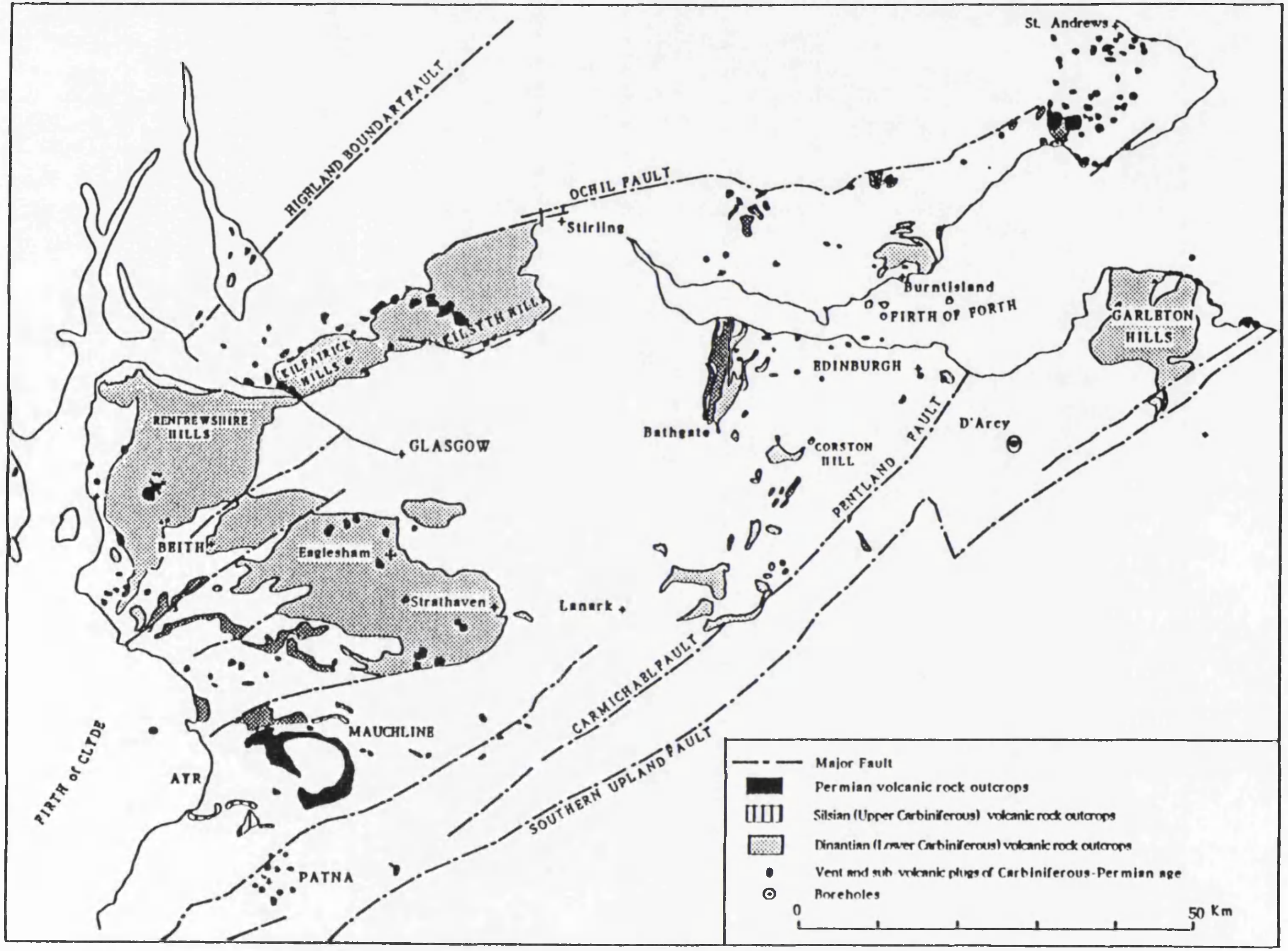


Fig 2. 1 Distribution of volcanic rocks of Carboniferous and Permian age in the Midland Valley (after Cameron and Stephenson 1985)

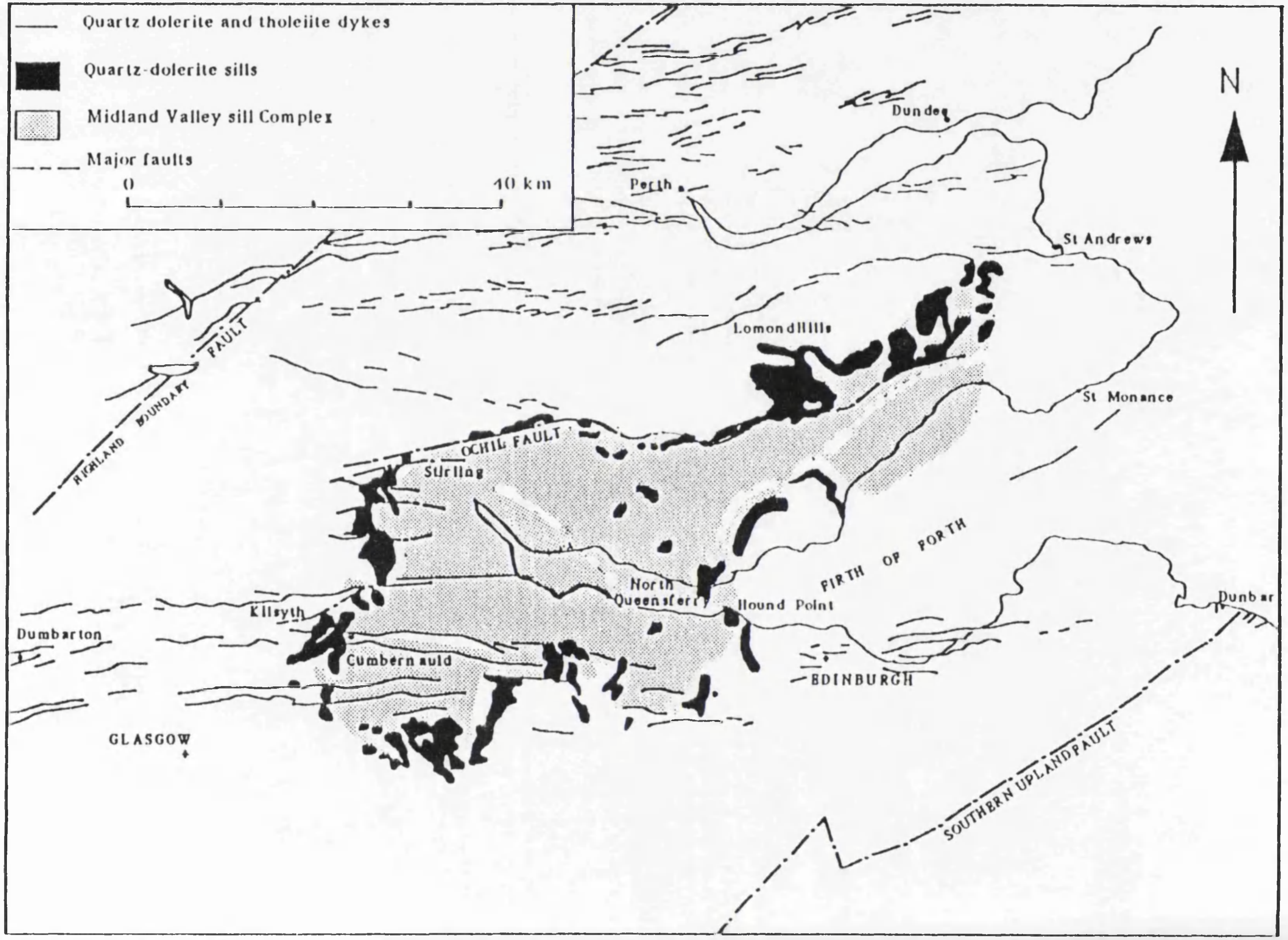


Fig 2. 2 Late-Carboniferous quartz dolerite and tholeiite intrusions in the Midland Valley

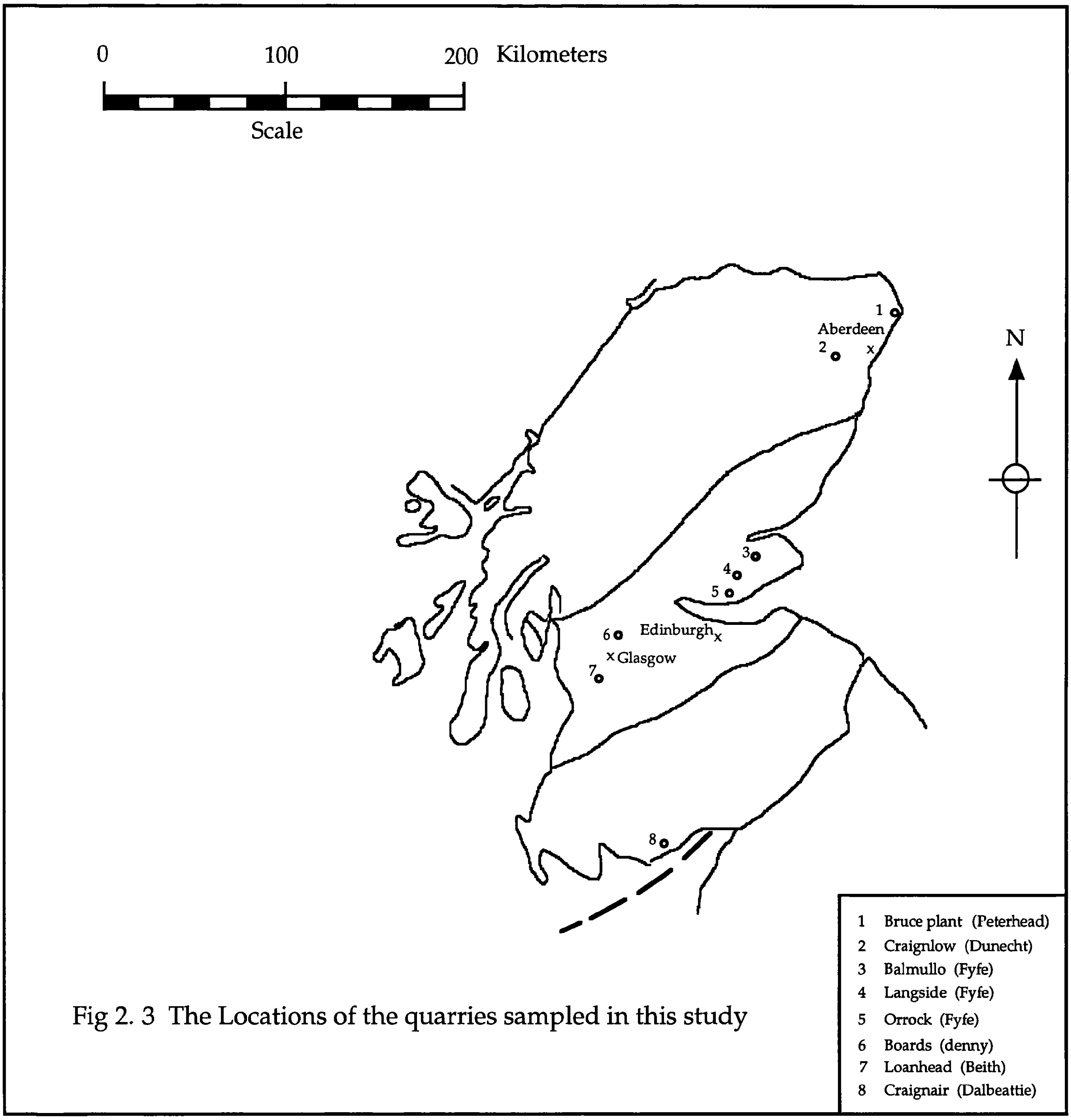


Fig 2. 3 The Locations of the quarries sampled in this study

2 2- Geological setting of the studied rocks

2 2 1- Basalt

During the Carboniferous Period large quantities of basalt lavas were extruded in the Midland Valley of Scotland, from early Visean onwards (Fig 2. 1). In East Lothian the sequence reaches a thickness of 520 m in the Carlton Hills. The basal horizons are tuffs interbedded with thin lagoonal sediments (200 m), succeeded by 160 m of ankaramite, basalt, hawaiite, and mugearite flows and an upper group of trachyte flows and tuffs (160 m). Similar but thinner sequences are observed elsewhere in East Lothian, i.e. D'Arcy borehole (75m), Spilmersford borehole (250 m). In Midlothian the sequence of lava flows and tuffs which form Arthur's Seat and Carlton Hills in the centre of Edinburgh occur at a similar stratigraphic horizon to the East Lothian sequences. Arthur's Seat comprises some 13 flows with well defined tuff bands and reaching a thickness of 400 to 500, m but at Carlton Hill the sequence is only 200m thick. Relatively thin occurences of tuff and lava of the same age are scattered throughout Midlothian, i.e. the basalt plug of Edinburgh castle rock, and south of Arthur's Seat, between Corston Hill and Crosswood reservoir. In Renfrewshire the succession may reach thicknesses of 800 m of Markle type, hawaiite and mugearite. This is part of the Clyde Plateau Lava sequence, the thickest and the most extensive in the valley. Stratigraphical evidence indicates that the plateau attained a thickness of 900 m in places, and accumulated within a short time. The plateau occurs in several fault bounded blocks (Campsie Fells, Kilpatrick, Renfrew and Lanarkshire Hills) each with it's own characteristics. The Campsie Fells block has a thickness of 400 to 500 m, while the Kilpatrick and Renfrewshire blocks are 400 and 800 m thick respectively.

The southern part of the Clyde Plateau which is the thickest of the four blocks extends East - South - East from the Renfrewshire Hills. In the Beith

Hills the volcanic sequence is subdivided into two groups, an upper group represented mainly by microporphyritic basalt of Dunsapie type, and a lower group comprising three sequences:

a) macroporphyritic basalt of Markle and Dunsapie types.

b) Dalmeny and Craiglockart basalts.

c) Markle basalt.

The volcanism continued almost continuously throughout the Selsian but at a less productive level than in the Dinantian. Lava flows continue to dominate in Ayrshire and Bathgate while in Fife the volcanic activity is widespread in the form of small necks and plugs of Hillhouse type olivine-basalt and bedded tuffs.

The tropical climate of the Midland valley in Carboniferous times resulted in contemporaneous disintegration and decay of the lava particularly in upper zones of flows. This weathering resulted in a series of rocks ranging from recogniseable decomposed basalt to a residual material rich in alumina and iron. After burial most of these rotted materials have experienced consolidation, i.e. in north Beith, Markle basalt shows a peculiar subsidence stratification of the rotted lava, due to some reduction in volume consequent upon rotting, later emphasized by the weight of overlying materials. It seems that this process of weathering is confined to flat lying lava flows and plugs, as far as the material tested is concerned, are not weathered.

The basalt tested in this programme is of two types, Markle type macroprphyritic olivine basalt from Loanhead quarry north of Beith, and Hillhouse type microporphyritic olivine basalt from Orrock and Langside quarries in central and west Fife respectively.

2 2 2- **Quartz dolerite**

Towards the close of the Carboniferous Period in Britain (late

Westphalian-early Stephanian, 295 - 290 Ma). A widespread suite of tholeiitic sills and dykes, comparable and contemporaneous with the Whin Sill of northern England, was emplaced in the Midland Valley of Scotland (Fig 2. 2).

The dykes trend E - W and were mainly intruded along fault planes. They cut rocks from Lower Devonian to middle Coal Measures. In the northern Midland Valley individual dykes can be traced for up to 130 km, and they occur between Loch Fyne and Tayside, cutting the Highland Boundary Fault and the Dalradian metamorphic complex. In the central Midland Valley they form a swarm approximately 20 km wide, extending from East Lothian to Dumbarton. Generally the average width of individual dykes is between 20 and 50 m.

Quartz dolerite sills outcrop in many places in the Midland Valley, giving rise to prominant scarp features such as Castle Rock in Stirling, the Bathgate and Lomond Hills, Hound Point (north Queensferry), and Kilsyth. The sills occupy various levels in the Calciferous Sandstone Measures, Lower Limestone and Limestone Coal Groups. They are believed to be connected by near vertical dykes or step and stair transgression and form one continuous body known as the sill complex. The whole of the sill complex is up to 150 m and occupies some 1600 km^2. Francis (1982) has shown that the shape of the complex approximates to a series of "saucers", the lowest and the thickest of which occur in the centre of the Carboniferous basins. Individual sills display a systematic variation in grain size from top to bottom as described by Robertson and Haldane (1937). A zone of coarse crystallisation (pegmatitic zone) at about one third of the distance below the top of the sill grades upwards and downwards through a coarse grained zone to fine and chilled margins.

In the Stirling Sill as exposed in Boards Quarry and elsewhere, more or less vertical belts of highly decomposed dolerite run through the sill from top to bottom in a direction roughly parallel to that of the major joints. This is thought to have formed as a result of hydrothermal activity following consolidation of the sill.

2 2 3- **Dacite**:

The Dacite intrusion (sill) of Lucklaw Hill and Forret Hill in Fife is part of the widespread Caledonian calc-alkaline igneous province, and often classified among the minor felsic intrusions. These intrusions are abundant in the south of the Midland Valley, and concentrated especially in the north western part of the Southern Upland Fault. In the North there are rare dykes and larger intrusions in the Ochil Hills (Forret Hill and Lucklaw Hill). They are extensively quarried for road metal and are a major source of the desirable "pink chips" characteristic of some Scottish roads.

The Lucklaw intrusion is the largest dacite intrusion south of the Firth of Tay, and was originally thought to be a lava flow (Geikie; 1902), but is now considered to be a laccoloth. It occupies around a square kilometer in outcrop and is best exposed in Balmullo Quarry.

2 2 4- **Granite**:

Granitic intrusions were emplaced into the Caledonian mountain range of Scotland during different phases of the Caledonian Orogeny. However, Read (1961), on the basis of the mode of emplacement of these granitic rocks classifies them as:

Migmatites: (Older granites) Generated by the regional migmatisation of the quartzo-feldspathic material of the Moine and Dalradian rocks. They are widely distributed in the north of Scotland.

Newer Granite: These are the most spectacular and the widespread of all the Caledonian igneous rocks. They outcrop in the Northern Highlands east of the

Moine Thrust and in the non-metamorphic Southern Uplands (Fig 2. 4a and b). Large bodies were emplaced after the metamorphism and folding of the Moine and Dalradian sediments. They are calc-alkaline in composition and more than 80% are granodiorite and adamellite.

Last Granite: A number of granitic bodies were emplaced in Devonian times and coincide with the outpouring of the Lower ORS Lavas. They were classified by Read (1961) as permitted or passive intrusions. They rose up in the near surface in the crust as a consequence of down-faulting along ring shaped fractures in the country rocks. In Scotland, they occur chiefly in the South-West Highlands and comprise the cauldron-subsidence of the Etive complex, of Glencoe and Ben Nevis.

The material tested in this programme comes from different areas, Peterhead, Hill of Fare, and Criffel (Dalbeattie). Peterhead granite belongs to group1 and is a member of Read's older and Newer forceful granites (pre-Silurian). The Hill of Fare and Criffell (Dalbeattie), on the other hand, belong to group 2, i.e. Siluro-Devonian, of Read's newer permitted granites.

Peterhead granite has been extensively studied, and is typically a red, coarse-grained rock intensively decomposed at the surface. In thin section, it displays orthoclase feldspars in a more or less decomposed and fractured state together with quartz and biotite. Radiometric dating using the $^{87}Rb/^{86}Sr$ methode yielded an age of 385 ± 8 m yrs (Bell 1968).

The Hill of Fare granite is a large granite mass in North-East Scotland with a laccolithic rather than stock-like form. It is a medium to coarse grained grey, biotite granite with little or no muscovite. In Dunecht Quarry the granite shows decomposition and discoloration at the surface, progressively changing to fresh material downwards.

In the Southern Uplands, The Criffel pluton (Dalbeattie) is mainly

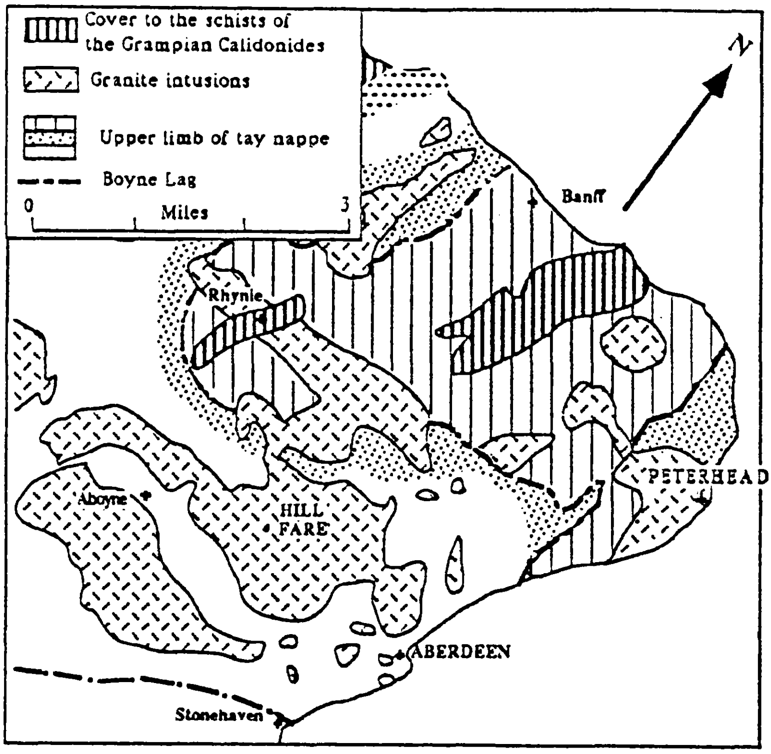


Fig 2. 4a Sketch map of the North East Grampian Highland (after Read and MacGregor 1971)

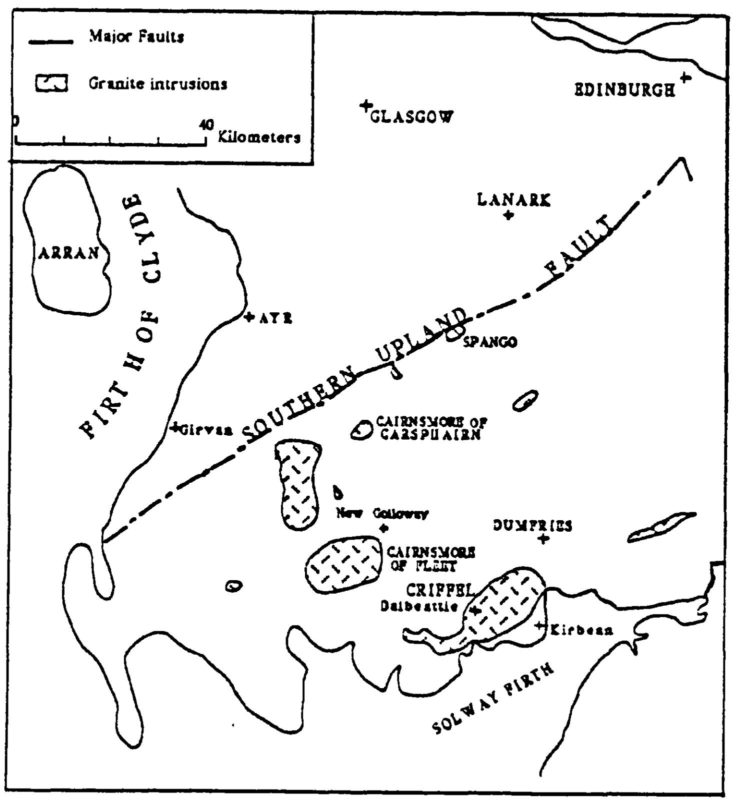


Fig 2. 4b Outcrops of Old Red Sandstone intrusive rocks in the South of Scotland (after Greig et al 1971)

granodiorite with associated quartz diorite. At Dalbeattie the material quarried is a grey medium to coarse grained granodiorite containing plagioclase ranging from oligoclase to andesine, quartz, hornblende, and accessory sphene. Radiometric age determination using the K/Ar method gave an age of 390± 12 m yrs, while Rb/Sr gave ages of 388 ± 19 m yrs and 410 ± 20 m yrs (Greig 1971).

2 3- **Petrography**

2 3 1- **Basalt:**

a- Macroporphyritic Basalt:

In the field macroporphyritic basalt, when fresh, is dark grey with transparent feldspar, black augite, and sometimes yellow to brown looking olivine. When weathered, which is usually the case, it is rusty colored with conspicuous red brown ferric olivine. They also display a greenish tinge as a result of replacement of ferromagnesian minerals and feldspar by chlorite.

The macroporphyritic basalt quarried at Loanhead, north of Beith, is markle type olivine basalt (McGregor 1928). The rock contains abundant phenocrysts of plagioclase feldspar and olivine set in a groundmass of plagioclas feldspar, olivine, augite, and iron ore. In places amygdales of zeolite, prehnite and chlorite are present. The mineralogical composition of this material is displayed in Table (2. 1).

Feldspar phenocrysts: Modal analysis of the Markle type olivine basalt from Beith shows that the feldspar phenocrysts constitute 20% to 30% of the rock. Their average dimensions range from 2 mm to 5 mm and composition from labradorite to bytownite. These phenocrysts are usually corroded and sieved with iron ore and patches of fine grained aggregates of greenish chlorite. They also show irregular ragged rims with the ground mass, and signs of reaction rim from which very fine chloritic material developed.

Olivine phenocrysts: Olivine phenocrysts are less conspicuous than feldspar and generally have microporphyritic dimensions. It usually occurs in an altered state either as a brick-red iddingsite or a greenish aggregate, most probably chlorite. Pyroxene (augite) is less common than olivine, and usually present as tiny fresh crystals within the groundmass.

Ground mass: The ground mass is fine to medium grained according to the size of the feldspar laths. It is usually medium-grained in the vesicular and amygdaloidal part of the lava and fine elsewhere. It comprises plagioclase feldspar, small augite crystals, serpentinised olivine, iron ore, and apatite needles. The feldspar in the ground mass is usually less basic than the phenocrysts and varies from labradorite to oligoclase with dimensions between 0.1mm and 0.4 mm. Augite which has the form of granules and prisms is generally fresh, and pale yellowish to brownish in colour. Very fine greenish aggregates of chlorite are also found in the interstices of the rock.

Amygdales: Amygdales are usually restricted to the uppermost parts of each lava flow and their frequency diminishes considerably towards the base of the flow. They are usually rounded in shape and consist chiefly of zeolite, prehnite, and chlorite. In their dimensions they range from 0.1 to 10mm in diameter.

b- Microporphyritic basalt:

In hand-specimen the microporphyritic olivine basalt is a black to dark grey compact rock. Their fine grain character make it difficult to identify the microphynocrysts of olivine and feldspars, although with the aid of a hand lens they can be easily identified. The microporphyritic basalt studied in this programme can be accommodated within Hillhouse type olivine basalt.

The microscopic observations show a rock which consists of abundant microphenocrysts of olivine with less frequent augite embeded in a goundmass of small, granular, prismatic augite and plagioclase feldspar laths, abundant iron ore and an isotropic base of glass. Apatite needles occurs as accessory minerals, and calcite, anatase, chlorite, and serpentine all occur as secondary minerals resulting from alteration of the rock.

Olivine: Olivine occurs as microphenocrysts and in the groundmass. The microphenocrysts are generally granular in shape with dimensions varying from 0.15 mm to 2 mm and constitute a significant part of the rock. Fresh olivine is abundant, although sometimes altered to a form of yellowish green fibrous or lamellar serpentine. Larger phenocrysts may be intensively corroded.

Pyroxene: In thin section the rock can be seen to be rich in pyroxene. It occurs dominantly as tiny grains and prisms of yellowish and brownish augite and only rarely as microphenocrysts. Augite in the rock is rarely altered and usually includes tiny grains of iron ore as black spots. Zoning has been observed in some augite crystals which dislpay colourless cores with pale brown outer zones.

Iron ore: Iron ore occurs as black grains and crystals, probably magnetite, evenly distributed and fairly abundant. This increases in percentage and size as the ferromagnesian content of the rock increases. Although it occurs as very tiny grains in all the microporphyritic basalt investigated larger crystals of the order of 0.2 mm also occur.

Modal Analysis for Basalt

Table 2. 1. Modal analysis of basalt studied in the present programme

Sample	Feldspar	Olivine	Pyroxene	Ground M	Sec+Amyg+Cra +Voids	Iron Ore
1N	35	altered	5	42.9	15.1	2
1E	8.3	altered	altered	52.3	336.4	3
4N	22.3	altered	altered	57	19.6	1.1
6N	28.2	altered	3.7	54.1	11.7	4
8N	26	0.5	6.5	53.7	11.5	3
O4	1	13.3	6	75.3	2.3	2
Ln4	No Phynocrysts	11	5.6	76.6	5	1.6

2 3 2- **Quartz dolerite**:

In hand specimen, the quartz dolerite of Boards Quarry varies in colour from greenish-black to dark grey and pink with grain size ranging from fine to coarse, i.e. 0.4 mm - 2.74 mm. The mineralogical composition is in all cases simple and consists of plagioclase feldspar, pyroxene, iron ore, quartz, and rarely olivine. Apatite needles occur as accessory mineral and calcite, chlorite, serpentine, biotite, and anatase as secondary minerals. Table (2. 2) summarizes the mineralogical composition of the studied quartz dolerite.

Feldspar: Feldspar is the most abundant mineral constituent in the rock and modal analysis shows that it varies between 40 and 60 % of the rock (Table 2. 2). It occurs as long prismatic crystals with a length to width ratio of around 4, and encloses pyroxene to form a dense interlocking sub-ophitic texture. In size they range from 0.4 mm at chilled margins upto 2.77 mm at the coarse pegmatitic fraction. The bulk of the feldspar seems to be labradorite in all parts of the sill, save the zone of coarse crystallization where albite and oligoclase predominate with potassium feldspar in the ground mass. Feldspar crystals are usually corroded to heavily corroded, especially in the coarse fractions, and become extremely difficult to recognise. This corrosion is in fact the alteration of the feldspar to sericite. In the pink, coarse-grained quartz dolerite feldspar is frequently surrounded by micropegmatite and contains apatite needles. As a result of subsequent tectonic stress and alteration, two patterns of fractures have been observed in the plagioclase minerals. In the first type fractures are filled with chlorite, and always oriented perpendicular to the longest dimension of the laths. These are mainly observed in the vicinity of shear zones (Plate 3. 1). The second type, which is thought to be the result of stress generated as a result of alteration of the ferromagnesian minerals, are transgranular, running from altered pyroxene through feldspar minerals.

While the fracture, especially the first type, are filled with very fine aggregate of chlorite, the second type are usually open and stained.

Pyroxene: Pyroxene is the second most abundant of the major constituents of the rock. The modal analysis shows that it constitutes between 20% to 25 % of the rock. In form they vary between euhedral feathery, elongated prismatic crystals and tiny granules. Clinopyroxene (augite) is the main pyroxene in the rock, although, orthopyroxene may also occur, particularly in the coarse grained quartz dolerite. The augite occurs as colourless to pale brown grains with sharp borders and studded with iron ore. Chemical instability of augite in the sill environment can be inferred from the many instances of the replacive sequence augite - hornblende - biotite. At a more advanced state of alteration augite crystals are partly to wholly replaced by fibrous serpentine, chlorite and calcite.

Ores: Both iron ore and pyrite are present in the rock. The former occurs as inclusions mainly in pyroxene and filling the intergranular spaces in the groundmass. It occurs both as skeletal, which is most probably ilmenite, and as large granules of up to 1 mm in size. Pyrite on the other hand was observed to occur as a crack filling mineral which suggests that it has been deposited from a late circulating hydrothermal fluid.

Ground mass: Generally speaking the ground mass consists of tiny granules of quartz, potassium feldspar, plagioclase feldspar laths, devitrified brownish glass with alteration minerals such as chlorite, serpentine, hornblende, biotite, calcite and anatase. The ground mass constitutes a small proportion of the rock, i.e. 5 to 8 %. In the zone of coarse crystallisation (Robertson and Haldane 1937) the groundmass is dominated by micropegmatite which surrounds plagioclase feldspar crystals, along with tiny grains of quartz and chlorite as small spherules with radial extension filling the interstitial spaces. In the other parts of the sill the micropegmatite is less common and the groundmass consists mainly of quartz, plagioclase microliths, chlorite, biotite, and a base of greenish material probably altered glass.

Other minerals: Among the primary minerals of the rock quartz occurs in small amount as sparse tiny crystals, most commonly filling the interstitial spaces. It also occurs as a by-product of alteration of ferromagnesian minerals. As an accessory mineral apatite needles of variable length occur on plagioclase laths and in the ground mass. Honblende, biotite, chlorite, serpentine, calcite, and anatase all result from alteration of the primary minerals of the rock, i.e. plagioclase feldspar, augite, hypersthene, and olivine. Fibrous serpentine and chlorite are the most frequent alteration products of the ferromagnesian minerals, partly or wholly replacing augite. Olivine was seen in only one thin section, partly altered to serpentine with a heavy staining of its characteristic fractures.

2 3 3- **Dacite**:

Dacite is a fine-grained, white to pink rock, with phenocrysts of plagioclase feldspar. The mineralogy consists mainly of plagioclase feldspar phenocrysts embedded in a ground mass of fine grained quartz and feldspar, with dimensions of 0.1 to 0.3 mm, and a cryptocrystalline, iron stained quartzo-feldspathic base. Biotite also occurs as phenocrysts, although smaller than feldspar and also as tiny grains in the ground mass.

Phenocrysts of andesine occur as large euhedral to subhedral crystals ranging between 1.4 mm and 2.88 mm in size. They are sometimes altered to calcite, sericite, muscovite, and kaolinite, or merely dissolved leaving voids (Plate 4. 2). In the ground mass, the alteration of plagioclase feldspar mainly

Modal Analysis for Quartz Dolerite

Table 2. 2. Showing the modal analysis of some tested samples of quartz dolerite

Sample	Feldspars	Pyroxene	Ground Masse	Quartz	Ores	Apatite	Sec + Cracks
St2	44.2	23.4	5.5	1.8	8	1.2	15.8
St4	36	12.6	24.8	5	4	1.5	18.8
St8	38.2	24.6	11	1.6	8.5	1.5	14.5
St10	47	15.3	18.3	3.6	5	-	12
St11	47.3	24.3	5.3	2	6.6	nd	12.2
St12	40.6	22.3	8.3	3.8	2.3	-	22.7
St19	47.1	28.5	7.7	0.5	3.5	-	13.18
St21	35	12.8	7.7	6.2	7.2	-	32
St22	46.3	-	25.2	5.3	1	1.3	20.83
St26l	36.6	8	27	4.7	2.6	0.6	20.5
St30	29	8.4	8	-	6.6	0.3	55.9
St31	33.9	12	2.3	-	5	-	46

affects the central parts of some grains while the rims remain fresh.

Ferromagnesian minerals such as biotite alter to pseudomorphs of chlorite with iron staining on cleavage traces, or to red brown, heavily iron-stained secondary minerals. Zircon occurs as an accessory mineral

2 3 4- **Granite:**

The granitic material studied in this programme comes from three different localities, Bruce Plant Quarry in Peterhead, Graigenlow quarry in Dunecht (Hill of Fare), and Craignair Quarry in Dalbeattie (Crieffel). Peterhead granite is a coarse-grained red granite, Dunecht is medium-grained grey granite, while the granodiorite from Dalbeattie is a medium to coarse grained grey rock. The modal analysis results for the three studied granites is presented in Table 2. 3).

a- Peterhead granite: Peterhead granite is a coarse grained variety, red in colour when fresh, with grain size between 1 mm and 6 mm. The texture is coarsely granular with potassium feldspar and quartz being the major constituents of the rock. These comprise around 80 % of the rock, with the remainder consisting of minor plagioclase feldspar, biotite, chlorite, sphene, zircon, and ores.

Feldspars: In thin section feldspars are cloudy in appearance. Potassium feldspar occurs as large xenomorphic crystals with perthitic texture. Plagioclase feldspar with a composition of (An_{20-25}) is also present with conspicuous albite twining, and sometimes enclosed by potassium feldspar. They are generally more corroded than potassium feldspar. This corrosion is due to their alteration to tiny particles of sericite and kaolinite.

Quartz: Quartz is second in abundance, it's modal proportions varying between 25 and 33 %. It occurs as large xenomorphic grains with dimensions varying between 0.8 mm to 5 mm. From the potassium feldspar, it is easily distinguishable by being free from corrosion and it has slightly higher interference colour. It include in many instances, small crystals of biotite and other minerals.

Biotite: Biotite is the commonest ferromagnesian mineral present with modal percentage varying between 5 to 6 %. It occurs mainly as irregular clumps in the rock, but they are also found as small inclusions in quartz and potassium feldspar. Biotite itself usually contains zircon and sphene as inclusions, while altered samples show biotite completely replaced by chlorite with iron staining the cleavage planes.

b- Craigenlow Granite

The Craigenlow granite is a medium to coarse-grained grey granite, comprising feldspar and quartz as the main constituents, together with hornblende, biotite, sphene, zircon, and iron ore.

Feldspar: The major constituent of the rock, potassium feldspar, is present as large xenomorphic crystals some enclosing laths of biotite and sphene. They are slightly perthitic, and generally cloudy in comparison to quartz. On the other hand, plagioclase feldspar shows euhedral to subhedral crystals with conspicuous albite twining. They are more cloudy than the potassium feldspar, and usually alter to a fine aggregate of sericite. Zoning in the plagioclase feldspar is not uncommon. These feldspars also have a sodium rich core and potassium rich margin reflected in the decreasing amount of alteration from the core outwards. In the ground mass potassium feldspar grew as a space filling mineral.

Quartz: Quartz constitutes about 25 % of the rock, and occurs as large xenomorphic crystals 0.5 to 2 mm, while in the ground mass it occurs as tiny grains filling the intergranular spaces. In fresh material the quartz exhibits hair-like microcracks but in weathered samples a branching network of intra and intergranular microcracks is prominent, some partly filled with chlorite, clay or silica, others remaining tight and clean. Inclusions of small grains of biotite and sphene in quartz crystals are not uncommon.

Biotite: Biotite occurs as scattered irregular clumps some of which display perfect cleavage. Their dimensions vary between 0.2 to 2.4 mm and they usually enclose tiny grains of zircon and iron ore. Small biotite grains occur within potassium feldspar and quartz minerals and they are also found filling intergranular spaces. Wavy extinction and microcracks were also found in biotite crystals.

Hornblende occurs but less frequently than biotite. These are euhedral to subhedral in form with yellow brown to dark green pleochroism. As a result of alteration, it has been observed to be partially replaced by hornblende.

c- Dalbeattie Granite:

The rock known as "Dalbeattie Granite" is in fact a medium grained grey granodiorite, consisting mainly of plagioclase feldspar, microperthitic potassium feldspar, quartz, biotite, and hornblende, with sphene, apatite, zircon, and iron ore as accessory minerals.

Feldspars: Plagioclase feldspar is the most abundant mineral consisting about 46 % of the rock. It occurs as euhedral crystals of oligoclase to andesine and averages about Ab_{75} An_{25} (Richey et al 1930). Plagioclase is more anorthitic in the cores of grains passing almost to albite at the extreme edge of the crystals. On alteration, this produces a heavily corroded central zone and clear margins.

Potassium feldspar, on the other hand, occurs as xenomorphic crystals of microperthite enclosing grains of other minerals. It constitutes about 20 % of the rock and quite often occurs as an interstitial mineral. Feldspars alter to both sericite and clay (Kaolinite). Kaolinite blades that have the typical morphology of kaolinites but containing potassium and iron, are an alteration product of feldspar (Plate 3. 5). Melden (1967) also showed that kaolinite may contains iron and potasium.

Quartz : The volume of quartz in this rock amounts to around 20 to 22 %. It is present as xenomorphic crystals with plagioclase feldspar being euhedral towards it which suggests it is an interstitial mineral. It is usually clear and encloses tiny grains of other mineral.

Biotite : Biotite constitutes about 6 % by volume of the rock, occurring as ragged grains with strong pleochroism from dark brown to pale straw, and containing inclusions of zircon and iron ore. It occurs also as inclusions in quartz and microperthite. In weathered samples, biotite is found altered to colourless and non pleochroic material which is most probably clay.

Hornblende also occurs in a lesser proportion than biotite. It occurs as relatively small crystals with yellowish green to dark green pleochroism. It sometimes show partial alteration to biotite.

Modal Analysis for Granite

Table 2. 3. Showing the modal analysis of some tested samples of granite

Sample	Feldspar	Quartz	Micas	Hornblend	Sec Minl + Cracks
Gd1	60	26	8.5	-	4.8
Gd2	35.9	23.6	5.4	-	35 (no cracks)
Gd3	53.1	21	10	1.8	15.5
Gd4	50.4	28.1	5.4	-	15.8
Gd5	56.5	33.1	0.4	-	9.4
Gd6	50.4	25.4	-	-	21.8
Gd7	45	39.5	7	3	5.4
Gd8	46.8	25	5.2	2	20.9
Gd9	47	38.5	6.5	1.5	6.3

CHAPTER III

Weathering and Hydrothermal Alteration

3 1- Weathering

Plutonic and hypabyssal rocks which crystallise at high temperature and pressure within the Earth's crust undergo a series of transformations in their chemistry, mineralogy and fabric when they experience the new conditions of pressure and temperature at the Earth's surface. Saunders and Fookes (1970) followed Weinert (1964) in defining weathering as that process of alteration of rocks occurring under the direct influence of the surface or near the surface situation. It may also be envisaged as the result of equilibration or partial equilibration of rock minerals and fabric to a change in the physical environment. Most of the minerals of igneous and metamorphic rocks are stable only at the pressures and temperature of their formation. A change in pressure and temperature conditions through change of location in the crust is accompanied by progressive alteration to create a modified mineral assemblage and texture. Generally weathering reactions proceed until the whole rock becomes soil.

Weathering is in general the result of two dominant processes acting together on the rock. These are physical weathering which results in the disintegration of the rock by fracturing with minimal mineralogical changes, and chemical weathering which is a decomposition of the constituent minerals. Although the two processes seem to act independently, in fact their occurrence in isolation is extremely rare and most commonly one acts to enhance the other.

Over the last 30 years or so, many studies have attributed in service failure of rock material to the presence of secondary minerals and *in situ* active weathering agents (Weinert. 1964, 68, Van Atta. 1974, Wylde. 1976, 82, Cole and et al Cawsey and Massey. 1983, 88, Mellon. 1985, and Fookes et al. 1988). Any potential evaluation of the durability of rock material, therefore, must take into account the geological history and the weathering state of the material at the time of selection and the environmental conditions to which this material will be subjected while *in service,* i.e. (present day weathering agents and conditions).

3 1 1- **Physical Weathering**

Physical or mechanical weathering can be defined as any process which causes in situ fragmentation or communition without contributory chemical change (Reiche 1950). This is a process brought about by a series of cycles of internal and external stresses such as loading and unloading, freezing and thawing, wetting and drying, heating and cooling and salt action. These stresses, most commonly acting on previous discontinuity surfaces and flaws within the material, lead to strain and eventually rupture and breakdown of the rock.

The freeze-thaw of water in pore spaces and open cracks can exert a stress upto 200 MPa (Ollier. 1984, Fookes et al. 1988), which considerably exceeds the tensile strength of the rock. Diurnal cycles of freezing-thawing can inflict much more damage to the material than if the temperature remains below freezing for a long time.

Like freezing-thawing, the pressure of salt crystallisation can lead to disintegration of the rock. The processes of crystallisation, thermal expansion, and chiefly hydration of salt within the pore spaces of the rock builds up a pressure of several tens of megapascals, sufficient to disintegrate the rock (Winkler & Wilhlem. 1970, Winkler & Singer. 1972 and Fookes et al 1988). The process of crystal growth from saturated or supersaturated solutions leading to rock disintegration is fully described by Evans (1969).

Water on its own is a very important agent of weathering. Alternate cycles of wetting-drying of the rock material can lead to breakdown and disaggregation of the rock material, both strong, and fresh as well as weak. The disaggregation can be explained by the fact that clay minerals and amorphous mineraloids have the property of expanding when wet and shrinking when dry leading to the initiation of microfractures within the rock. Ollier (1984) suggested the mechanism of "ordered water" molecular pressure as an explanation for the disintegration caused by the wetting-drying process, while Cawsey and Mellon (1983) have reported that after several cycles of wetting and drying weathered samples of graywackes developed a net of shrinkage cracks.

Disintegration following thermal expansion results from the fact that rock is generally a poor conductor of heat and the constituent minerals have different coefficients of thermal expansion, and the majority are anisotropic with respect to thermal expansion. When the surface of the rock is heated a thermal gradient develops between the near surface and the inner parts of the rock leading to an expansion of the outer shell away from the cool interior. Diurnal repetition of this process progressively affects the cohesion of the outer shell and leads to it's eventual disruption. In addition, the differential expansion of different minerals within the polymineralic fabric causes a development of stress along the grain contacts which initiate and propagate cracks and under appropriate conditions can lead to disruption of the material.

As will be discussed more fully in the next section, cracks and pores in rocks can also be initiated and developed as a result of mineral alteration processes. This alteration of primary minerals to secondary minerals results in a volume increase particularly in the presence of water, which in turn generates internal pressures causing crack development and opening.

3 1 2- **Chemical Weathering**

The general principle underlying the chemical reactions of weathering is the Lechatelier principle, which states that any system in equilibrium will react to restore the equilibrium if any stress is applied. Goldish (1938) states that resistance of rocks to chemical weathering depends on the susceptibility of the component minerals, and this decreases in the direction of the crystallisation series of (Bowen 1928).

Chemical weathering is a response of the primary mineral constituents to water, free oxygen, carbonic acid, organic acid and nitric acid. Under the new conditions of the weathering zone the primary minerals progressively convert to more stable secondary minerals. These secondary minerals differ from the parents in having a lower density, and containing H^+ and OH^- from their reaction with the water. They are also typically poorly crystalline or even amorphous, and mechanically weak. The mechanical weakness of these components, frequently results in weakening of the whole rock. Many authors have studied the effect of these agents on the weathering of silicate minerals. Ollier (1984) observed that weathering can be achieved by nothing more than rock and water. Water is the most active agent of the process, not just in providing H^+ and OH^- for hydrolysis reactions, which effect a breakdown of the mineral structure, but also for the removal of soluble components, allowing the reaction to proceed (Keller 1957). What remains will be reconstituted into secondary minerals. Raggatt et al (1945) showed that water alone is required to convert basalt into bauxite through the removal of Ca, Mg, Na, and K. Correns (1961) demonstrated that the pH of the water affects the rate of decomposition.

In the hydrolysis process the H^+ ions, on account of their high mobility and size, can easily penetrate into the silicate framework, and substitute for the metal cations in the interstices of the framework. Subsequently, metal cations are expelled from their original sites to the surrounding water. Because Si-O-Si linkages are not very strong, and even weaker when the Al replaces Si, and the stability of the structure is maintained by the complex interplay of geometrical and electrostatic factors involving all the atoms in the final compound. The substitution mentioned above causes a charge imbalance leading to instability and disruption of the silicate framework. This leads eventually to their breakdown. The resultant silicate fragment will constitute a silicate gel from which by ageing the stable or metastable secondary minerals will be reconstituted, i.e. In the zone of weathering, the soluble Fe^{++} will be oxidised to Fe^{3+} and precipitate as $Fe^{3+}O\text{-}OH\text{-}H_2O$ gel, which as it ages will become a true mineral such as limonite (hydrated goethite).

Chemical weathering can also be activated by biological agents as living plants in the exchange of nutrients provide a continuous source of H^+ which creates an acid environment and causes the weathering of the nearby rocks by hydrolysis.

The secondary mineral products of weathering are generally very difficult to identify. This arises because of their very small size, poor structural ordering and the fact that most of them are a mixture of several mineraloids.

The problem of chemical alteration should be considered on the one hand as the changes preceeding extraction from the quarry and on the other, the likelihood of change once the material is in service.

The manner in which alteration occurs can strongly influence the engineering properties of the material. Knight and Knight (1935) noted that alteration may proceed on the periphery of grains, within grains (kernel alteration) or throughout the whole grain. Peripheral alteration effects a reduction in the bonding strength between grains to such an extent that they can easily plucked out, while a small amount of kernel alteration generally has a slight effect.

3 2- **Hydrothermal Alteration**:

Hydrothermal alteration is the decomposition of minerals resulting from the direct interaction between rock and hydrothermal fluids. The final extent depends largely upon the degree of this interaction, lithology and the ambient physico-chemical conditions. This process of alteration is directly or indirectly caused by igneous activity. In the later stages of igneous activity, with the falling temperature of the intrusion, hydrothermal fluids can be generated from the condensation of heavy gases within the intrusion. On the other hand, the altering liquid can also be circulating groundwater heated by the igneous body.

The similarity in end products of hydrothermal alteration and weathering processes has in most cases made it impossible to distinguish between the two. In some cases however, the difference in composition, temperature and pressure conditions of the hydrothermal liquid may lead to the formation of some distinctive secondary mineral assemblages such as clays of unusual composition and zeolites. In the field, by mapping the intensity of decomposition in any particular outcrop it should be theoretically possible to determine whether the decomposition is due to weathering or hydrothermal alteration processes, since with depth the former decreases while the latter showed increase.

Since hydrothermal alteration results in a decomposition which is physically similar to that caused by weathering, the net effect on the engineering properties of rock will be the same.

3 3- **Products of Weathering and Alteration**:

In order to determine the weathering products from the different rock forming minerals in the rocks studied such as pyroxene, olivine, feldspars, and micas standard thin and polished sections have been prepared from samples covering the whole range of visible weathering present in each quarry visited. Thin sections have been examined using a petrological microscope, and polished section and intact samples have been examined in a Leica Cambridge Stereoscan 360 with an Integrated Link Analytical AN 1085 S Energy Dispersive X-Ray Micro-Analyser and a Four Quadrant Backscattered Electron Detector.

Weathering of the quartz dolerite shows a progressive decrease in intensity from the top of the sill downwards. It has been found to be both a granular disintegration and decomposition developing inwards from joint and fracture planes. In the upper part of the quarry, this is expressed in residual corestones enclosed in several cohesive heavily weathered concentric shells (Plate 3. 11), and a sandy and angular gravel-like material called gruss.

In thin section, despite the fact that the rock is heavily stained and weathered, fresh ferro-magnesian minerals where found in large amounts

(Plate 3. 9). Some pyroxene was completely replaced by chlorite or very fine grained chlorite and iron ore. In addition, it can be clearly seen that the grain boundaries of some pyroxenes are the site of a reddish to yellowish-brown material showing a very weak pleochroism. This material generally develops inwards from the grain boundary and mineral fractures and cleavages and is most likely vermiculite. Basham (1974) found similar material in deeply weathered gabbro which he suggested was vermiculite. pyroxenes are also heavily spotted by a residual iron stained material or irregularly shaped cavities formed from dissolution. Plagioclases are slightly to heavily corroded due to alteration to sericite.

In the zone of coarse crystallisation where felsic differentiates are patchily mixed with the more normal types, late stage reactions between the rock and hydrothermal fluid are chiefly responsible for mineral alteration (Plate 3. 10). Pyroxene is variably altered to serpentine, chlorite, calcite and in a few occasions anatase has been observed.

Plagioclase-feldspar on the other hand, appears to hold it's structure together and is only slightly altered to sericite, giving it a corroded appearance. In places the feldspar is partially replaced by reddish green aggregates, most probably chlorite, formed after the fixation of iron and magnesium ions on the residual structure of the feldspars. Again joints and fracture surfaces are slightly to heavily stained.

In the lower part of the quarry joints and fracture surface are only slightly stained to clean. Although the rock contains small amount of secondary minerals it is considered to be fresh.

In terms of microfracturing, three types of cracks have been developed and are well displayed on plagioclase laths. These are intergranular, intragranular and grain boundary cracks (Plate 3. 1, 3. 2). These result from swelling of secondary minerals or by tectonic activity associated with shear zones. Their pattern changes from simple to branched and reticulate as weathering increases. In the slightly stained to fresh quartz dolerite the cracks are predominantly simple and intragranular, chiefly perpendicular to the longest axis of the plagioclase laths. Although there is no staining, many display pleochroic filling material, probably chlorite. On the other hand, in the weathered and heavily stained material the three types of cracks are actually present in a branched and reticulate pattern. Both tight and open microcracks are generally heavily iron-stained and some open microcracks may be partially to wholly filled by secondary minerals. Clean open microcracks are not considered here to be due to weathering processes but are an artefact of slide preparation. SEM analysis of the crack infilling revealed chlorite and clay minerals and in some instances pyrite.

The macroporphyritic basalt of Markle type from Loanhead Quarry, Beith, consists of a succession of flows, each unit of which is characterised by a lower zone of dense, hard rock with high Fe, Mg content and an upper one of vesicular and amygdaloidal aspect.

In the tropical climate of the Midland Valley during Carboniferous times, the flows experienced deep weathering. The upper vesicular portion of flows are very heavily decomposed with a heavy iron-staining (Plate 3. 7). These rotten lavas display a complete destruction of the material fabric subsequently consolidated by burial load. In the more lowly stained vesicular portion, immediately beneath the rotten lava, an interstitial greenish material has been observed widely disseminated throughout the rock (Plate 3. 8). This is probably an alteration product of feldspars and devitrified volcanic glass. The amygdales are filled with zeolite (analcime, natrolite), chlorite, calcite and cryptocrystalline silica ranging from 1 or 2 mm up to 1 cm in size. Plagioclase feldspar of andesine composition is moderately to highly altered to a very fine greenish aggregate in their inner parts, and to clay minerals along their grain boundaries, and frequently highly albitised. On the other hand pigeonite grains show little alteration to chlorite.

The lower parts of flows are finer in grain size, with larger and more abundant phenocrysts and occasional amygdales. Plagioclase phenocrysts up to 5 mm in length are labradorite to bytownite. Replacement of a zone or core of plagioclase by a fine grained greenish aggregate of chlorite and calcite minerals has been observed. The ferro-magnesian minerals show very little alteration along their grain boundaries, although their is some total alteration to serpentine. Olivine, on the other hand, is generally altered to a pale yellow-olive fibrous material probably serpentine or to a reddish brown iddingsite. The few vesicles observed are filled with fine grained chlorite and cryptocrystalline silica. In the lower as in the upper parts of the flow iron ore occurs as euhedral to subhedral microphenocrysts and as fine grains scattered throughout the ground mass.

The microcracks developed in this material are intergranular and intragranular or grain-boundary (Plate 3. 3). The latter are better seen in the plagioclase phenocrysts, where they are slightly to completely stained and tight. The intergranular microcracks on the other hand are wide (up to 2 mm wide) and filled with deutric alteration product such as chlorite, calcite, zeolite and cryptocrystalline silica or clay such as illite (Plate 3. 4). These intergranular cracks may exhibit in some instances a shear displacement of up to 4 mm, observed and measured on plagioclase and pyroxene phenocrysts. The frequency of cracks in basalt is far less than in quartz dolerite or granite. Weathering in basalt is mainly dominated by mineral alteration rather than cracks.

Granite material from 3 quarries, Craignair (Dalbeattie), Bruce plant (Peterhead), and Craignlow (Dunecht), show a wide spectrum of weathering decreasing from the surface downwards. In the fresh state the mineral constituents of the granite are unstained, hard, and unaltered with few hair-like intragranular microcracks mainly restricted to quartz grains. The texture is granular with tight grain boundaries. As weathering gradually progresses, both the amount of staining and mineral alteration increases. At the stage of slight discoloration, plagioclase is altered to sericite, mainly along cleavage planes, potassium feldspars are slightly cloudy, and iron segregation along biotite cleavage planes becomes apparent. Microcracks are still tight and most commonly restricted to quartz grains. At an advanced weathering stage, the weakened stage, the rock is completely stained, the biotite is partly to completely decomposed, and it's pleochroism is lost to a great degree. Plagioclase is partly altered to a very fine grained material, and potassium feldspar in general shows a cloudy appearance. Instances where feldspars alter to clay minerals (kaolinite) were also observed (Plate 3. 5). Intensive leaching and iron staining has caused opaque areas and voids to form. Microcracks at this stage are not restricted to quartz, and range from simple to branched and reticulate patterns, and in most of the cases intergranular. At this stage the rock is characterised by a high density of microcracks most of which are opened and filled or stained. Granite weathering in the quarries studied is one of intense granular disintegration rather than decomposition.

3 4-Effect of Weathering on Engineering Properties of Rock:

The in-service record of weathered rocks over the years has drawn the attention of many researchers involved in the field of testing and evaluating the engineering properties of rock material. Poor performance, lack of durability, and premature failure of weathered material while in service are chiefly attributed to a reduction of the bonding strength between the constituents mineral grains on the one hand, and structural defects such as cracks and voids on the other. As weathering proceeds the bonding forces between mineral grains are weakened by grain boundary alteration, and cracks propagating due to swelling. Thus the whole process results in a general weakening of the rock. Mendes et al (1965), Onodera et al (1974), Simmons et al (1975), Turk and Dearman (1986) and Hamrol (1961) has shown (Fig 3. 1) that the strength and elastic characteristics of rock material decrease as weathering proceeds.

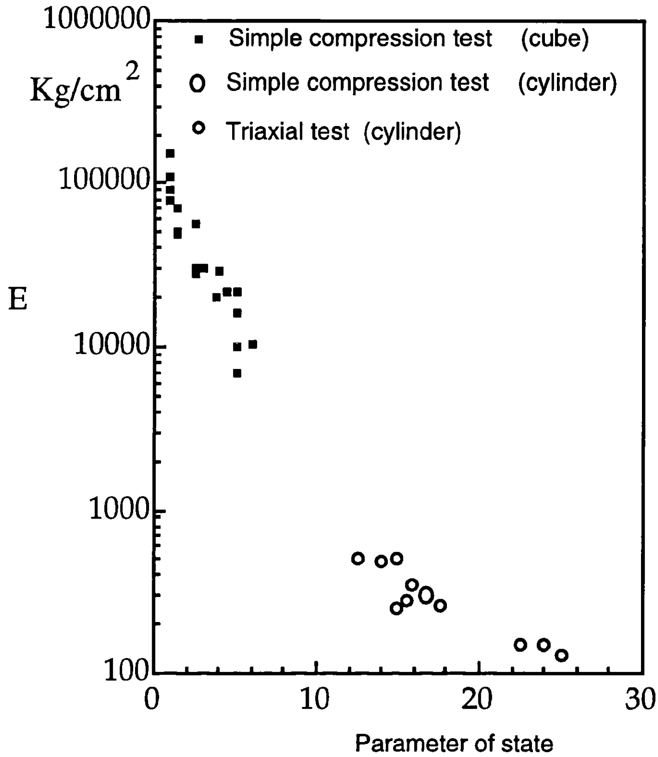


Fig 3. 1 Relationship between the modulus of elasticity E and the quick water absorption for granite (after Hamrol 1961)

Scott (1955) investigated the effects of weathering, in terms of secondary mineral content, on the engineering behaviour of rock material in service. He found that the chief cause of material failure was attributable to the secondary mineral content. He emphasised the severity of the problem when he stated that "failures have occurred in spite of the best design and careful control". Subsequently, the effects of secondary minerals (alteration products) on the engineering behaviour and performance of rock material has been extensively studied by many authors. In South Africa, Weinert (1964, 65, 68, 84) found that the durability of doleritic aggregates in road foundations is directly related to their secondary mineral content and the ambient climatic conditions. The fines released in the presence of water after the breakdown of the aggregates produced a highly plastic layer which is the chief cause of the premature failure of the road. In Western Oregon in the United States Van Atta et al (1974, 76) found that the premature failure of road material is due to the presence of secondary minerals especially clay, while in Australia Wylde (1976) came to similar conclusions. In the UK Hosking and Tubey (1969) found that strong wear and degradation of some doleritic material used as road surfacing material occurs as a result of their weathering state. It should be noted, however, that the presence of secondary minerals does not mean automatic rejection of the material and experience has shown that up to 20 % secondary mineral could have no effect on the performance of rock material (Weinert. 1965, Cole and Sandy. 1980).

The distribution and nature of the secondary mineral content does have an effect on the engineering behaviour of weathered rock material. Scott (1955) stated that even moderate amounts of decomposition on the grain boundaries significantly reduces the strength of the rock.

Mellon (1985) found that the distribution of devitrified basaltic glass has a significant influence on durability and strength of basaltic aggregate. Considering the nature of the secondary mineral content, Cole and Sandy (1980) used a rating system in which each type of secondary mineral has been given a value. Secondary minerals having the most deleterious effect on material durability, eg, expandable clays like smectite and saponite, were given a value of 10, and those believed to have the least detrimental effect, eg, calcite and micas, were assigned a value of 2. This rating system gave very satisfactory results in comparison with the results from other well known tests such as Washington Degradation Test and the methylene blue dye absorption .

The weakening of rocks during weathering stems from the fact that several minerals undergo major changes in their chemistry and structure. EggIton (1984) has shown that the alteration of olivine to iddingsite involves first, a stage of olivine breakdown into a mozaic of fine needles which results in the opening of solution channels from the rim of the grain inwards. These channels will later be filled with smectite, one or two layers in each channel. In a second stage precipitation from solution as water migrates through the solution channels causes an enlargement of saponite and goethite nuclei. He also showed that the reaction preserving Fe, requires Al and water and releases Mg and silicon. Smith et al (1987) stated that the degradation of olivine structure is probably due to the dissolution of Mg and it's replacement by H^+ thus distorting the structure and weakening the remaining inter-elements bonds. Such a process when occurring on the grain boundary causes a major decrease in the intergranular bonds as a result of which material failure in service is most likely to take place.

In pyroxene, the tetrahedra are arranged in chains which are bonded together by metallic ions, the most common being those which enter in octahedral co-ordination with oxygen, such as Mg^{2+}, Ca^{2+}, Fe^{3+}, and Al^{3+}. Bonding by the octahedral cations is relatively weak and pronounced cleavages with planes approximately normal to each other parallel to the silica chains. Access of water by way of the cleavages promotes solution of the bonding cations and causes rapid breakdown of the structure. Upon release, the chains polymerise into sheets incorporating residual alumina and magnesia forming chlorite or montmorillonite.

In phyllosilicate minerals such as biotite, alteration to chlorite at 300 ^{0}C occurs through brucitization of a trioctahedral mica. Actually two hypothesis has been put forward to explain such a process. Olives Banos et al (1983) and Olives Banos (1985) suggest that brucitization occurs in the interlayer space of biotite (potassium plane) where partial slip or cleavage has occurred. The new brucite layer with a biotite layer give together one chlorite with 30 % volume increase. Eggleton and Banfield (1985) suggest a mechanism where two biotite layers transforms to one chlorite. the mechanism involving 35 percent volume decrease and only the octahedral layer of one biotite is inherited intact by the chlorite. However, in tropical regimes where leaching is intense, the loss of the interlayer K (potassium) and it's replacement by hydrated cations transforms biotite to vermiculite (Banfield and Eggleton 1988). These processes are all accompanied by a increase in the interlayer spacing and eventually an interlayer bond decrease which leads to exfoliation and microdivision later on (Robbet and Tessier 1989).

The feldspars are framework silicates and their structure is actually too dense to be transformed and K, Na, and Ca cannot be exchanged without severe destruction of the mineral structure (Robbert and Tessier. 1989). Therefore feldspar weathering involves destruction of it's structure and rebuilding of new minerals. In other words, feldspars weather via a non crystalline compound i.e, gel, which will evolve according to the prevailing conditions to give different secondary minerals.

It is clear, however, that the weathering processes, whichever way they proceed, all tend to weaken the strength of the rock by reducing both the structural strength of minerals and their mutual bonding strength. Such reactions when they are evenly scattered throughout the rock, result in almost certain in service failure of the material.

From the foregoing, the weathering state or alteration undergone by rock material must be carefully studied before their use in any engineering structure. For this purpose a descriptive classification scheme has been developed by Moye (1955) and has been improved since by many authors such as (Little. 1969, Anon. 1970, 71, Fookes et al. 1971, Dearman. 1976, Baynes and Dearman 1978) and Lumb 1983. Hamrol (1961), Mendes et al (1965), Onodera et al (1974) and Irfan and Dearman (1978a, 1978b) using simple engineering tests such as the quick water absorption, porosity, relative density, Schmidt hammer, point load, seismic velocity to evaluate the degree of weathering and give to the former weathering scale a quantitative aspect.

3 5- **Quantitative Petrological Characterisation of Weathering :**

The engineering properties of rocks largely depends on many petrological factors such as mineralogical composition, grain size, texture and structure, including volume of microcracks and the nature of their infillings. Petrological characterisation aims to establish a simple index upon which sound judgement of the material performance can be based, by compiling the factors which have the most detrimental effect on the engineering behaviour of the rock.

Weathering is the most significant of the deleterious attributes affecting rocks. Many authors have established different indices from which the state of weathering can be estimated.

Lumb (1962) used the weight ratio of quartz and feldspar in the original and decomposed granite to define a weathering index:

$$X_d = N_q - N_{q0} / 1 - N_{q0}$$

where X_d is the weathering index, N_q is the weight ratio of quartz and feldspar in the weathered rock and N_{q0} is the weight ratio of quartz and feldspar in the original rock. In similar vein, Onodera et al (1974) showed that ratios such as Na_2O/K_2O, $Al_2O_3/(SiO_2 + Al_2O_3)$, and $(Na_2O + K_2O + Ca0 + MgO)/Al_2O_3$. can be employed as chemical weathering indices. Good correlation between these indices and the modulus of elasticity of the material has been demonstrated by the authors. Moore and Gribble (1978), in studying crushed granite aggregate from Peterhead, used Fe_2O_3/FeO ratio as an index of weathering and concluded that this ratio increases with the increase in the degree of weathering. Based on laboratory results they proposed the ratio of 150 % as a cut-off level beyond which the crushed rock aggregate should be rejected.

Weinert (1964) used the amount of secondary minerals present in the rock, estimated from modal analysis of thin sections (Chayes and Fairbairn 1951) as a weathering index. The classification established, relates well to the material performance in service. Onodora et al (1974) used only microcrack porosity as an index of physical weathering and also found a good correlation with the modulus of elasticity of rock.

Mendes at al (1966) established a more general and representative weathering index. This was defined as the ratio of sound to unsound constituents in the rock, including microcracks, voids and minerals which have a detrimental effect on the rock quality. The quality index K is :

$$K = \frac{\sum_{i=1}^{n} PiXi}{\sum_{j=1}^{m} PjYj}$$

X_i is the percentage of sound minerals or minerals having a favourable influence on the mechanical behaviour of the rock, Y_j is the percentage of altered minerals or minerals which, although sound, have a detrimental effect on the mechanical properties of the rock together with features such as open and infilled fissures and voids. P_i and P_j are weights which measure the influence of one or other peculiarity on the mechanical characteristics of the rock. The application of the method to granite and granite gneiss yielded a good correlation between K and the modulus of elasticity E.

Following the previous authors Irfan and Dearman (1978) established a micropetrographic index (I_p):

$$Ip = \frac{\%\ \text{of sound constituents}}{\%\ \text{of unsound constituents}}$$

Where sound constituents are the fresh primary minerals and the unsound constituents are the secondary products of alteration. The correlation between this Ip and weathering grade classification recommended by the Engineering Group of the Geological Society Anon (1970) confirmed the value of Ip as a weathering index. Moreover, a high degree of correlation was obtained when Ip is compared with standard engineering index and design values.

Estimation of the amount of primary and secondary minerals present is based on the standard petrographic modal analysis of Chayes and fairbairn (1951), and the recognition criteria were those used by (Weinert 1964). Dixon (1969) estimated the index of physical weathering from the number of fractures encountered in a squared traverse of 10 mm side in a thin section. The results obtained correlated with the elastic constant (E) of the material. In addition to the crack number by unit area or volume, Simmons et al (1975) modified this approach by taking into consideration crack dimensions, orientation and distribution. The results obtained also exhibited a good correlation with the elastic properties of the material. To facilitate the recognition of cracks they impregnated the rock with furfuryl alcohol and hydrochloric acid before preparing the thin section.

3 5 1- **Calculation of the Micropetrographic Index**

In the present study the petrographical index Ip has was selected as the weathering index to characterise quantitatively the weathering stages encountered in the investigation. The index is calculated from the formula:

$$Ip = \frac{\text{\% of sound constituents}}{\text{\% of unsound constituents}}$$

The sound and unsound constituents have been counted on standard thin sections prepared from selected samples. The sound constituents are the primary minerals, usually with sharp borders and bright and clear colours with the exception of opaque ores. In the case of basalt and quartz-dolerite primary minerals are olivine, pyroxene, plagioclase-feldspar, apatite needles and iron ore, and quartz in the quartz dolerite. Unsound constituents, on the other hand, are the secondary minerals (alteration products), and voids and cracks. The secondary minerals usually have ill-defined boundaries and have a cloudy appearance. For the quartz dolerite the secondary minerals are chlorite, calcite, anatase, serpentine, iddingsite...etc. For the basalt on the other hand the secondary minerals are all vesicle filling materials, devitrified volcanic glass, chlorite, zeolites, saponite, calcite and clay minerals ...etc.

The standard modal analysis method (Chayes and Fairburn 1951) was used to count the amount of the different constituent. For this, a mechanical stage holding the thin slide, mounted on a petrological microscope and controlled from a Swift Point Counter was used. The mechanical stage constrains the slide to move in orthogonal traverses which facilitates the counting. An average of 300 points was recorded from each thin section and the volume percent of the constituent were automatically read on the counter screen. Tables (3. 3, 3. 4, 3. 5) lists the results of Ip calculations and the volume of secondary minerals with the macroscopic description of every weathering state encountered. It should be noted that the higher the numerical value of Ip the fresher the rock.

3 6- **Quantitative Physical and Mechanical Characterisation of Weathering:**

As previously outlined the mineralogical and structural changes during weathering are believed to affect the physical and mechanical characteristics of rocks. The state of weathering, then, may be estimated from simple physical or mechanical index tests. According to Cottis et al (1971) such index tests should be,

1- Rapid and simple, involving a minimum of specimen preparation.

2- Relevant to rock properties.

3- Relevant to engineering problems, and

4- Capable of discriminating between grades of engineering significance.

3 6 1- **Quantitative Physical Index Tests:**

Porosity, water absorption, and density alter significantly when a rock undergoes weathering. In the fresh state igneous rocks generally possess low porosity, low water absorption and high density. On weathering, the fracturing of the rock together with the voids created after the removal of the soluble material by water, lead to an increase in the porosity and the value of water absorption. Conversely density decreases, due to the lighter nature of the secondary minerals and the generation of voids (Plate 3. 7, 3. 8).

The sensitivity of these physical indices to the mineralogical and structural variations attending weathering and/or alteration processes is well demonstrated by either comparing Ip with porosity and water absorption Fig (3. 2) or comparing the volume of secondary constituents (secondary minerals + cracks + voids or secondary minerals + vesicles + amygdales) with density (d) Fig (3. 3), porosity (n) Fig (3.4), and water absorption (Wab) (Fig 3. 5a,b). The high correlation coefficients 0.89 - 0.90 indicate the sensitivity of these indices.

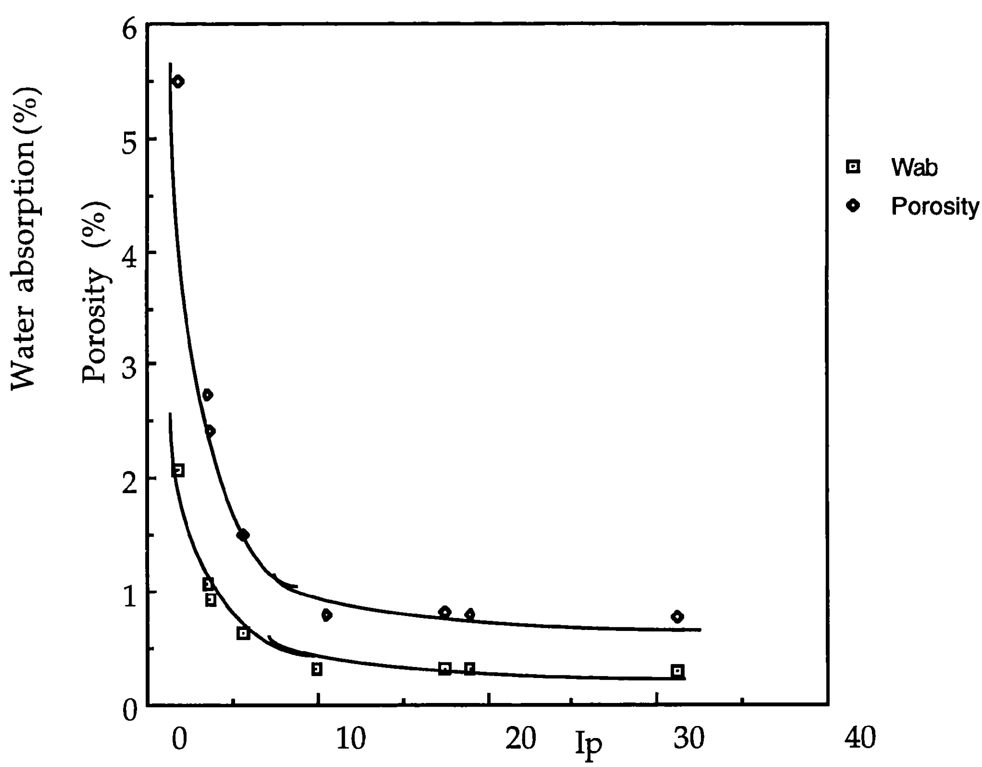


Fig 3.2 The relationship between Water absorption Porosity and the micropetrographic index (Ip) for the studied granite

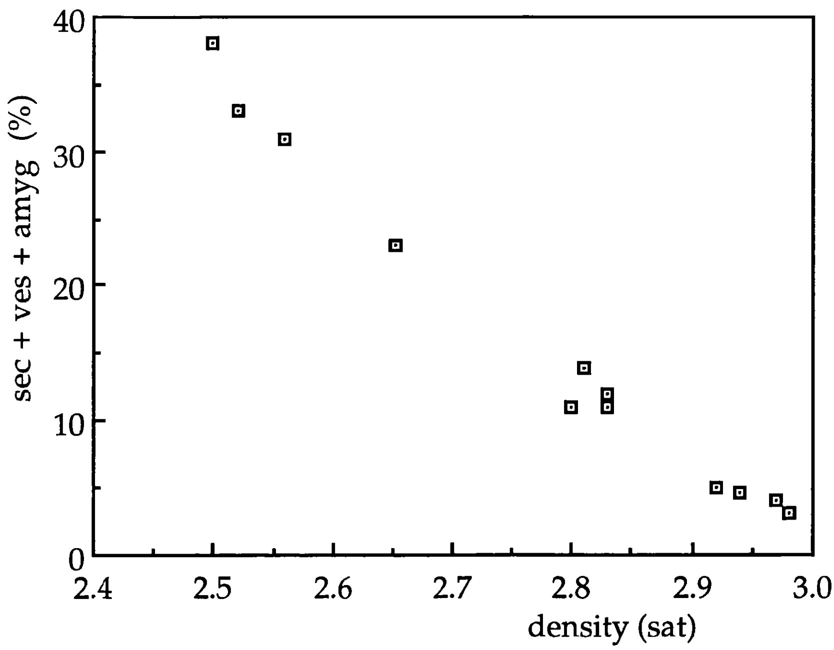


Fig 3.3 The relationship between the volume of secondary constituent of the rock and density for basalt

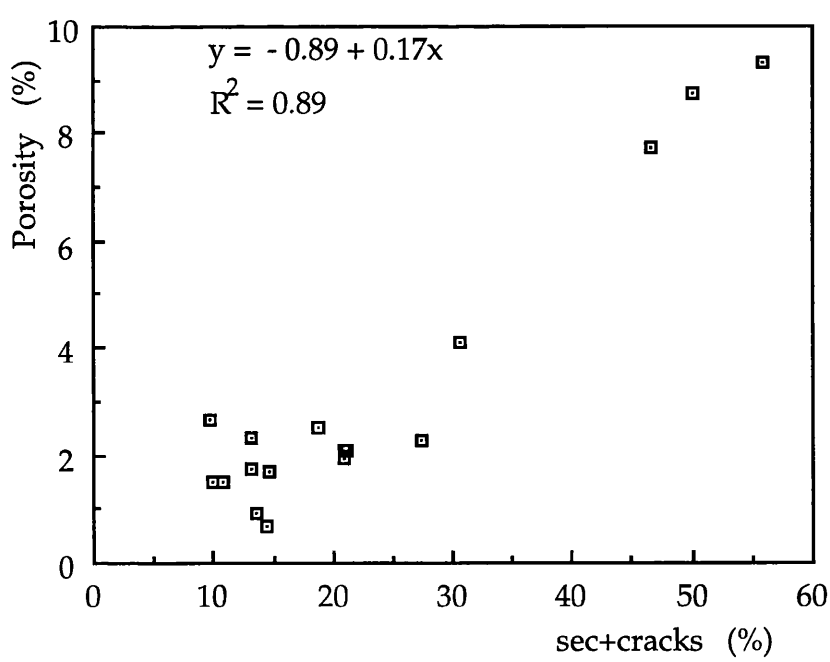


Fig 3.4 The relationship between porosity and secondary constituent for quartz dolerite

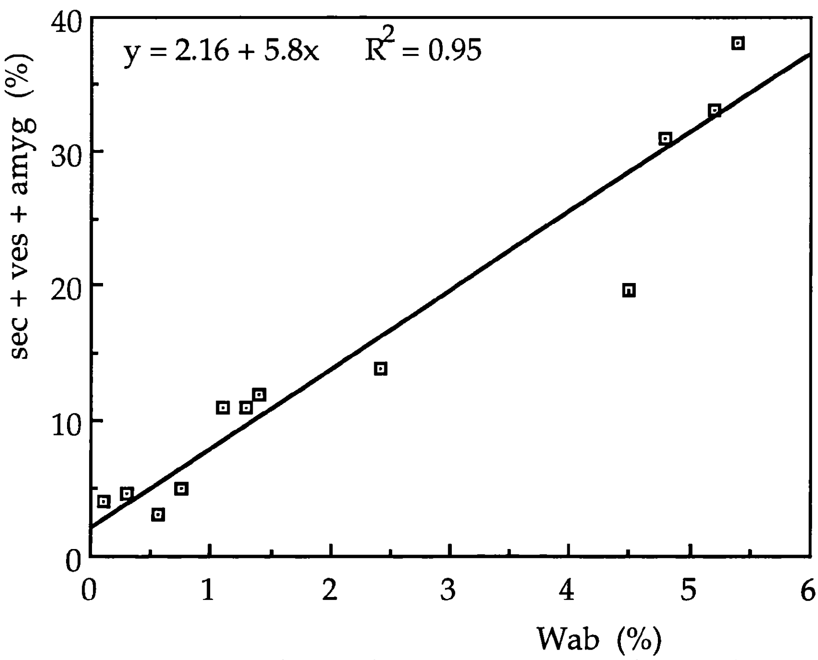


Fig 3.5a The relationship between the volume of secondary constituent and water absorption for basalt

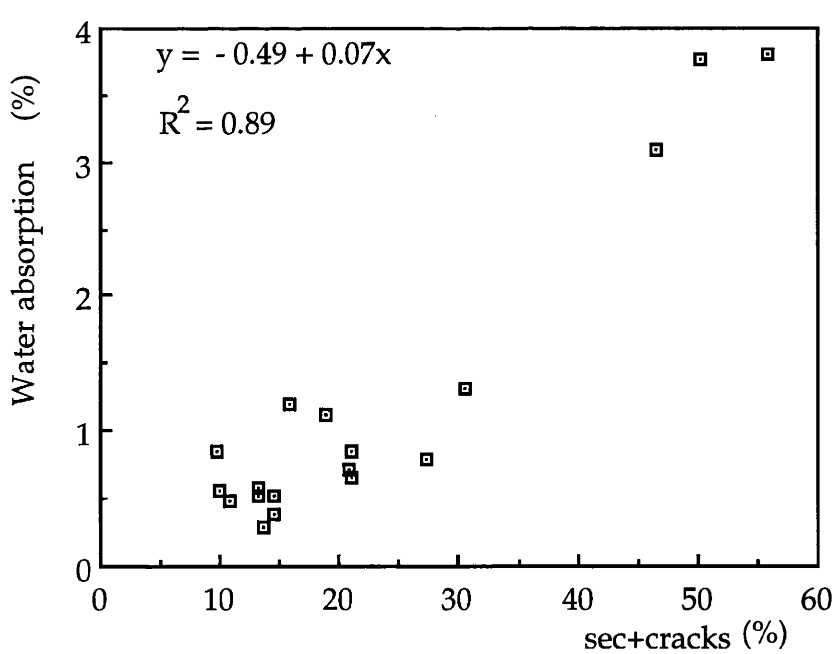


Fig 3.5b The relationship between water absorption and secondary minerals +cracks in quartz dolerite

In addition to their sensitivity to the mineralogical and structural changes in rocks, physical indices are also sensitive to the changes affecting their mechanical properties as a consequence of weathering. Onodera et al (1974) established a relationship between porosity and strength of sandstone. The relationship is of the form:

$$n = a\sigma^{-b}$$

n = porosity

σ = UCS

a, b = constants

He concluded that porosity also affects the mechanical and elastic properties such as young modulus (E), shore hardness and triaxial shearing. A similar relationship was obtained in the present study by plotting the uniaxial compressive strength of granite and basalt against their respective porosity values (Fig 3. 6 and 4. 7).

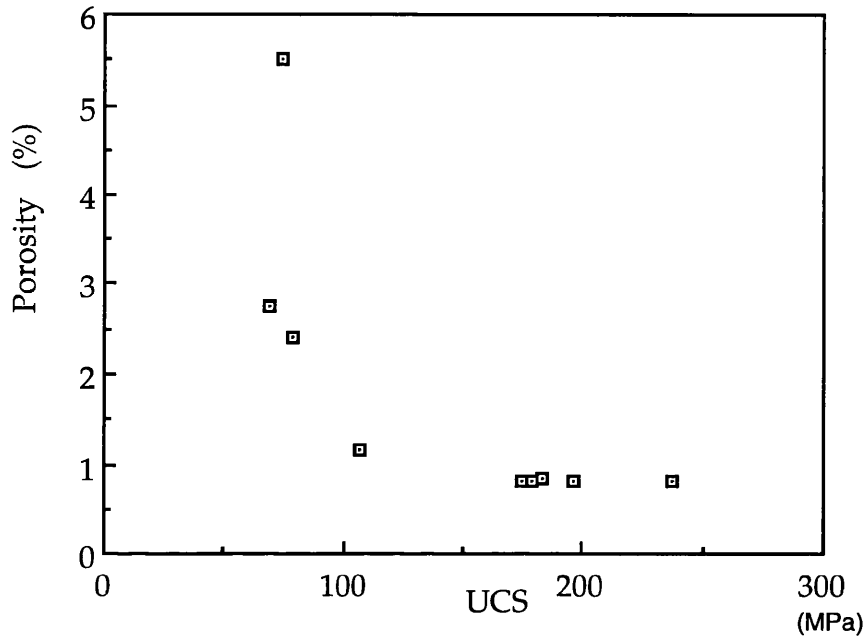


Fig 3.6 The relationship between porosity and uniaxial compressive strength for granite

3 6 2- **Quantitative Mechanical Index Tests:**

To characterise the weathering state in terms of the fundamental design parameters, material strength, and elasticity a suite of mechanical index tests such as Schmidt rebound number, point load strength, uniaxial compressive strength, ultrasonic pulse velocity and dynamic Young Modulus have been performed on samples of quartz dolerite, basalt and granite exhibiting different weathering states.

As explained earlier, the process of intergranular bond weakening, crack opening, and development of structurally weak secondary minerals, upon weathering, results in a general decrease in the rock strength and elasticity. Subsequently, the mechanical test values such as Schmidt Rebound Number, Point Load Strength, Seismic Velocity, and Vicker's Indentation Hardness, theoretically, should decrease. The correlation between Schmidt Rebound Number and the petrographic index (Ip) Fig (3.

7a, b) shows that as Ip increases, i.e. the rock becomes fresher, the rebound number increases. The logarithmic relationship obtained expresses the sensitivity of this index test. It can be clearly seen that below Ip = 2.5 for quartz dolerite and 3.5 for granite, there is a sharp decrease in the rebound number. This point corresponds to the state of weathering where the rock loses it's strength and become weak. The point load strength index when plotted against the volume of secondary constituents similarly shows that the rock becomes weaker as the percentage of the weathering products in the rock increases Fig (3. 8). This is interpreted as a general bond loosening between the material constituents, due either to crack development or weakening of the static forces between the minerals resulting from the replacement of the alkali metals by OH^-. The positive exponential relationship between the Point Load strength and Ip with high coefficient of correlation shows the sensitivity of the test to the development of deleterious constituent within the rock (Fig 4. 6). Lumb (1983) in studying the engineering properties of decomposed igneous rock from Hong Kong found the point load to be very useful in discriminating between rippable and non-rippable material. The point load value for such a practice however was 2.5 MPa.

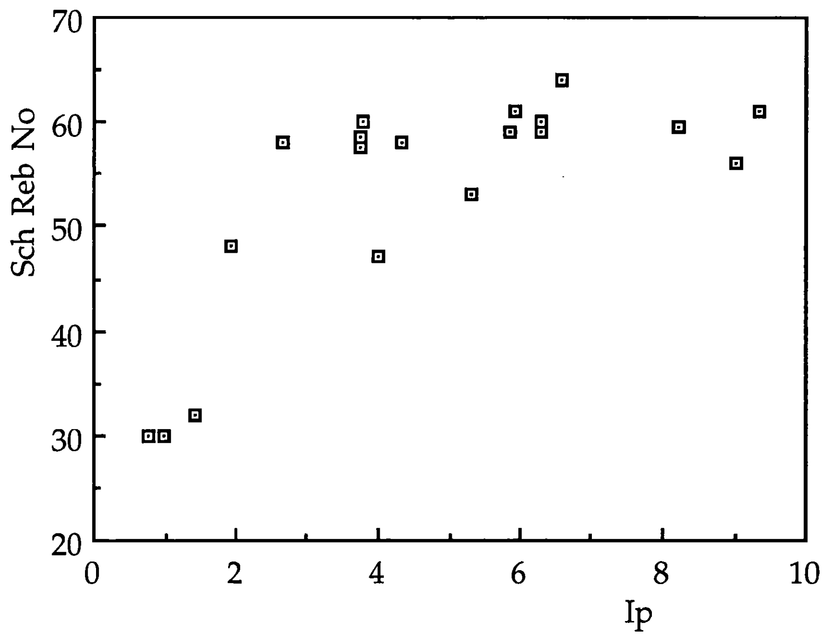


Fig 3. 7a The relationship between Schmidt Rebound Number and the micropetrographic index for quartz dolerite

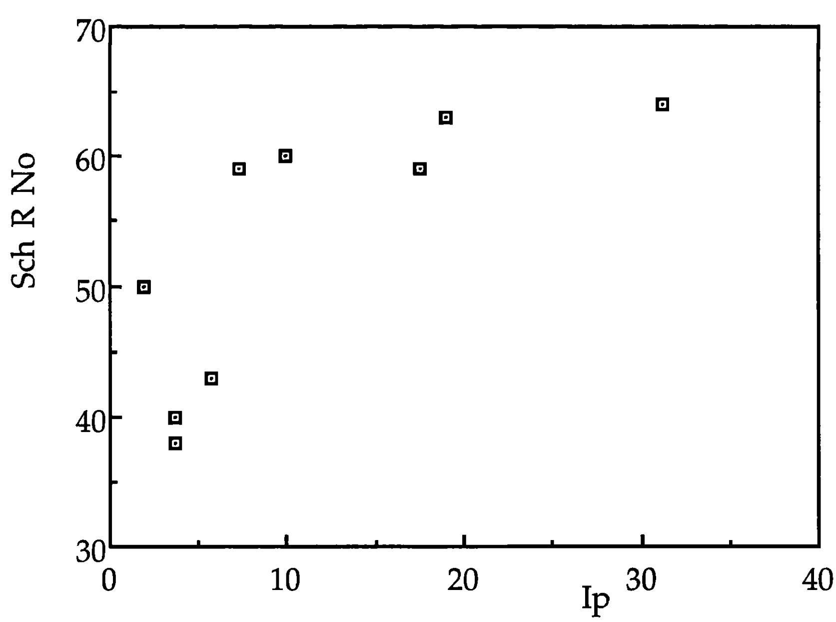


Fig 3.7b The relationship between Schmidt Rebound Number and the micropetrographic index (Ip) for granite

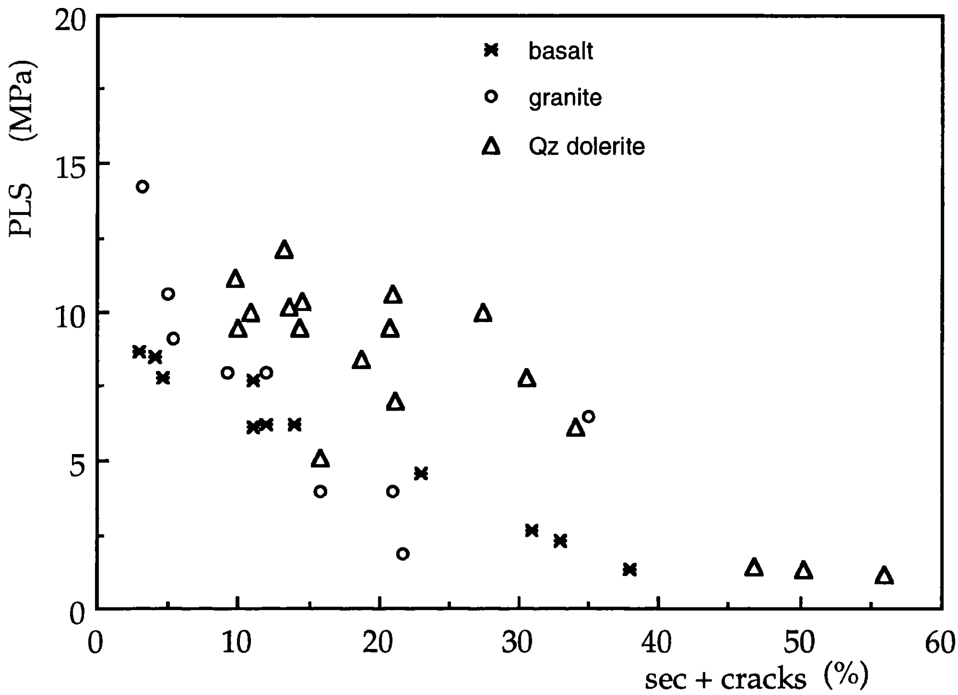


Fig 3. 8 The relationship between Point Load Strength and volume of secondary constituent for dolerite granite and basalt

The uniaxial compressive strength which is the most commonly performed engineering test on rock, has been found to be very sensitive to the decomposition of the rock. For basaltic material, the UCS gives very distinctive values for different weathering stages. The fresh, dense and vesicles/amygdales free basalt is characterised by very high UCS values of the order of 370 MPa while the very weathered and rotten material has a UCS as low as 20 Mpa. Ip values representing the whole weathering spectrum presented by basalt, when plotted against their UCS values show that the latter increases in an exponential manner as the Ip value increases (Fig 3. 9).

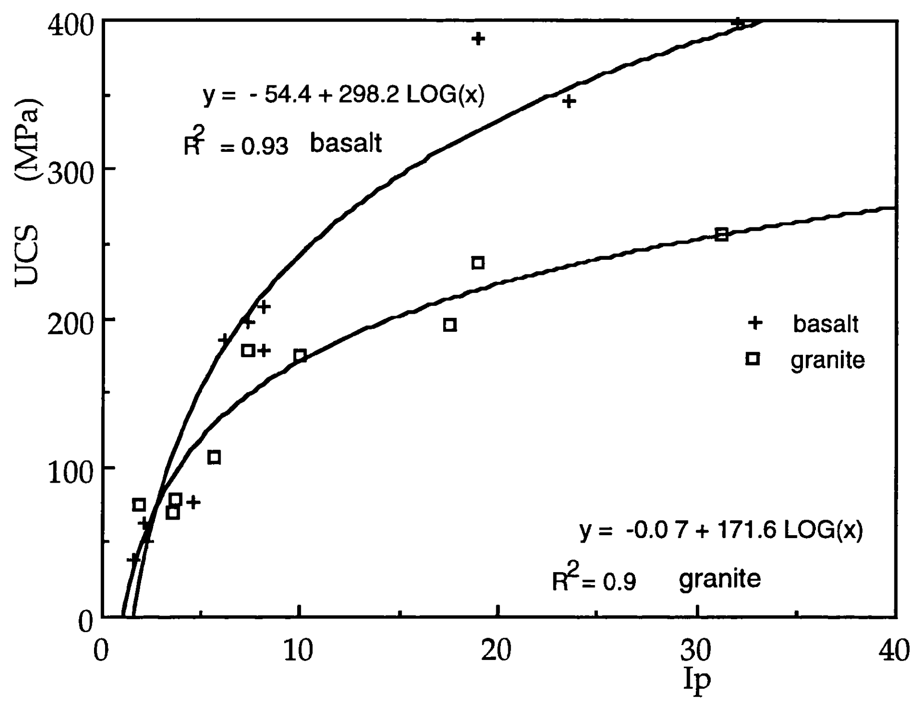


Fig 3.9 The relationship between the UCS and the micropetrographic index for granite and basalt

The ultrasonic pulse velocity has been successfully used by Iliev (1966) as a weathering index (Table 3. 1). He showed that the factor K which is:

$$K= (V_0 - V_W)/V_0$$

Where V_0 = Ultrasonic pulse velocity of fresh granite and V_W = Ultrasonic velocity for weathered granite, correlates well with the degree of weathering as assessed visually in the field. The decrease of the ultrasonic velocity as explained by Iliev (1966) is due to the increase in the proportion of voids and cracks determined as porosity.

Descriptive terms	Ultrasonic velocity m/s	Coefficient of weathering K
Fresh	> 5000	0
Slightly weathered	5000 - 4000	0 - 0.2
Moderately weathered	4000 - 3000	0.2 - 0.4
Strongly weathered	3000 - 2000	0.4 - 0.6
Very strongly weathered	< 2000	0.6 - 1.0

Table 3.1 Quantitative classification of the degree of weathering in monzonite (after Iliev 1966)

This technique is in fact an indirect indication of the variation of the elasticity of the material with weathering, since Edy (dynamic modulus of elasticity) is proportional to the square of the velocity times density. However, in the present study a direct comparison between the product of weathering and the ultrasonic velocity of weathered rocks has been carried out. Fig (4. 29) is a plot of ultrasonic velocity against the volume of secondary constituents and shows that an increase in the latter results in a decrease in the velocity of the supersound. In this study the alteration product which contributed most to reducing the elastic wave propagation velocity in the rock was cracks. A hydrothermally altered sample of granite, although it has about 35 % secondary minerals and is mostly crack free, has a relative density of 2.52 and a velocity of 4700 m/s. Other samples have higher relative densities 2.59 - 2.63, secondary mineral content of about 8 % to 10 % but 10 to 15 % cracks have velocities of about 2800 m/s (Table 4. 2).

This confirms the comment that elastic wave velocity is more affect by the presence of discontinuities in the rock than merely presence of lower density secondary minerals. Onodera et al (1974) showed that the dynamic Young's modulus decreases sharply when the rock undergoes weathering and loses it's cohesion, in other words when it reaches the weakening stage.

3 7- **Conclusion**:

As already discussed, the more reliable physical and mechanical tests for assessing weathering which comply most closely with the requirements of Cottis et al (1971) for index properties are the quick water absorption test, Schmidt Rebound Number and Point Load strength. Furthermore these index tests can be used with confidence to calculate other rock properties as demonstrated by Deer and Miller (1966) and Irfan and Dearman (1978) and this study.

The Point Load strength Test has several favourable criteria which qualify it as a weathering index. It gives very reliable and reproducible results over the entire weathering spectrum, and over a wide range of test-specimen sizes. Moreover other rock properties such as uniaxial compressive strength, tensile strength can be estimated with a fair accuracy. Lumb (1983) from Hong Kong has demonstrated that the test is very effective in discriminating between sound and unsound rock. It is a standard test (ISRM 1978).

The Schmidt Rebound test also has several advantages which qualify it as a weathering index test. In addition to the ease with which the test can be carried out everywhere, it has shown a systematic and sensitive relationship to the volume of the secondary constituents in the rock. It can also be used to estimate several other rock properties such as uniaxial compressive strength and tangent Young Modulus (Deer and miller 1966, Irfan and

Dearman 1978). It is also very useful in the field for obtaining *insitu* estimates of rock strength from the rock surface. In comparison with the ultrasonic velocity the Schmidt Rebound Number shows a linear relationship over the entire weathering spectrum has been obtained.

The water absorption test or the quick water absorption of Hamrol (1961) has been widely used as a weathering index Deer and Miller, 1966, Sarafim 1966, Irfan and Dearman 1978. In addition to a good linear relationship with the volume of secondary constituents of the rock, it can be used with accuracy to predict other rock properties such as Schmidt Hammer, Point Load, Ultrasonic Velocity and the weatherability and weathering state of the material.

The combination of these three engineering quality index tests can be used with a high degree of confidence to assess the strength and to estimate the state of weathering and weatherability of the rock material. The uniaxial compressive strength which is widely used in engineering practice can be estimated with some confidence from the Schmidt Rebound Number and the Point Load Strength (Deer and Miller, 1966) and the present investigation (Fig 3. 10). The coefficient of correlation obtained from the relationship of water absorption and the other tests such as Uniaxial Compressive Strength, Point Load, Schmidt Rebound Number and the ultrasonic velocity confirm it's reliability in predicting these index values.

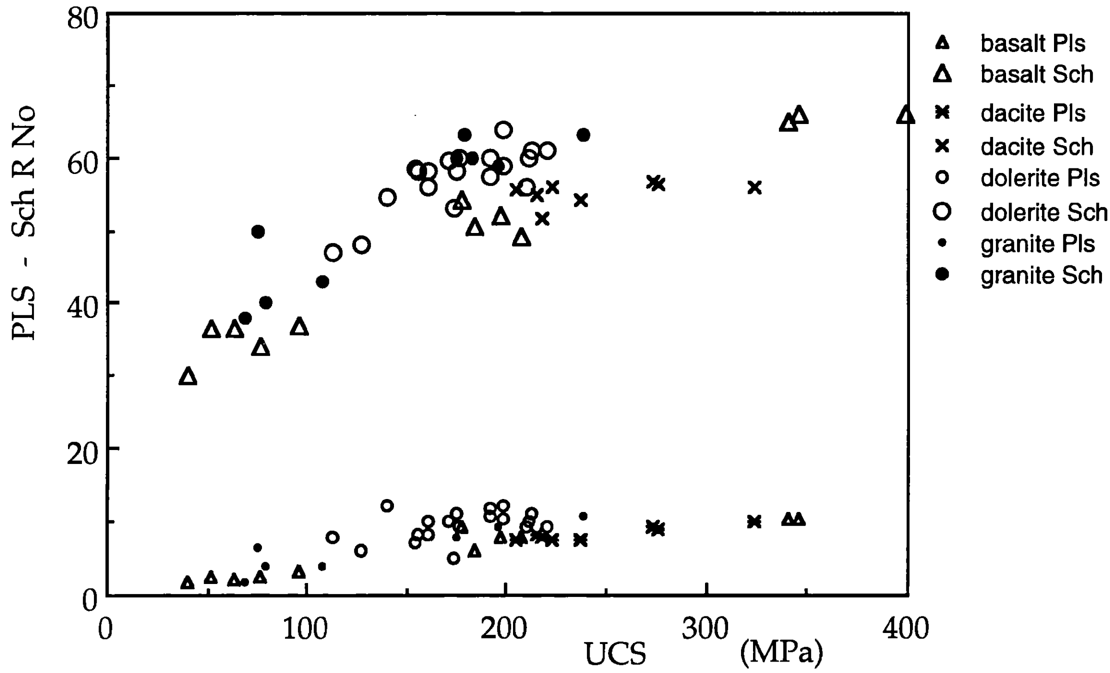


Fig 3.10 The relationship between Point Load Strength - Schmidt Rebound Number and the Uniaxial Compressive Strength

Weathering grade classification

Table 3. 2. Intact rock weathering classification (after Anon 1977)

Term	Grade	Description
Fresh	IA	No visible sign of rock material weathering
Faintly weathered	IB	Discolouration on major discontinuity surfaces
Slightly weathered	II	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker than in its fresh condition.
Moderatly weathered	III	Less than half of the material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.
Highly weathered	IV	more than half of the rock is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corstones.
Completly weathered	V	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
Residual soil	VI	All rock material is converted to soil. the mass structure and material fabric are distroyed.There is a large change in volume, but the soil has not been significantly transported

Weathering classification of Basalt

Table 3. 3. weathering grade classification of the studied basalt

Mass weathering	Description	Sec M	Ip value
Fresh Grade I	Black, greenish in colour, compact basalt fine grained to porphyritic, no staining, very little alteration, very few amygdales very high strength, Sh= > 60	3 - 5	19 - 33
Slightly weathered Grade II	Greenish in colour, very slightly stained fine to coarse porphyritic material presence of few amygdales, high to very high strength, Sh= 50 - 54	up 14	6
Mod weathered Grade III	Reddish and greenish, highly stained, coarse grained highly amygdaloidal and vesicular, devitrified glass, altered pyroxene, feldspar partially altered, calcite, chlorite, zeolite clay minerals, moderate to high strength Sh= 30 - 36	33 - 35	1.85 - 2.03
Highly weathered Grade IV	Red brown, completly stained, completley altered and decomposed, low strength, calcite, clay minerals, iron ore, Sh= 30 and below	> 50	< 1
Complet weathered Grade V-VI	soil and lateritic material.	nd	nd

Sec M = secondary minerals + cracks + voids + amygdales.

Mod = Moderately

Complet = Completely

Sch = Schmidt Rebound Number

Weathering grade classification for quartz dolerite

Table 3. 4. Weathering grade classification for quartz dolerite

State of weathering	Description	Grade	UCS (MPa)	PLS (MPa)	Sch R Nber	R. Density	Por (%)	AIV	LAAV	Ip
Fresh	Not stained	I	198 - 212	12	60 - 65	2.93	1.5	7 - 8	11	> 6
Slightly weathered	Slightly stained	IIi - II ii	170 - 190	9-10	58 - 62	2.86 - 2.9	1.5 - 1.7	8 - 10	10 - 13	4 - 8
Moderately weathered	Partly to comp stained	IIiii - III	100 - 150	5 - 10	53 - 57	2.6 - 2.8	2.5 - 4	12	10 - 18	2 - 4
Compl weath, weakened	Comp stained and rotted	III - IV	nd	1 - 4	10 -36	2.3 - 2.6	7 - 9	> 20	37 - 66	< 2
Compl decomposed	Sandy like soil+plastic clay	V - IV	nd	nd	nd	nd	nd	nd	nd	nd

Por (%) = porosity

Compl = Completely

weath = weathered

nd = not determined

Weathering grade classification for granite

Table 3. 5. weathering grade classification for the studied granite

State of weathering	Description	Grade	UCS (MPa)	PLS(MPa)	Sch hammer	R.Density	Porosity	Ip	AIV	LAAV
Fresh	Not stained	I	180 - 260	10	60 - 64	2.63-2.67	0.8-0.84	>11	15-20	23-30
Slightly weathered	Slightly stained	IIi - II ii	175	5.13	58- 60	2.61	0.85	9	25	45
Moderately weathered	Partly to comp stained	IIiii - III	107	3.94	45 - 50	2.60	1.17	5	nd	nd
compl weath, weakened	Compl stained and rotted	III IV	70-80	1.86	< 40	2.59	2.4-2.8	3.5	50(mod)	62
Granitic soil	Soil	nd	nd	nd	nd	nd	nd	nd	nd	nd

Compl = Completely

weath = weathered

nd = not determined

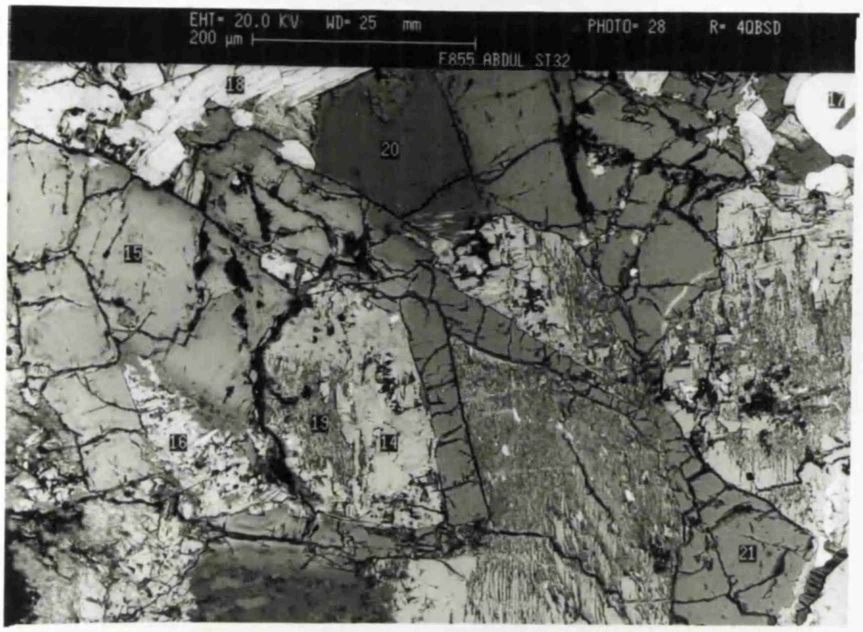


Plate 3. 1. SEM photomicrograph showing inter and intragranular microcracks in quartz dolerite

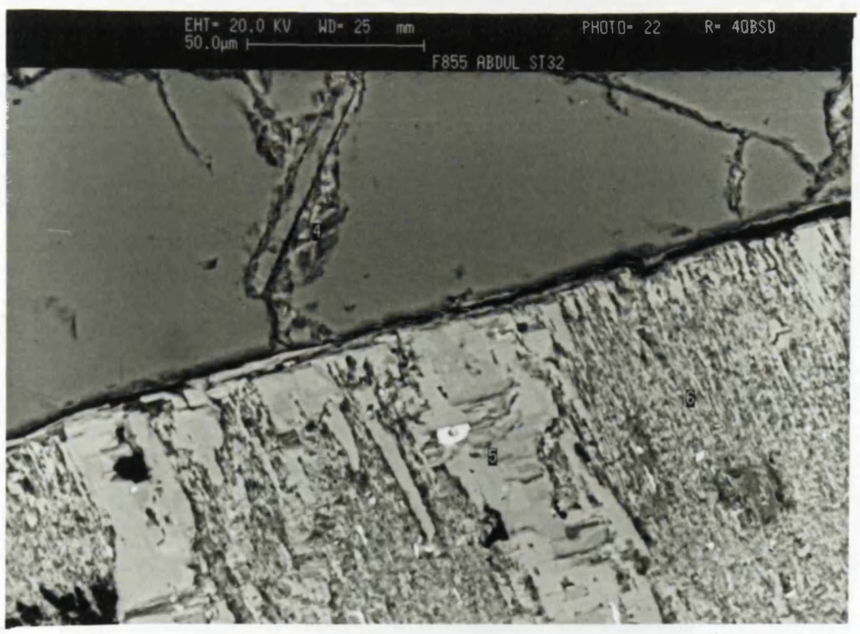


Plate 3. 2. SEM photomicrocraph showing grain boundary crack between plagioclase and pyroxene in quartz dolerite

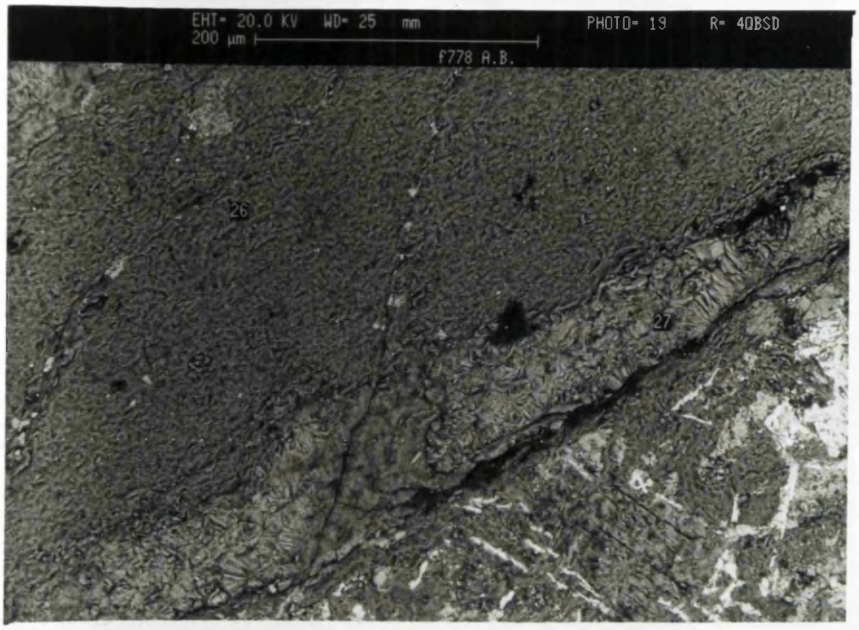


Plate 3. 3 SEM photomicrograph showing grain boundary cracking and alteration in basalt. 27- chlorite, 26 plagioclase

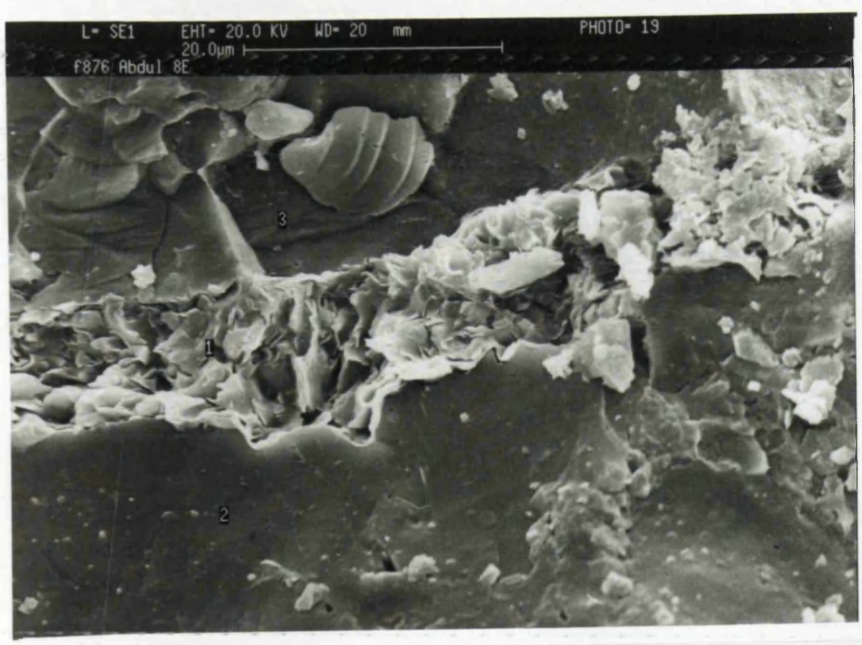


Plate 3. 4 SEM photomicrograph showing illite filled microcrack in basalt, 2 and 3 are volcanic glass

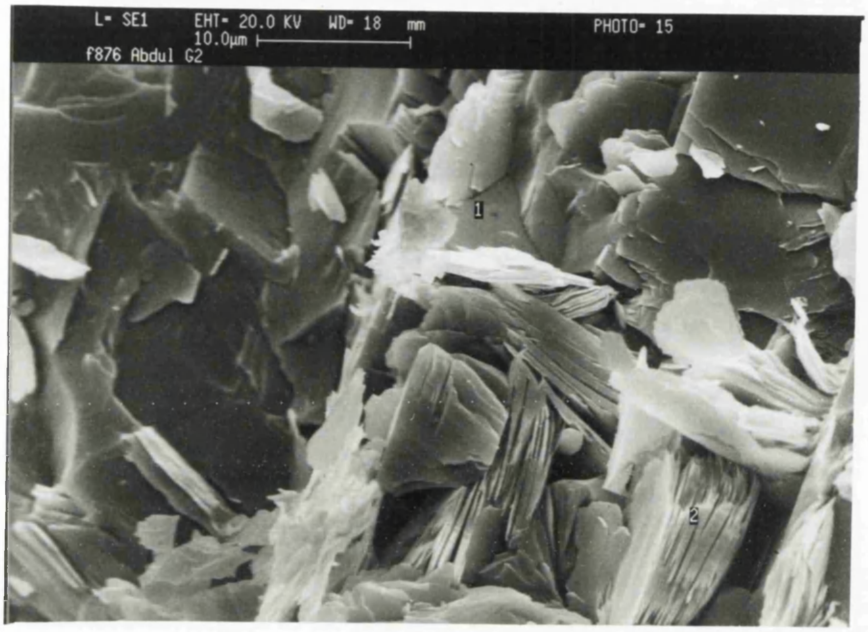


Plate 3. 5 SEM photomicrograph showing the development of secondary minerals, Kaolinite as alteration product of feldspar in granite from Dalbeattie

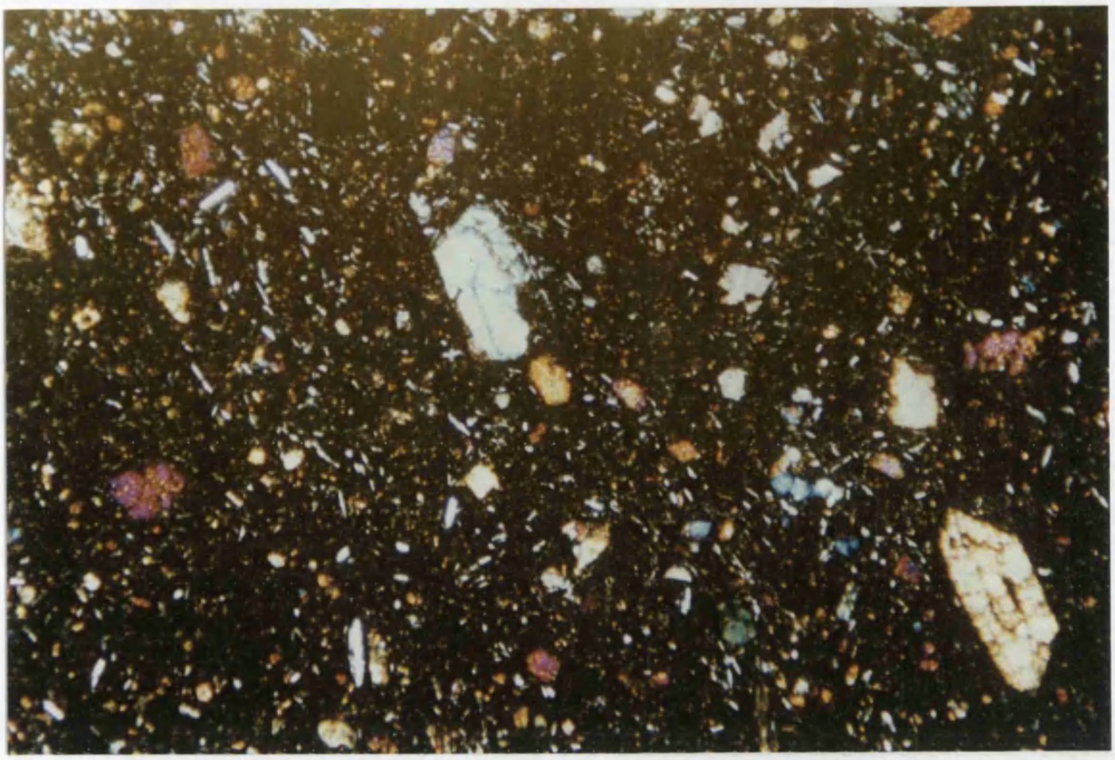


Plate 3. 6 Photomicrograph showing fresh microporphyritic basalt scale x 25

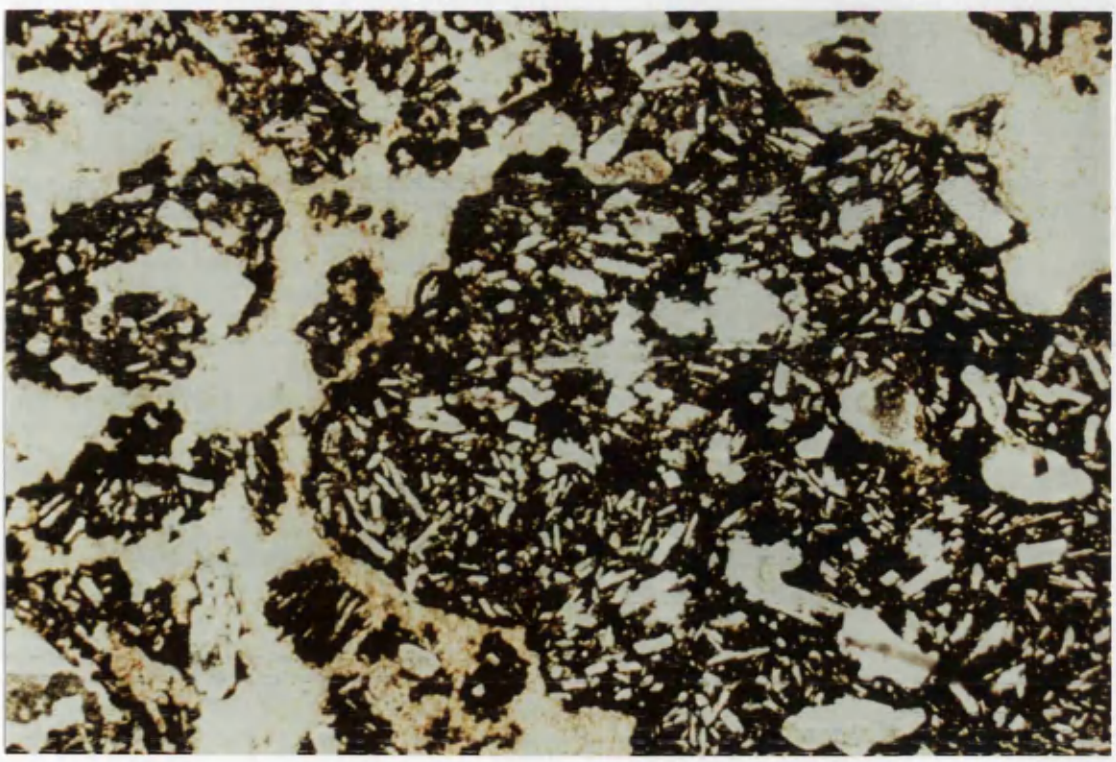


Plate 3. 7 Photomicrograph showing a highly weathered amygdaloidal and vesicular basalt from Beith scale x 25

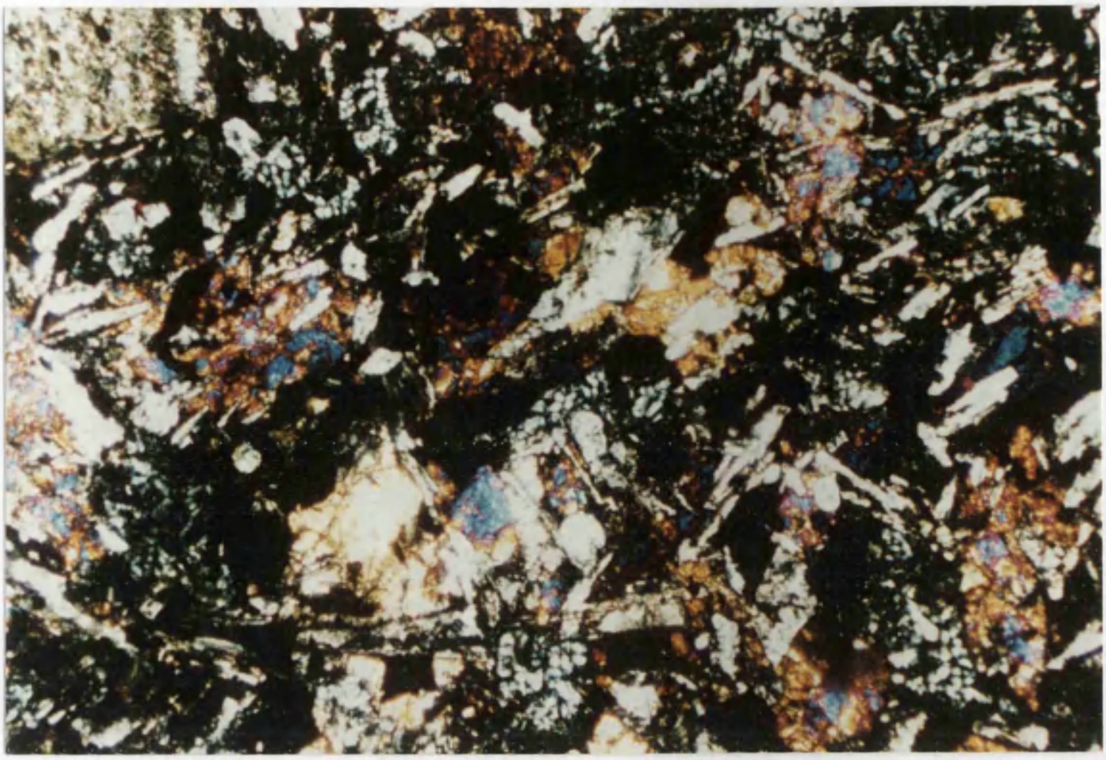


Plate 3.8 Photomicrograph showing the devitrification of basaltic glass to a greenish aggregate (chlorite) from Beith. scale x 25

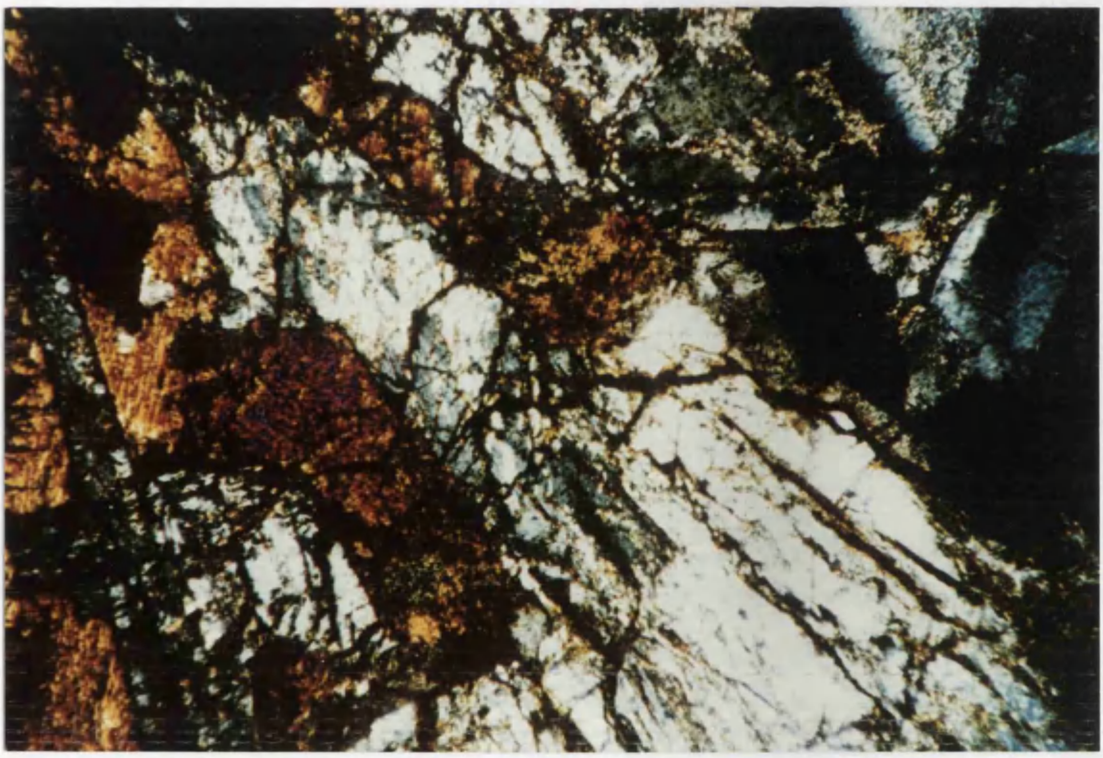


Plate 3. 9 Moderately altered and highly fractured quartz dolerite scale x 25

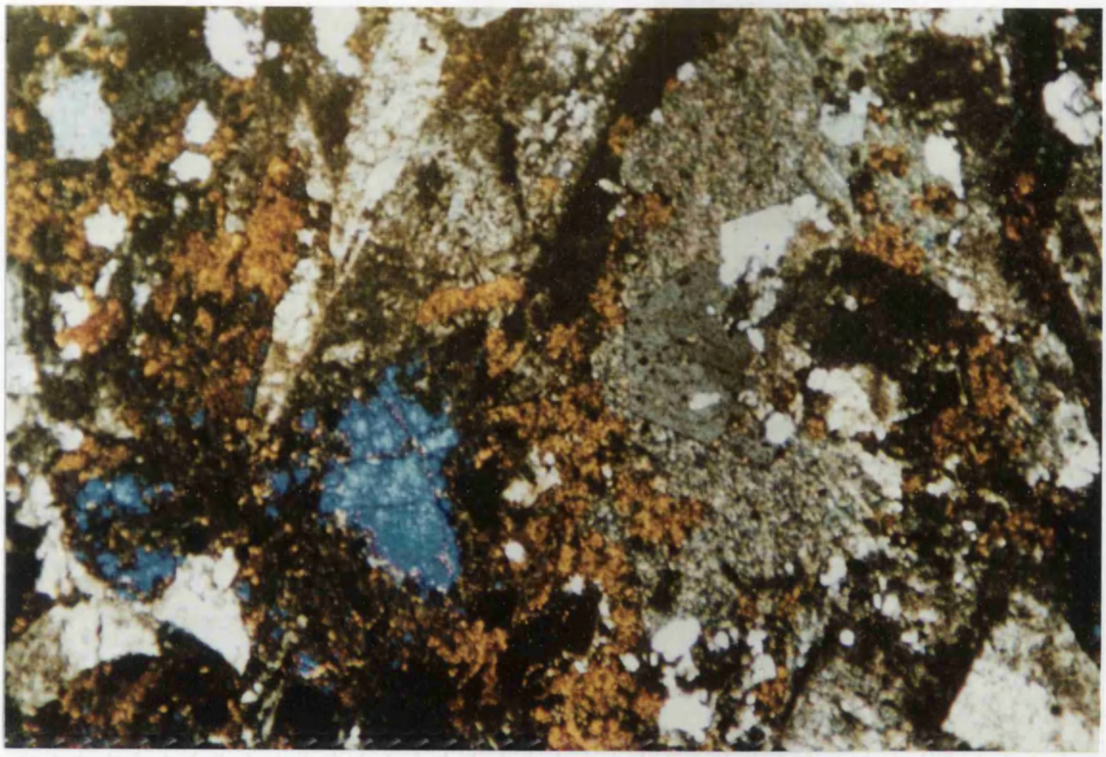


Plate 3. 10 Photomicrograph showing a slightly altered quartz dolerite scale x 25

CHAPTER IV

Methodological Study of the Los Angeles Abrasion Test

4 1- Introduction:

The Los Angeles Abrasion Test (ASTM C131 1976) for small size coarse aggregate and (ASTM C535 1976) for large size coarse aggregate is designed to measure the resistance to abrasion and impact in the dry state. The wear and impact is induced by mutual collision and abrasion between aggregate particles and between aggregate and a charge of steel balls.

The test was introduced as an ASTM standard in 1937 to overcome some of the deficiencies found in the Deval Abrasion Test (ASTM D2 and D289). At first it was used for testing 38.1 - 19 mm (11/2-3/4 in) maximum size aggregate (grading A and B) . Later 9.5 mm (3/8 in) and No 4 sizes were added (grading C and D) , then three more gradings for large aggregate size 75 mm (3 in) , 50 mm (2 in) and 38.1 mm (1.5 in) respectively E , F and G grading (1947).

The apparatus for the Los Angeles Abrasion Test is a closed, hollow steel drum of 711 mm (28 in) diameter and 508 mm (20 in) length, fitted with an internal, full length, hardened-steel shelf 89 mm (3.5 in) wide, from which the sample and the steel balls fall in the course of each revolution. A cover plate is fitted snugly into the loading aperture in the drum to ensure that no fine material is lost during the test. The apparatus is sited on a flat, solid floor or concrete base to avoid accumulation of the sample and abrasive charges on one side of the machine . The cylinder rotates about it's horizontal axis at a speed of 30 to 33 revolution per minute driven by a small electric motor.

4 1 1- Test procedure:

The standard procedure for the test is described in the ASTM . C131 . 1976 for small size coarse aggregate and ASTM. C535. 1976 for large size coarse aggregate. The size of sample for the test depends on grading (Table 4.1), i.e. for aggregates in grades A , B , C and D the sample should be 5000 ± 10 gms., while for the grades E , F and G it is 10000 g . Prior to the test the sample should be washed thoroughly to remove fines, dust and coating materials. Following this, the sample must be oven dried at a temperature of 105 to 110 C^{o} to a substantially constant mass, i.e. approximately 16 - 18 hours (Hanks 1962). From the oven the sample should be cooled in a dessicator for a period of 4 hours and then the weight of the sample recorded to the nearest 1g. The sample and an appropriate charge of steel balls (Table 4. 1) is placed in the machine, and the appropriate number of revolutions is selected, i.e. 500 for grades A , B , C and D and 1000 for E , F and G, at a speed of 30 to 33 rev /min. After this operation, the sample is removed from the machine into a metal tray placed beneath the cylinder, and sieved on the No12 sieve (1.7 mm). The fraction coarser than 1.7 mm is then washed, oven dried to a constant weight and weighed to the nearest 1 g .

4 1 2- Calculations:

The amount of fines less than 1.7 mm produced in the test expressed as a percentage of the original sample weight is known as the Los Angeles Abrasion Value (LAAV):

$$\text{LAAV} = \frac{\text{mass of fraction passing 1.7 mm}}{\text{mass of the original sample}} \times 100$$

An additional value proposed in this study is the percentage of material remaining on the original sieves 14 & 10 mm sieve expressed as a percentage of the original weight of the test sample, and termed the Los Angeles Abrasion Value residue (LAAVR):

$$LAAVR = \frac{\text{mass of material retained on 14 and 10 mm}}{\text{mass of the original sample}} \times 100$$

The Los Angeles Abrasion Value is known to be highly repeatable and reproducible (Hanks 1962). Therefore, if the results of two tests differ by more than 5.7 percent (of their average) a new test must be performed.

4 2- **Causes of Variation in the Los Angeles Abrasion Value**:

In the original article on the Los Angels Abrasion Test Woolf and Runner (1935) mentions that an average standard deviation of 3.4 and 2.7 was found. In a later article, Woolf (1936) reported that an interlaboratory mean standard deviation of 5.6 expressed as a percentage of the population mean LAAV of 23 per cent. The interlaboratory tests were conducted under non standard conditions. Hanks (1962) in a study on the reproducibility of the Los Angeles Abrasion Test, demonstrated the need for a complete standardisation of the test. He showed that an error of up to 4 per cent can occur due to differences in test procedures in laboratories where the standard was vague and nothing was laid down to guide the operator . In the Los Angeles Test, the non geological variables which can affect test results arise chiefly from the sample preparation technique, and the condition of the apparatus.

In order to investigate the effect of operating variables on the LAAV, samples of the same grading, and particle shape, i.e. flakiness index (I_F) = 0, were employed. In these conditions the variables which are likely to affect the results are temperature, washing, drying, and the state of the machine.

Table 5. 1. Table showing the gradings used in LAAV and the corresponding number of balls

Sieve Size (Square Opening)		Weight of Indicated Sizes, g Grading						
Passing	Retained on	E	F	G	A	B	C	D
75 mm	63 mm	2500						
63 mm	50 mm	2500						
50 mm	38.1 mm	5000	5000					
38.1 mm	25 mm		5000	5000	1250 ± 25			
25 mm	19 mm			5000	1250 ± 25			
19 mm	12.5 mm				1250 ± 10	2500 ± 10		
12.5 mm	9.5 mm				1250 ± 10	2500 ± 10		
9.5 mm	6.3 mm						2500 ± 10	
6.3 mm	No 4 (4.75 mm)						2500 ± 10	
4.75 mm	No 8 (2.36 mm)							5000 ± 10
Total		10000	10000	10000	5000 ± 10	5000 ± 10	5000 ± 10	5000 ± 10
Number of steel ball		12	12	12	12	11	8	9

a-**Temperature** : The effect of temperature on the Los Angeles Value was revealed by testing three samples of the same quality and grading (Hanks 1962). One sample was tested immediately after removal from the oven, the second after 1 hour later, and the third 4 hours. The results obtained showed that test values increased by a small amount when the material is still hot (Table 4. 2). This behaviour can be explained by the intergranular stress buildup as a result of heterogeneous thermal expansion of the constituent minerals.

Table 4. 2. The influence of cooling time on LAAV (after Hanks 1962)

Operator	Time of cooling	LAAV
A	nil	20.2
A	1 hour	19.6
Standard mean	4 hours	19.4

b- **Washing**: The effect of washing the sample was found to have small effect although in an unknown direction (Hanks. 1962). In our opinion this irrational variation is less a consequence of washing than a normal variation due to the many uncontrolled factors affecting the results.

c- **Drying**: Hanks (1962) reported a significant variation in the Los Angeles Values with the drying time after washing. For example between 4 and 24 hours drying time the values decreased by up to 2 percent in absolute value (Table 4. 3). This decrease in LAAV is due to the weakening of the strength properties induced by the appreciable water content remaining in the material which has been dried for only a short time. In the present study, Fig (4. 1) shows clearly that the LAAV for dry and saturated amygdaloidal basalt can differ by about 6 %. Therefore, aggregate must be oven dried to a constant weight, i.e. 24 hours, to avoid the effects of water particularly in weathered rocks where these effects may be more dramatic, i.e. higher water content and weaker rocks.

Table 4. 3 The influence of drying time on LAAV (after Hanks 1962)

Operator	Time of drying	LAAV
B (test 1)	4 hours	20.5
B (test 2)	4 hours	20.1
B	16 hours	18.9
B	24 hours	18.4
Standard mean	24 hours	19.4

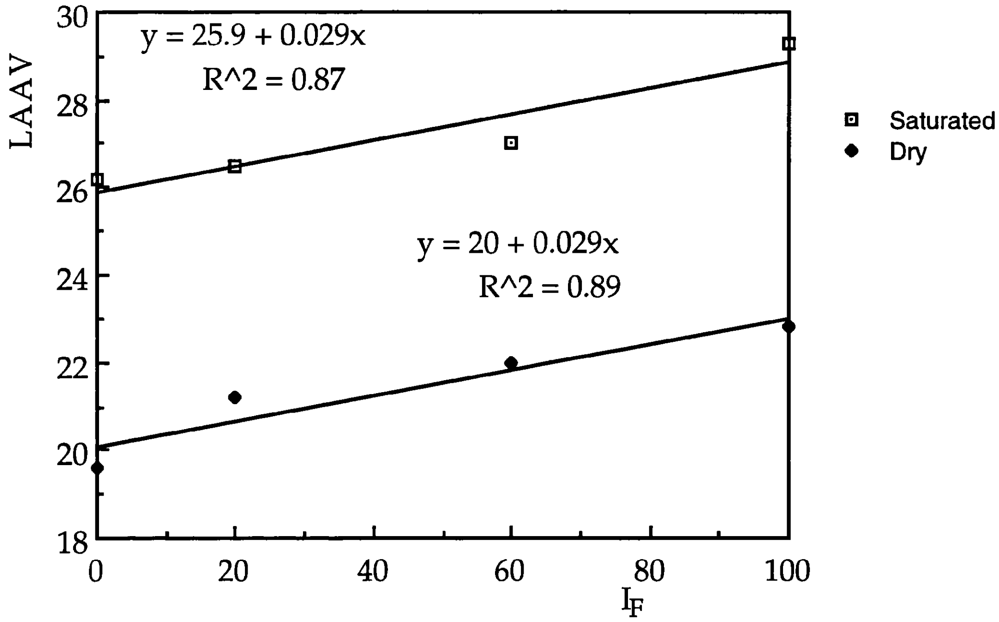


Fig 4.1 Graph Showing the influence of water on the LAAV for basalt

d- **Condition of the Apparatus**: The internal shelf of the machine is subject to severe surface wear from the pounding by the balls and aggregate, causing a ridge to develop towards the inner surface of the cylinder. In addition the shelf itself may be bent longitudinally or transversally from its initial position. Although the effect of this ridge is not yet known, in the interests of consistency and standardisation, it is recommended that the condition of the shelf is periodically checked. Where it is damaged the shelf should be repaired or replaced before any further test is performed. During the regular check of the shelf any ridge developed must be ground off if it exceeds a height of 2 mm (ASTM. C131. 1976).

e- **Test Procedure**: Another factor which affects the LAAV is the removal of the test sample after 200 revolutions. In the course of the test, fines created may coat the coarser aggregate particles and form an increasing body of material in which coarser material is embedded. This could provide a protective armouring which reduces the rate of further abrasion and cataclasis of the remaining coarser material. When the test is halted after only 200 revolutions and the sample removed from the machine and sieved, the larger aggregate particles will lose any coatings and the fines will be separated from the bulk of the sample. For the next phase of the test cushioning is considerably reduced, allowing more wear to occur. The effect of these interruptions on the LAAV results is approximately 1 to 2 percent, even when all the fractions of the sample were thoroughly mixed together prior to the second phase of the test.

4 3- **Influence of Geological Factors on the Los Angeles Abrasion Value**

4 3 1- Introduction:

Aggregates required for road metal or concrete must possess a reasonably high degree of strength, durability, tenacity, and stability. They must be able to withstand static and dynamic load stresses and the abrasion experienced in concrete production and handling, road laying and, ultimately, in service. The nature of attrition and degradation mechanisms in service are generally of importance in the selection of material. Resistance to impact and abrasion are measures of this and therefore important in the evaluation and selection of material. These required characteristics are in turn dependent on the petrology and texture of the rock, particularly mineralogy, grain size, fabric, and the amount of secondary minerals and cracks. These features together with the shape pattern of aggregate particles have a significant influence on the engineering properties of the rock aggregate. It is relevant, therefore, to evaluate the influence of these features on the LAAV to assess it's value as an index property. To this end a suite of igneous rocks widely used as aggregate in

Scotland were investigated. These included micropophyritic basalt, amygdaloidal basalt, quartz dolerite, dacite, granite, and aplite.

4 3 2- Aggregate Shape:

The influence of aggregate shape on the LAAV was first mentioned by (Woolf et al 1935), who noted that the presence angular pieces in the sample would cause an increase through loss by abrasion. Hanks (1962) showed that an increase the percentage of flakes within a sample of rock aggregates also causes an increase in the Los Angeles value (Table 4. 4)

Table 4. 4 The influence of flakiness index on LAAV (after Hanks 1962)

Flakiness index	0	10	25	50	75
LAAV	20.8	21.1	20.5	21.9	23.4

In the present study, the effect of aggregate shape was evaluated by artificially varying the flakiness index (I_F) between 0 and 100 for all the rock types tested. This was done by separating flakes from non-flakes and then recombining them in the desired proportions. The results obtained (appendix V), show a systematic relationship in which increase in flakiness index is matched by a significant increase of the LAAV. For example quartz dolerite from Stirling has a LAAV = 12 for $I_F = 0$ increasing to 18 for $I_F = 100$, an increase of 50%. Similarly, the microporphyritic olivine basalt from Orrock has LAAVs of 10 and 18 when $I_F = 0$ and 100 respectively. This relationship between LAAV and flakiness index is summarised in a synoptic plot from all the aggregates tested (Fig 4. 2). The relationship is linear and positive, although lines for individual rock types have different gradients and intercepts with the ordinate axis. This suggest that other factors of a physical-mechanical nature are also operating. The linear relationship between LAAV and I_F is similar to that established by Ramsay(1965) for AIV, Dhir et al (1971) and for ACV and Kazi et al (1982) for LAAV, i.e,

$$LAAV = C + n\,I_F$$

Where C is a constant depending on the nature of the source rock

n coefficient of flakiness

I_F is the flakiness index

In this investigation, coefficient of flakiness varies between 0.075 and 0,049. For a given material however, if the mean slope is 0.062, the difference in the LAAV between the cuboidal sample and the completely flaky sample is given by the equation LAAV = C + 0.062 I_F. Therefore 6.2 percent of the LAAV will be the result of flakiness as it varies between 0 and 100.

The relationship between LAAV and aggregate shape is quite rational, and can be appreciated if consideration is given to the mechanical properties of flaky material. Thin slivers of rock, even strong rock, have low tensile strength in the direction parallel to their shortest dimension, i.e. flaky particles can easily be broken between one's fingers (Ramsay et al 1974). When distributed throughout the test sample, these weaker flaky particles break easily in situations of point loading with the more massive cuboidal particles or the steel balls. Furthermore, an increase in the volume of flakes within the sample increases the surface area of the granular material exposed to abrasive wear.

This increase in the Los Angeles Abrasion Value due to the preferential elimination of the flaky particles can be clearly seen when the original flakiness index is plotted against the flakiness of the sample after testing, i.e. the residual flakiness index (I_{Fr}) (Fig 4. 3). This demonstrates how the flaky particles in the samples are being preferentially eliminated during the Los Angeles Test, and how their rate of elimination decreases as their percentage in the sample increases. When the I_F is more than 60 high percentage of fine material will be produced at early stage in the test, and reduces granulation by cushioning. This behaviour is similar to that recorded by Ramsay et al (1974), Spence et al (1977) from a study of factors affecting Aggregate Impact Value.

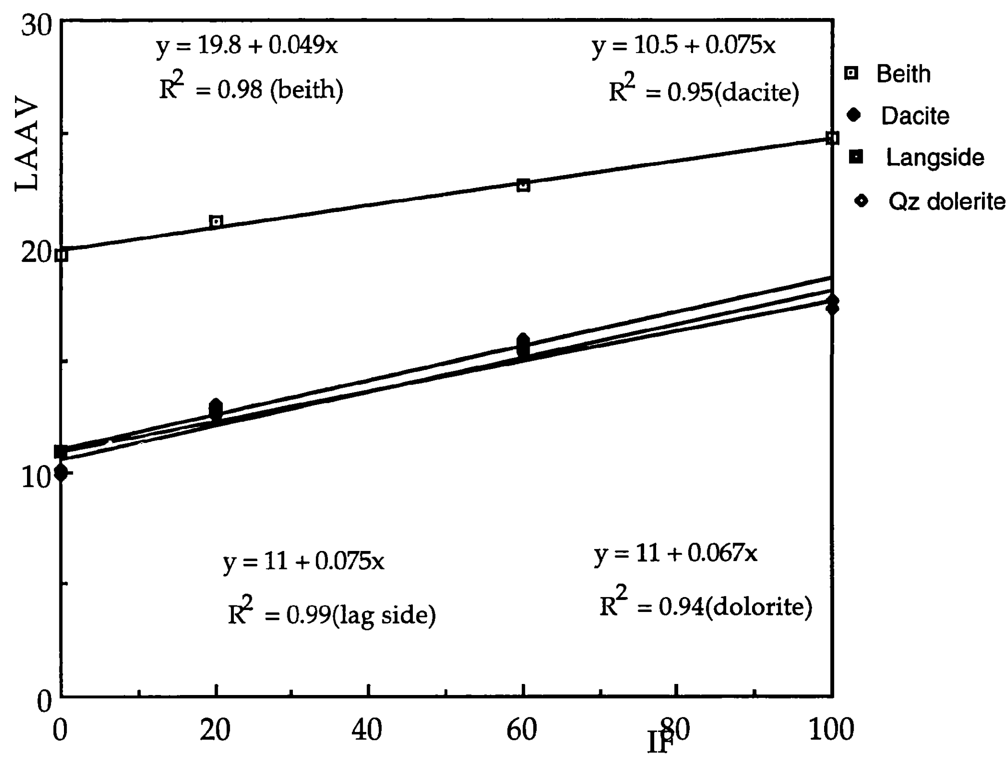


Fig 4. 2 The relationship between LAAV and IF
For different rock types

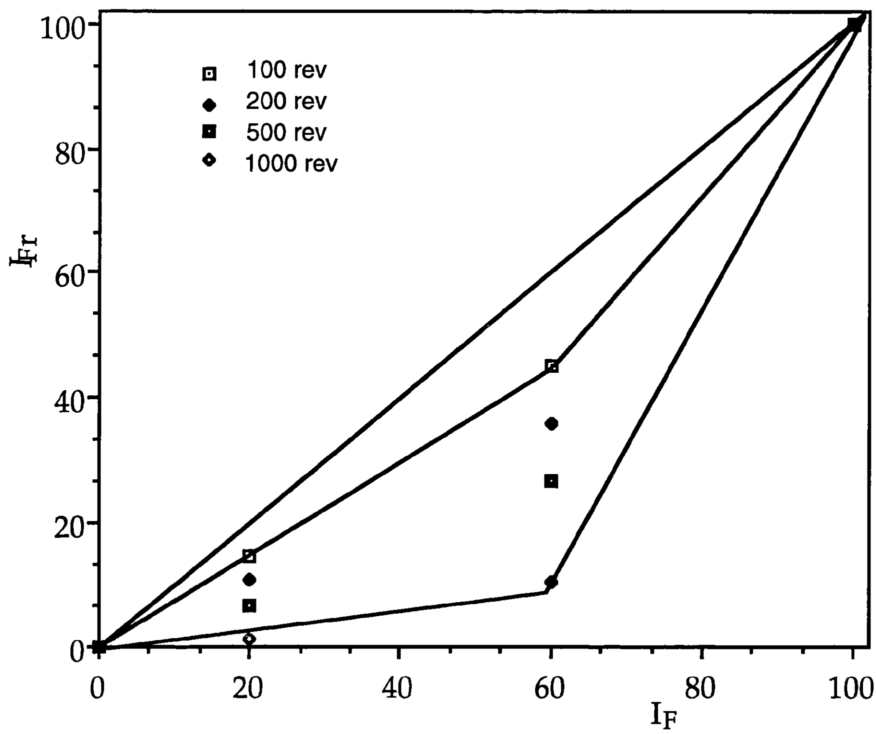


Fig 4. 3 Graph showing the preferential elimination of flaky particles in the course of the LAV test

4 3 3- **Influence of petrology and petrography**:

To study the influence of petrology and petrography on LAAV one should eliminate the effect of other parameters such as particle shape and grading (Hanks. 1962; and Mininger. 1978). Working within grade, B, and setting the flakiness index at zero, the influence of the particle shape and size is removed, and the LAAV obtained reflects the intrinsic strength or resistance to degradation of the rock material. On the basis of this shape constant a better comparison of strength can be made between different rocks. The lower the intercept C, the tougher is the material. The significant petrological and petrographical features which were found to affect aggregate strength were grain size, porosity, microfracturing, fabric, texture, mineral hardness, and alteration.

a- Texture and Fabric

In a study of the engineering properties of aggregates from Finland using the Los Angeles Abrasion Value as a gauge of abrasion Kauranne (1970) found an improvement in aggregate quality as mica content increases. He mentioned that the behaviour of this material is quite contrary to what might be anticipated from it's in-service performance, since it has been reported that an increase in the mica content causes heavy rate of degradation (Hyyppä 1966). During the granulation and shattering of the aggregate particles in the Los Angeles machine, however, mica, with it's weak bonding forces between atomic layers, are a medium in which minimum energy is required to propagate a crack. This implies that an increase in mica content will tend to increase the number of weakness planes within the rock, and subsequently lower it's durability. Having this in mind one should consider the flexible nature of micas which enable them to bend and absorb impact forces and remain as thin flakes with dimensions usually greater than the sieve aperture (1.7 mm). Hence an increase in the mica content of the material will cause a decrease in the fraction passing the 1.7 mm sieve and as a result a low Los Angeles Abrasion Value. Igneous rocks are considered to be isotropic in respect to strength, however, and this factor is not relevant to this to study and will not be dealt with in this study. The texture of the rock however, could have a big influence on the strength of the rock. Coarse grained quartz dolerite for instance, on account of it's ophitic texture displays a lower LAAV than granite of similar grain size but of granular texture. For similar grain size a quartz dolerite has a LAAV of 18 while granite has a value of 26.

b- Grain Size:

The influence of grain size on the strength of intact rock, particularly massive, well bounded granular types has been extensively studied by many authors, e.g. Price (1966), Jaeger and Cook (1969), Hawks and Mellor (1970), Farmer (1983), Simmon and Richter (1976), with a common conclusion that as the grain size increases the ultimate strength of rock material systematically decreases. The theoretical basis for all the explanations given was Griffith crack theory. For crushed rock aggregate Ramsay et al (1974), Spence et al (1977) and Goswami (1984) also demonstrated that grain size of the rock is a significant parameter affecting strength in the AIV, ACV and LAAV tests respectively. Indeed they demonstrated that for fresh silicate rocks it was the dominant geological influence in the broad spectrum of igneous rocks investigated. Although the influence is less marked than in intact rock, the pattern is the same i.e. an increase in grain size correlates with a decrease in aggregate strength.

In order to explore the effect of grain size in the LAAV test, rocks with the same mineral composition but different grain size were compared, i.e. fine grained basalt with a grain size of (0.1 - 0.5 mm) and quartz dolerite (0.4 - 2.7 mm), and aplite (0.33mm) with granite (0.6 - 2.4 mm). Taken as a whole the grain size for the material tested ranges from (0.1 - 3 mm). These rocks were fresh, homogeneous in texture and lacking in any planar fabrics so that the only significant variable was grain size. In this case, any change in the petrographic constant "C" would be directly caused by a change in the grain size. The values of "C" in the basalt is 9.5 while that in the quartz dolerite is 11.4. In the case of acidic material, it was 9 for aplite and more than 25 for granite. Dacite a fine grain intermediate rock yielded a value of 9.7 (Table 4. 4). A synoptic plot of LAAV against the grain size for basic igneous rocks Fig (4. 4) shows that the LAAV increases linearly as the grain size increases, with a high degree of correlation. Within the quartz dolerite marked variation in the grain size occurred between the chilled margin and the central part of the sill. Samples were collected and laboratory aggregate prepared. Two from the chilled margin, with a grain size of 0.75 mm, yielded LAAV = 10, while samples from the central part of the sill with a grain size up to 2.7 mm had a LAAV of 18. The same type of variation was exhibited by granite aggregate from Dalbeattie quarry where aggregate with grain sizes of 1.5 mm and 0.63 mm gave a LAAV of 31 and 24 respectively. Aplite from minor intrusions with grain size of 0.33 mm has a LAAV of 11. Within one or between different rock types LAAV - grain size relationship is characterised by high coefficients of correlation.

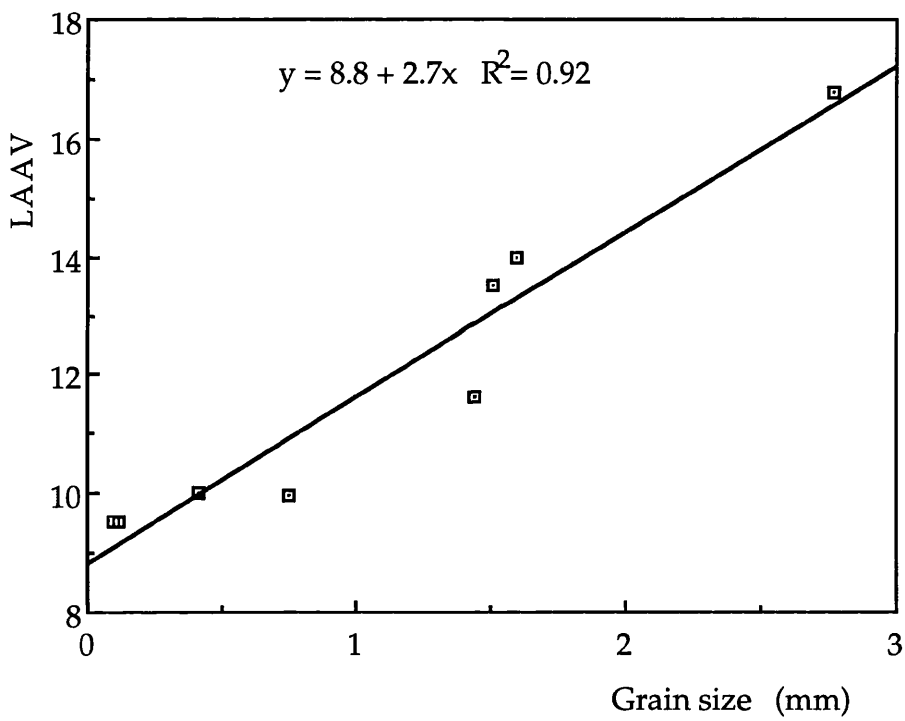


Fig 4. 4 The relationship between LAAV and grain size for dolerite and microporphyritic basalt

which confirms that influence of grain size influence on the LAAV is rational and not random.

Table. 4. 5 Showing some of the obtained results

Sample	LAAV	LAAVR	AIV	PLS(MPa)	Grain size(mm)
Sz2	13	54	10.5	7.8	-
St3	112	49	8	10.2	1.44
St4	17	34	12	8.4	2.77
St6	13.5	57	9.5	11.2	1.51
St11	10	57	6	-	0.41
St19	12.5	50	7	12	-
St24	14	43	11	8.4	1.6
St25	17	37	12.5	8.50	-
St26up	10	58	7	9.5	0.75
St26l	19	50	12	7.05	2.4
St30	38	20	-	1.2	-
St31	38	20	20	1.5	-
St32	39	18.5	23	1.3	-
Stw	29	32	17	-	-
G1	25.5	31	18	12.3	2.74
G5	45	18	-	8	2.39
G6	61.5	0.2	57(MAIV)	1.8	-
G7	25	33	18	9.2	1.22
G9	31	29	21	10.6	1.18
G10	24	36	17	13	0.63
Nairn	26.5	28	14	-	-
Aplite	11	53	9	-	0.41
Langside	9.5	56	6.5	10 - 12	0.1 - 0.2
Orrock	9.5	60	6.5	9 - 12	0.1 -0.2
Dacite	10	54	8.5	7 - 10	-

Sz, St = quartz dolerite

G, Nairn = Granite

Langside, Orrock = microporphyritic basalt c- Mineralogy:

Despite the difference in mineralogy of the various rock types tested, their Los Angeles Abrasion Values seem to be indistinguishable. At $I_F = 0$, values range from 11 for aplite, 9.5 - 10 for dacite and 9 - 10 for basalt. Dolerite being slightly coarser grained has an LAAV in the range of 10 - 18. The results indicate that the response of these materials to attrition and wear in the Los Angeles Abrasion Test is independent of their mineralogy. This is compatible with the finding of Ramsay et al (1974) and Spence et al (1977) who concluded that in the AIV and ACV tests, the different mineral assemblages in the range from acid to basic igneous rock exhibit similar mechanical characteristics.

d- Weathering and/or Hydrothermal Alteration:

It has been demonstrated in chapter 3 that mineral alteration and associated microcracking are factors which significantly affect the strength and durability of intact rock. It was felt that this influence should apply to crushed rock aggregate but to what extent?. To measure it's effect a LAAV programme was mounted on several rock types which displayed considerable variation in the degree of weathering.

Weathering was measured by several index properties such as micropetrographic index (Ip), water absorption, porosity, and density. The results of these are presented in the form of graphs. Typical data come from the quartz dolerite and granite. In the fresh condition the relative density and Ip of quartz dolerite are high, while water absorption is low (Table 4. 5). As weathering increases by mineral alteration and the development and opening of microcracks, Ip decreased to 0.9 - 1.5, density fell to 2.45 - 2.49 , and water absorption increases to 3 - 4. LAAV increases from 9 - 12 in fresh condition to 37

- 40 in the most weathered state. The same behaviour is exhibited by granitic and basaltic aggregates. Weathered amygdaloidal basalt from Beith with high porosity and water absorption and low density have a LAAV in the range of 18 - 20 , which is approximately double the values obtained from the dense, low porosity and low water absorption, fresh microporphyritic basalt of Orrock and Langside, i.e. 9 - 11. This wide variation in the Los Angeles Abrasion Values between fresh and weathered material regardless of their petrology shows that as the rock material weakens the resistance to abrasion and impact of its aggregates falls sharply, resulting in a higher LAAV. Figure (4. 6a,b) shows that as the petrographic index (Ip) of granite and quartz dolerite increase LAAV decreases in an exponential manner, while the other indices, secondary constituents, water absorption and porosity exhibit a linear relationship (Fig 4. 6, 7).

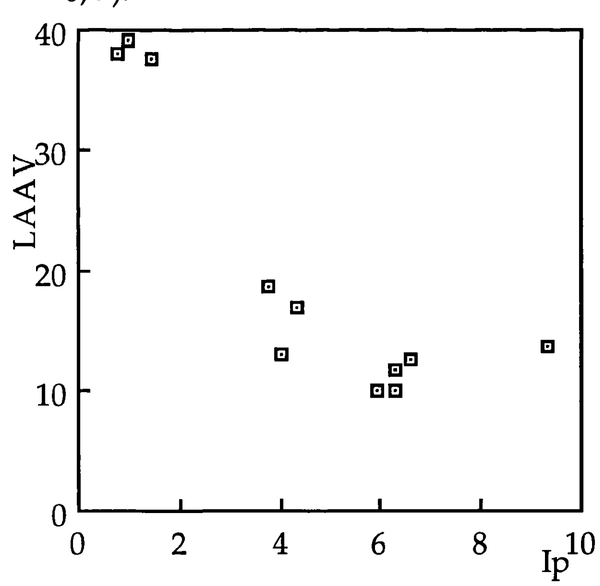


Fig 4.5a LAAV - Ip relationship for quartz dolerite

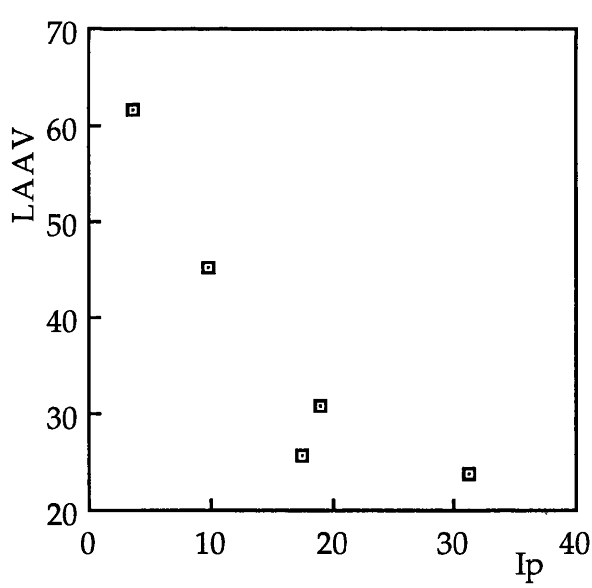


Fig 4.5b LAAV - Ip relationship for t
Granite

These relationships can be explained by the fact that in weathering, the secondary minerals which develop are generally softer, i.e. less resistant to abrasion, mechanically weak and more porous than the minerals they replace.

In addition expansion of the secondary minerals creates an internal pressure which causes fractures to propagate through the adjacent mineral grains. This process along with the fracturing induced by physical weathering agencies results in a higher porosity, higher water absorption, and lower density.

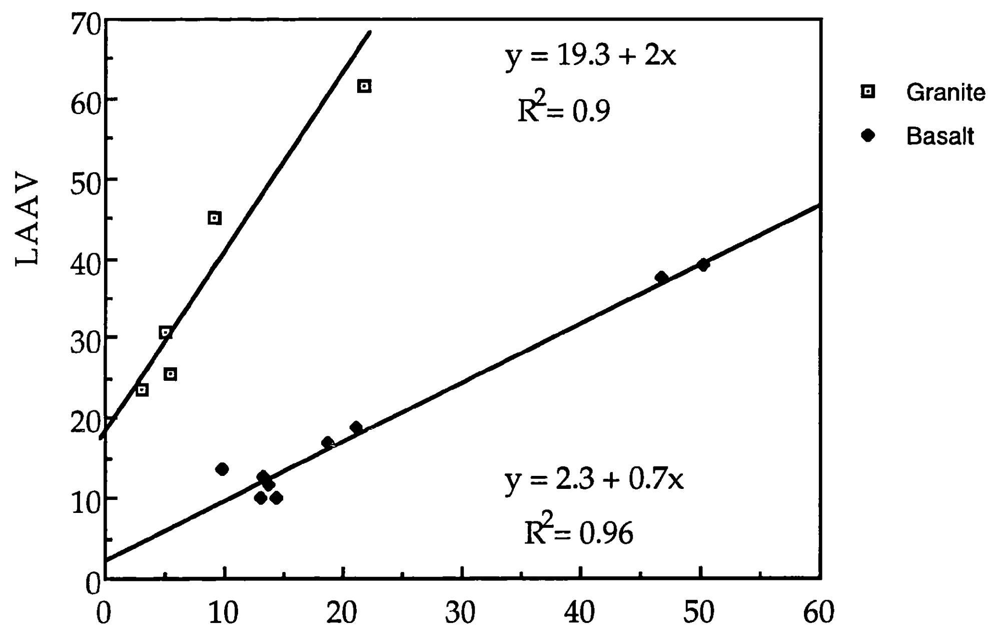


Fig 4. 6 The influence secondary minerals and crack content onthe LAAV for granite and quartz dolerite

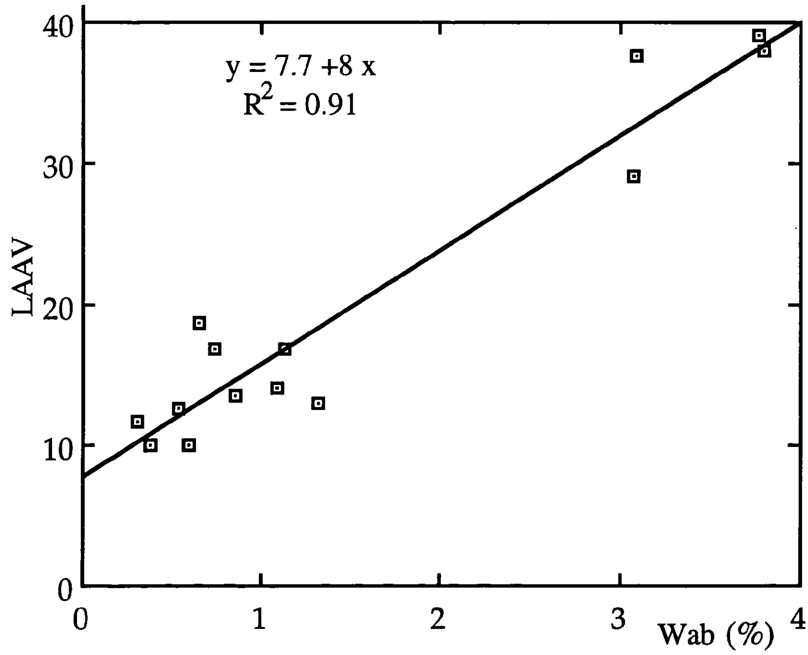


Fig 4. 7 The relationship between LAAV and water absorption for quartz dolerite

4 4- **The Los Angeles Abrasion Residue value:**

In the course of the Los Angeles AbrasionTest, material breaks down to a graded assemblage ranging in size and shape from fine to coarse. The choice of the fraction finer than ASTM No 12 (1.7 mm) sieve size as a measure of the resistance to impact and abrasion is an arbitrary one and tells little of the overall behaviour of the aggregate. Ramsay (1965) and Spence et al (1977) observed that any good aggregate should exhibit a minimum degradation to material of any size, and introduced the so called Aggregate Impact Value Residue (AIVR). This is the proportion of particles remaining within the size range of the untested sample.

In the present study the results obtained show that a flaky aggregate from one sample could be compared and preferred to another with lower flakiness if the aggregate resistance to wear is represented only by the LAAV. For instance a sample of the microporphyritic basalt aggregate with IF of 100 has an LAAV = 20 numerically superior to amygdaloidal basalt and coarse grained granite samples with IF = 0 but LAAV of 21 and 26 respectively. However an examination of the residue fraction which has resisted the impact and abrasion forces reveals that just 26 percent of the microporphyritic basalt sample remains unbroken, much lower than the fraction retained in the original size in both the amygdaloidal basalt and granite, i.e. 40 and 29 respectively. As far as the in-service performance of the material is concerned, the standard index value alone provides a less sensitive evaluation of the quality of the stone. However, following Ramsay (1965), Ramsay et al (1974) and Dhir et al (1971) and in the light of the latter results a more realistic measure of the material performance would be to include a record of the proportion of the fraction remaining within the original size range (20 - 10 mm) in addition to the LAAV. This non-standard value is termed the Los Angeles Abrasion Residue Value (LAAVR).

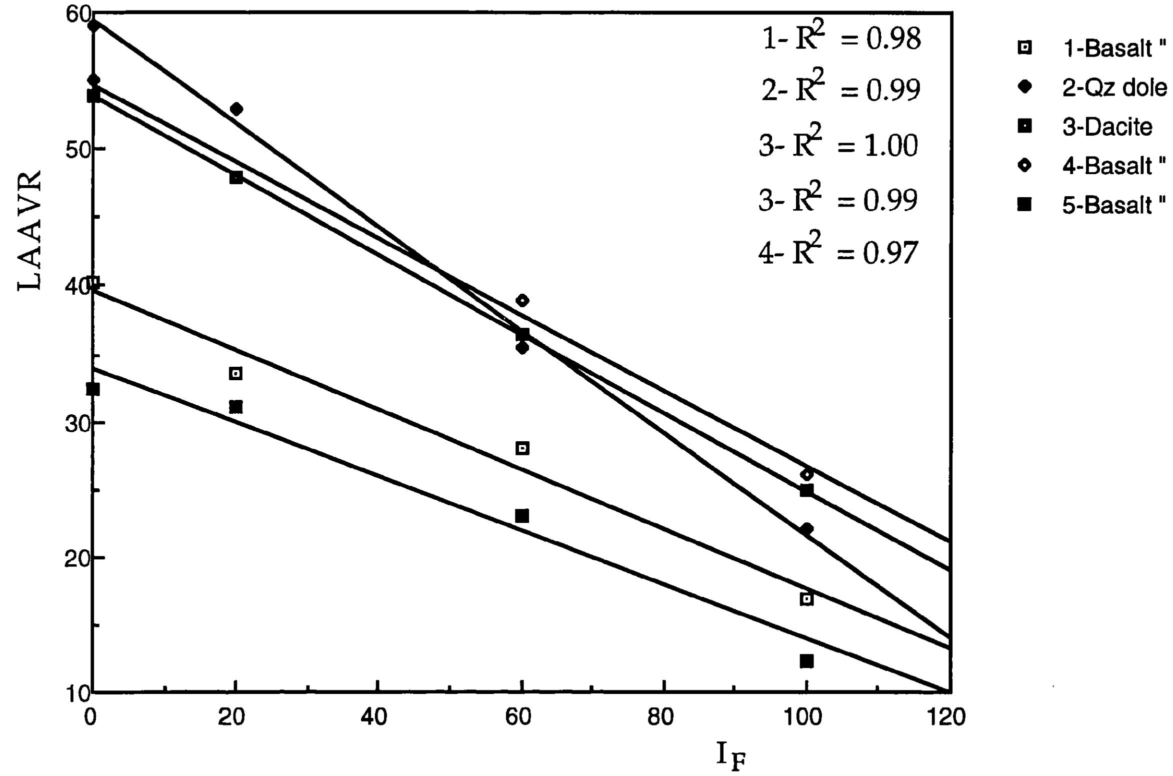


Fig 4. 8 The relationship between LAAVR and
Flakiness Index for the studied rocks

The use of this non standard index value seems to have strong petrological basis. As seen earlier, the misleading results caused by the presence of micas in the Finish study of foliated rocks would have been understood IF the LAAVR was considered.

Like the Impact and Crushing Residue Values, the variation of the Los Angeles abrasionValue Residue are rational with respect to geological and physical parameters. The plot of LAAVR against I_F (Fig 4. 8), regardless of rock type, displays a negative linear relationship of the type (LAAV = C - nI_F), with a high coefficient of correlation. The absolute value of the regression coefficient (n) for LAAVR - I_F is in the order of 8 to 10 times the regression coefficient LAAV- I_F. This indicates that the LAAVR is more sensitive to the variation of I_F.

The LAAVR display a rational response to several physical indices such as I_F, grain size Fig (4. 9), and weathering indices such as porosity Fig (4. 12), water absorption , specific gravity, and the micropetrographic index (Fig 4. 10, 11).

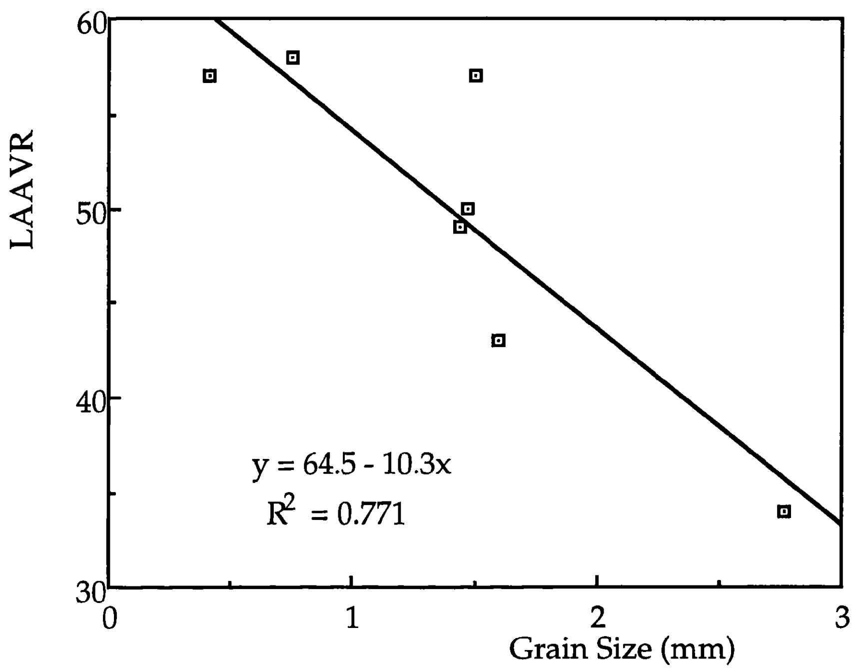


Fig 4. 9 The relationship between LAAVR and grain size in quartz dolerite

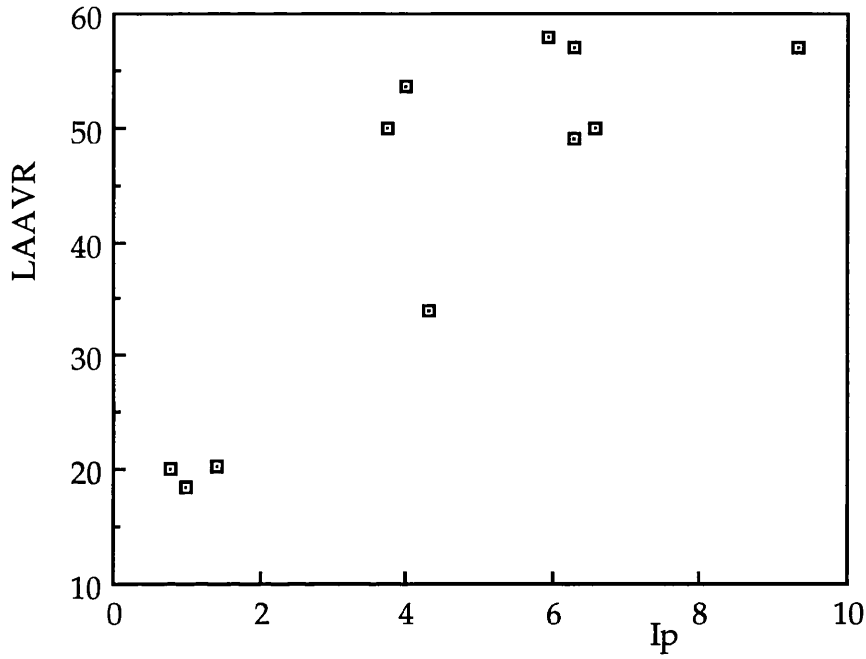


Fig 4. 10 The relationship between LAAVR and the micropetrographic index for quartz dolerite

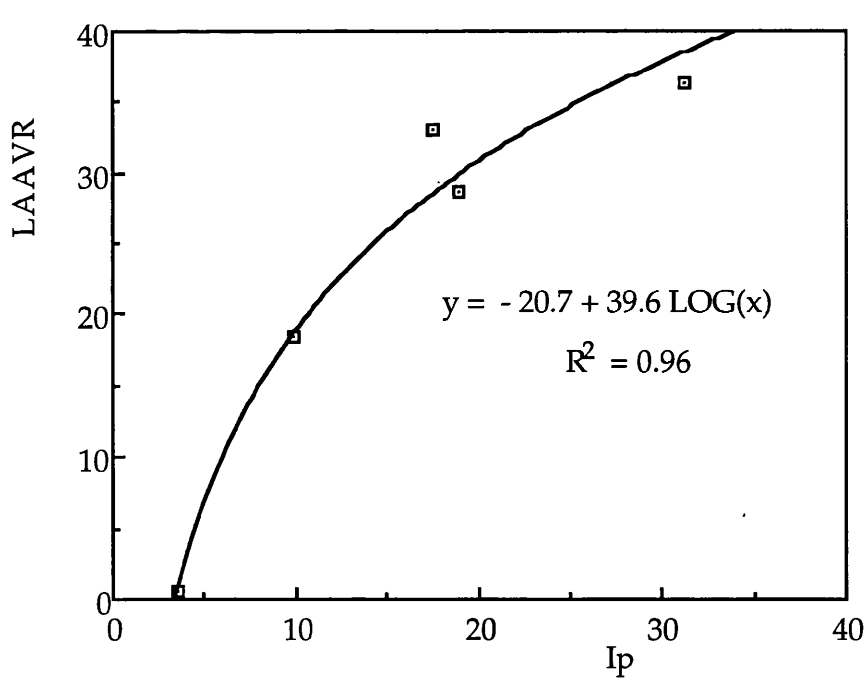


Fig 4. 11 The relationship between LAAVR and the micropetrographic index for granite

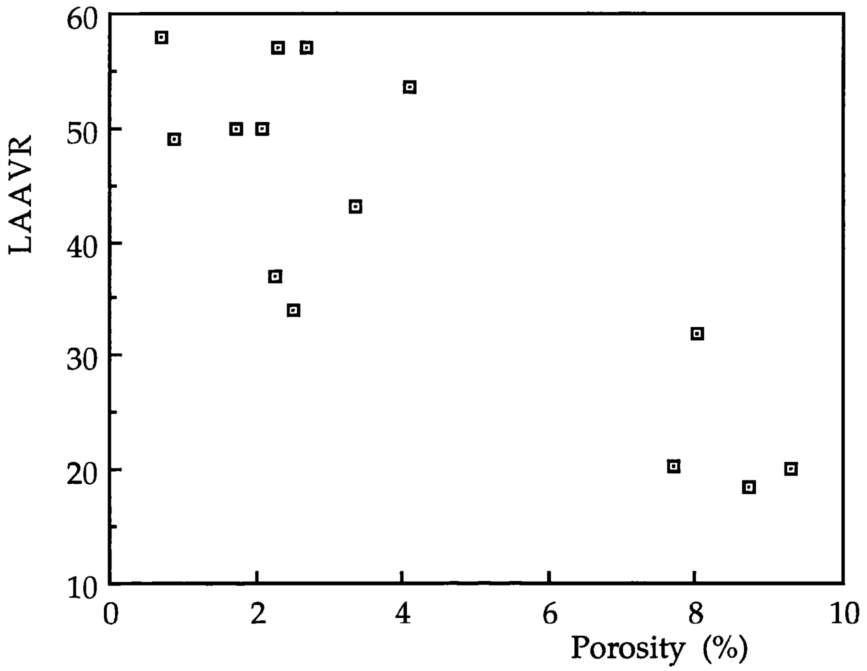


Fig 4. 12 The relationship between LAAVR and porosity for quartz dolerite

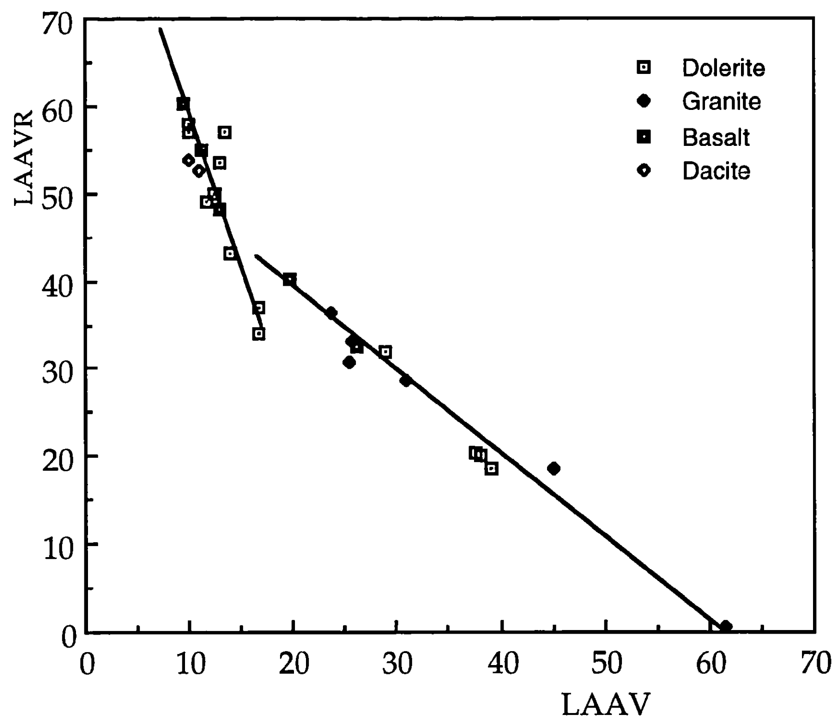


Fig 4. 13 The relationship between LAAV and LAAVR for different rock types at different weathering states

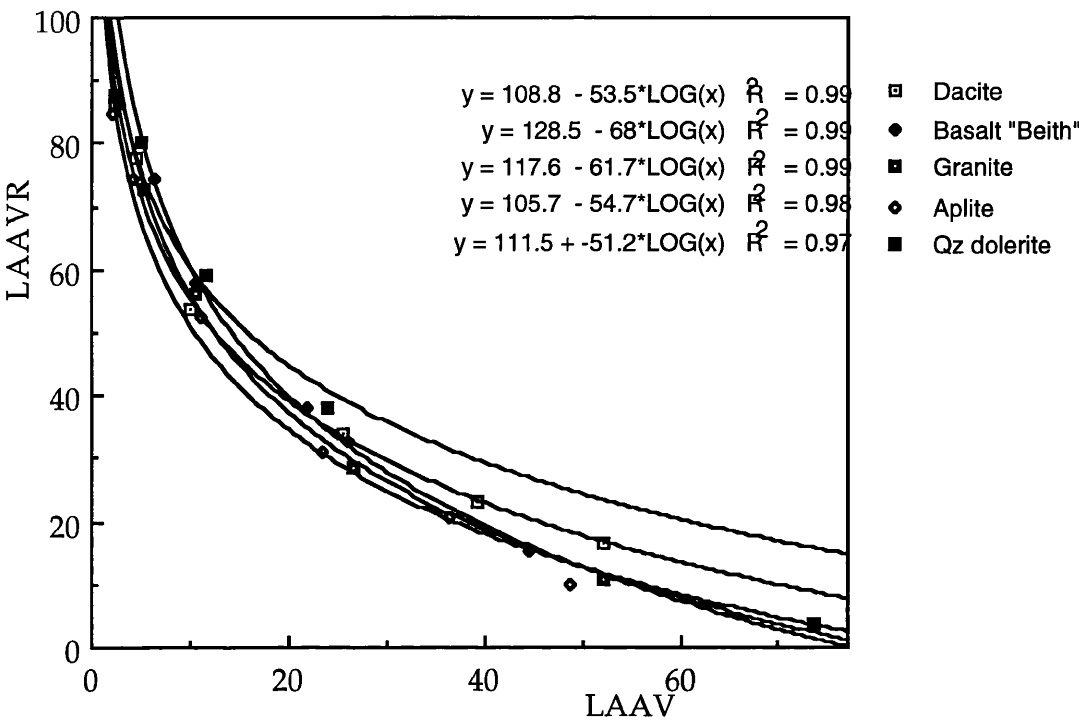


Fig 4. 14 The inter-relationship between LAAV and LAAVR during the Los Angeles Test when the Rev Nber is increased from 100 to 2000

4 5- The Communition of Aggregate in the Los Angeles Machine:

4 5 1- Introduction

Despite the international use of the Los Angeles Abrasion Test as an index of aggregate strength or resistance to abrasion, the processes and mechanisms of communition involved in the test have been little studied and are in consequence not yet fully understood. Meininger (1978), Kazi et al (1982), and Tourenq and Denis (1982) mentioned that both impact and abrasion are involved in the degradation of the material during the test. The effects of impact are believed to be greater than abrasion but the contribution of each to the final Los Angeles Value is still not really known. Therefore one object of the present study focussed on these mechanisms.

4 5 2- Communition Process:

To investigate the degradation of the aggregates in the Los Angeles machine, the number of revolutions of the drum was systematically increased from 100, 200, 500, 1000, 1500 up to 2000 revolutions, and the fractions LAAV, LAAVR and M (the fraction passing the 10 and retained on 1.7 mm sieve) were calculated and recorded at every stage. The effect of shape and size of the aggregate particles on the test results and it's repeatability were minimised by using only one grading, i.e. grade B and all the flaky particles removed.

Following previous procedures, the Los Angeles Abrasion Value is plotted against the number of revolutions Fig (4. 15). For all the rock types tested this relationship is linear, with high coefficients of correlation (0.9 - 0.99) and constant gradient up to 2000 revolutions. Despite the similarity in the pattern of fines production, the fraction passing ASTM 1.7 mm sieve, fines production rate, is quite different and depends on the intrinsic characteristics of the material, as a result of which each material has it's own distinctive curve on the LAAV vs Revolution number plot. Ekse and Morris (1959) showed that after 1 hour, corresponding roughly to 2000 revolutions, the LAAV vs Rev Nber curve started to become non linear. In our opinion this behaviour results from the fact that, after so many revolution, there is no longer enough material to sustain the early fines production rate.

The Los Angeles Residue value however exhibits a different evolutionary pattern from that of the standard LAAV as the revolutions number increases. The difference is that the LAAV increases linearly with the revolutions number, while the LAAVr does not (Fig 4.16). In fact for the first few hundred revolutions, i.e. up to 500 Rev, the original material shows a high degradation rate, and the LAAVR - Rev Nber relationship is quite linear. As the revolutions exceeds 500 a change in the pattern of degradation occurs with a marked fall in the refining of the original material. The LAAVR - Rev Nber relationship becomes non linear and can be best described using a logarithmic function of the type LAAVR = a - b Log(Rev Nber). The decrease in the degradation rate of the original material observed at and beyond 500 revolutions while the fines production increases constantly, indicate that at this stage the fraction passing 1.7 mm is developing at the expense of the M fraction, a further pulverisation of the already broken material of the M fraction.

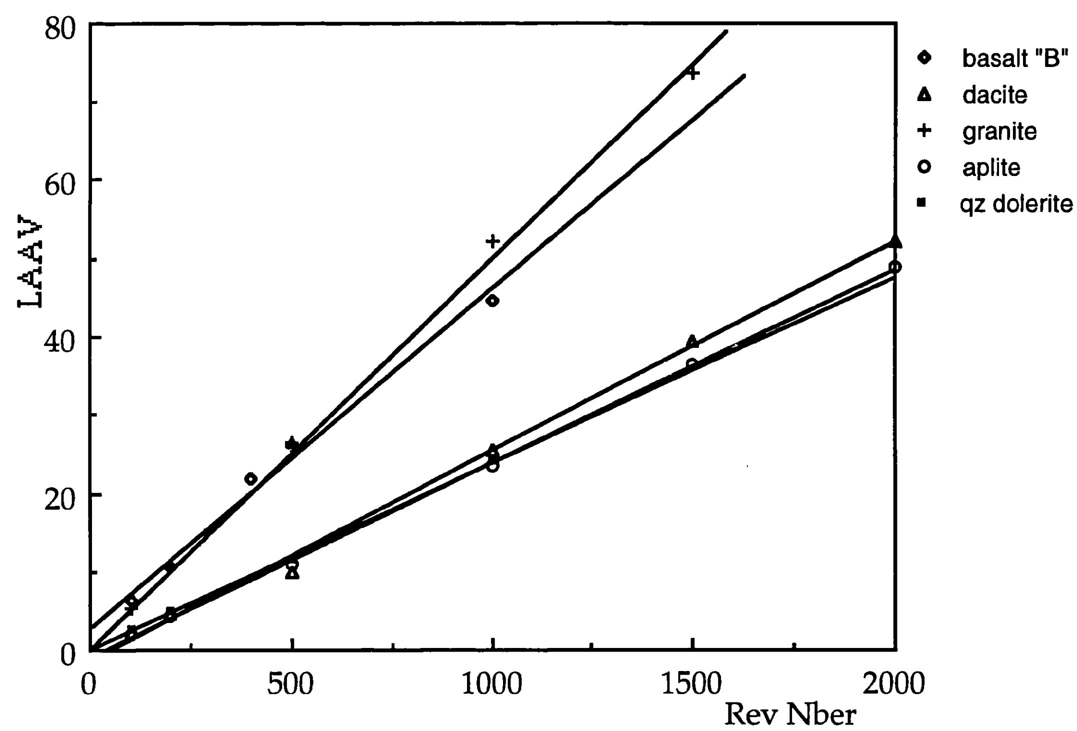


Fig 4. 15. The relationship between LAAV and the revolution as it increased from 100 to 2000

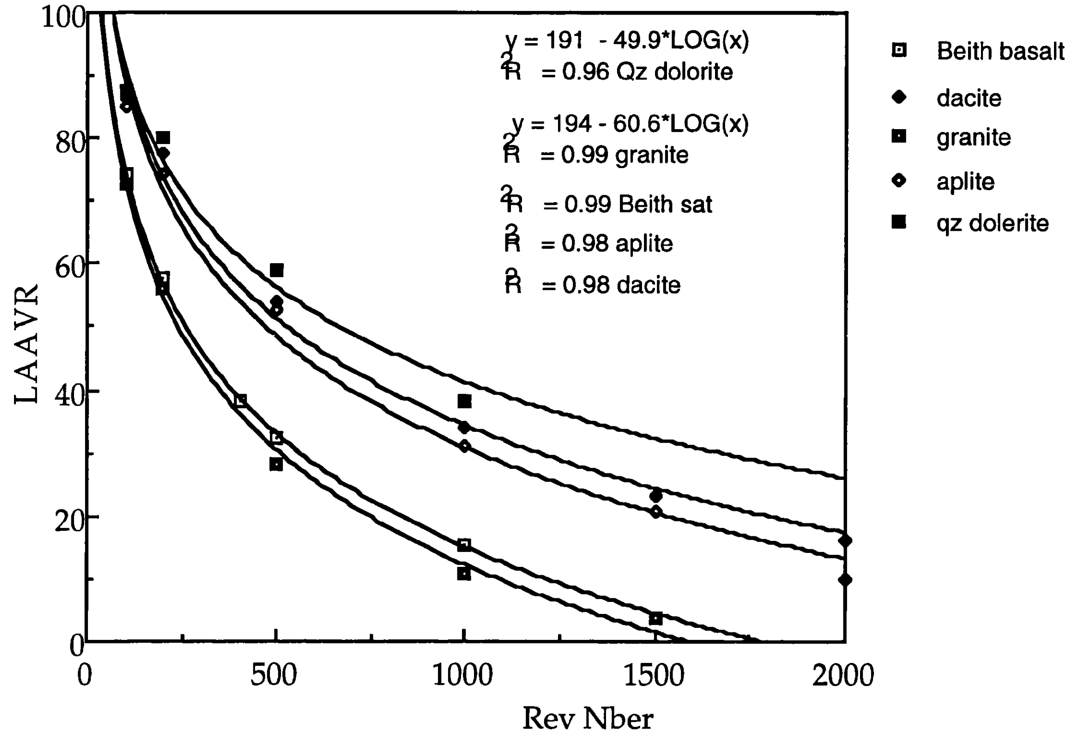


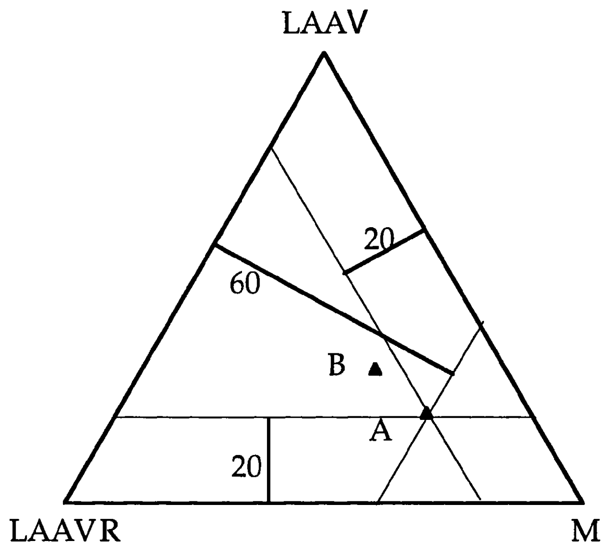
Fig 4. 16 The evolution of the LAAVR with the increase in the revolution number

For a better view of the evolutionary pattern of the cataclasis process with increase in the number of revolutions, values of LAAV, LAAVR, and M were plotted on a triangular diagram. This demonstrate the interrelationship between the three fractions during the granulation process more clearly. The representation of the three values on the triangular diagram requires that the sum of the three fractions LAAV, LAAVR, and M must be equal to unity (100 %). Each apex is a full unit (100 %)for one component and zero for the other two. i.e.

on LAAV apex:	LAAV = 100,	LAAVR = 0,	and	M = 0
on LAAVr apex:	LAAV = 0,	LAAVR = 100,	and	M = 0
on M apex:	LAAV = 0,	LAAVR = 0,	and	M = 100

Each fraction decreases progressively between it's apex the opposite sides of the triangle where it is zero. Fig (4. 16) gives an illustration of how to represent the two points A and B.

A	LAAV = 20,	LAAVR = 20, and	M = 60
B	LAAV = 30,	LAAVR = 25, and	M = 45



Triangular Plot

Fig 4. 17

Any variation in the percentage of the three fractions is displayed by a change in the position of the point within the area of the triangle.

The plotting of the data for a single material when the revolutions number increases from 100 to 2000 exhibits a systematic change in the position of the points along a regular path (Fig 4. 18, 19). It is clearly seen that for the first 500 revolutions the disposition of the points in the triangular diagram fall on a approxiamately straight line showing a steady decrease in the LAAVR coupled with regular increase in LAAV and a strong increase in M. After the 500 rev a marked deflection of the path of communition towards the LAAV apex occurs. This deflection is caused by the decrease in the granulation rate of the original material exhibited by a slowing down of the LAAVR decrease rate while the fines production rate proceeds on at the expense of the fraction M. Therefore, any further increase in the LAAV of the aggregate , after the 500 rev, is simply a further pulverisation of the already broken material (M fraction).

The degradation mechanism of aggregate material in the Los Angeles machine therefore appears to proceed in such a way that the material initially suffers a heavy break down to smaller fragments which can pass through the original sieve but only small amount can pass the 1.7 mm sieve. After the determined number of revolutions, i.e. 500 revs in the case of grading ,B, the accumulation of the material retained on 1.7 mm (M) appears to reach a volume sufficient enough to provide cushioning to the remaining unbroken original material. Any further communition is at the expense of the already broken material and then the LAAV increases mostly at the expense of M. This behaviour confirms the revolution count in the specification of the standard test for B grading.

Regardless of the intrinsic strength of the material tested all values obtained lie on or very close to the cataclasis curves previously described, and the weaker the material the further beyond the hinge point it falls (Fig 4. 20c). Similarly materials of different weathering states Fig (4. 20d) also show that the more weathered the aggregate the further it's position beyond the hinge point. The process of cataclasis, therefore, is consistent for all materials tested. The effect of the flakiness index on the evolutionary pattern of the cataclasis curve is, however, different. The preferential elimination of flakes seems to affect the form of the curve, i.e. Fig (4. 18, 19) shows that materials having 0, 20, 60, and 100 flakiness index evolve along distinct cataclasis curves, and the more flaky the material the closer it's curve comes to the corner M. This evolution along different cataclasis curves of the materials having different flakiness index is due to the difference of the rate difference in the degradation of the original material between the flaky and non flaky materials.

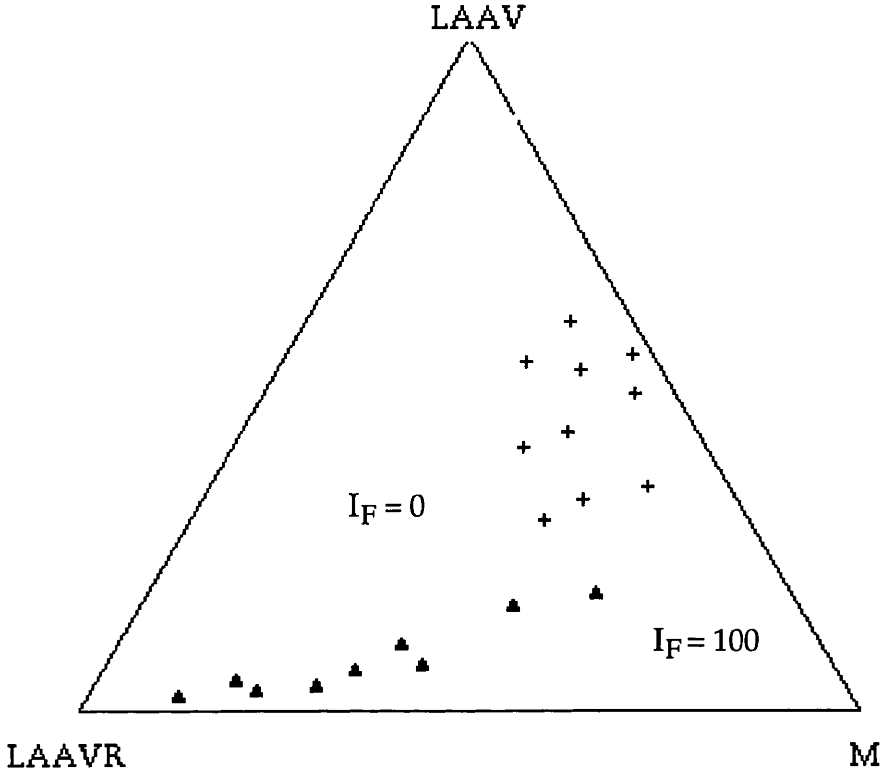


Fig 5. 18 The communition path in the Los Angeles Abrasion Test. ▲ Rev No = or < than 500

+ Rev No = or > than 100

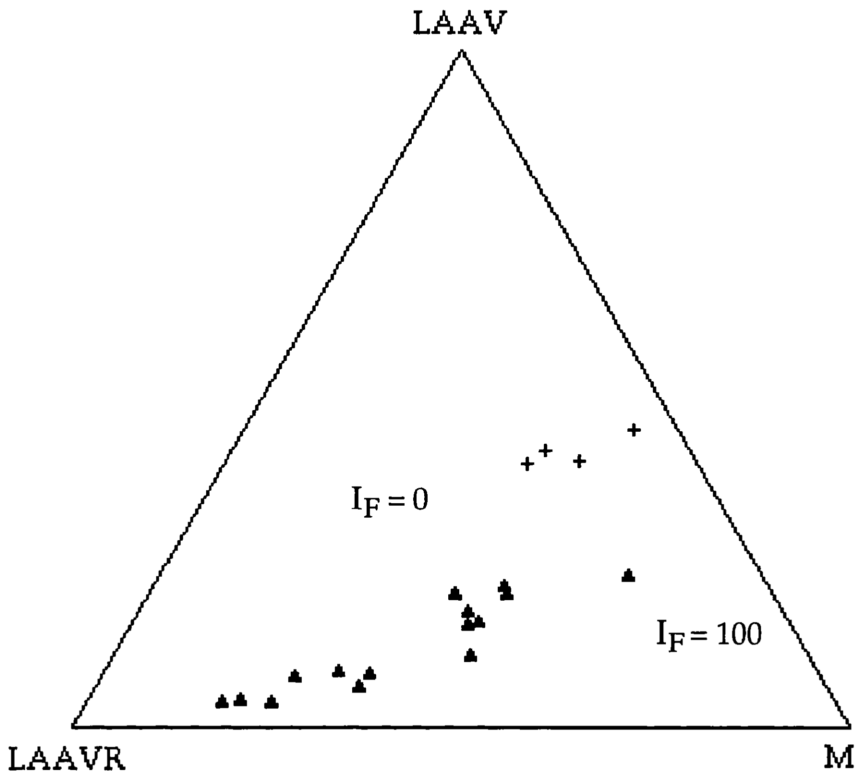
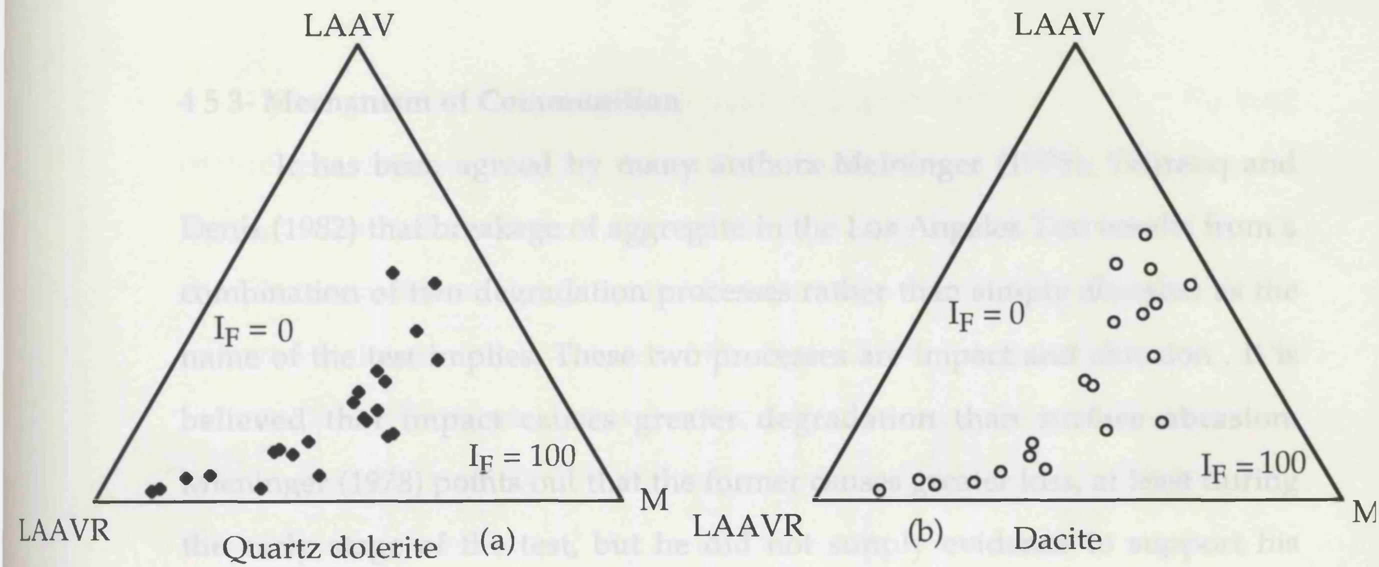
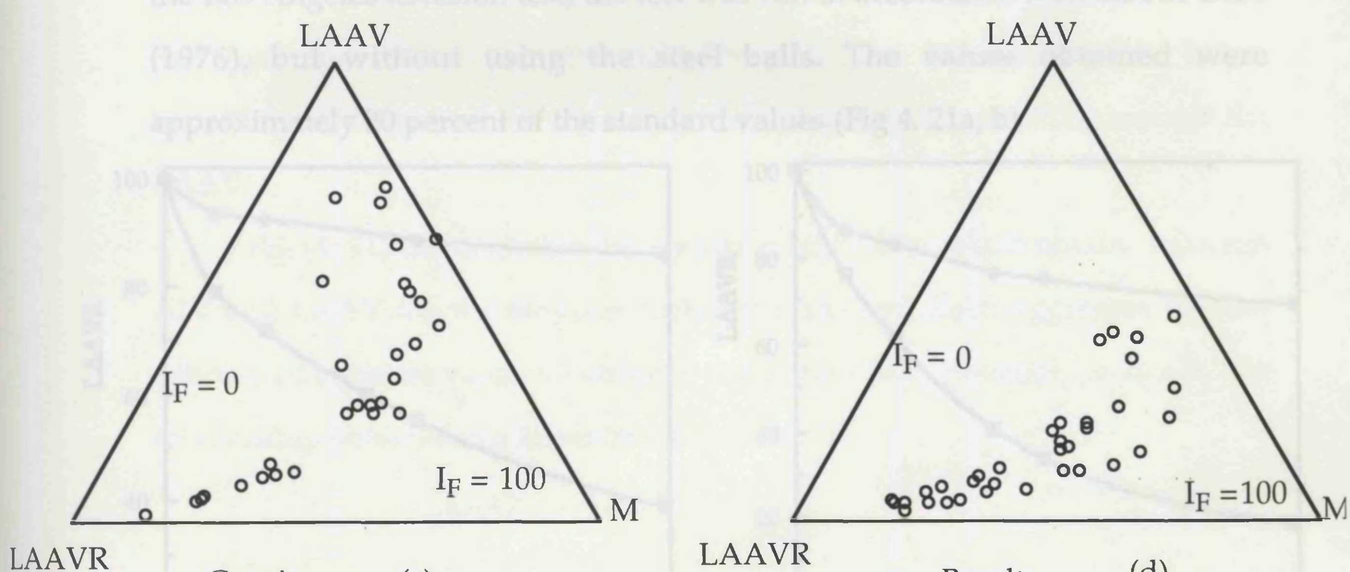


Fig 5. 19 The influence of IF on the Path of communition in the Los Angeles Abrasion Test (basalt)



The communition path of some aggregates
in the Los Angeles Abrasion Test
$I_F = 0 - 100$

Fig 4. 20



Peterhead, Nairn, dunecht and Dalbeattie

Loanhead, orrock, and Langside

4 5 3- **Mechanism of Communition**

It has been agreed by many authors Meininger (1978), Tourenq and Denis (1982) that breakage of aggregate in the Los Angeles Test results from a combination of two degradation processes rather than simply abrasion as the name of the test implies. These two processes are impact and abrasion . It is believed that impact causes greater degradation than surface abrasion. Mieninger (1978) points out that the former causes greater loss, at least during the early stage of the test, but he did not supply evidence to support his contention. The striking similarity between the granulation pattern of the original material in both the Los Angeles and Aggregate Impact Tests (appendix)) suggests that in the course of the Los Angeles test the impact component is dominant.

In order to investigate the contribution of abrasion in the final result of the Los Angeles abrasion test, the test was run in accordance with ASTM C131 (1976), but without using the steel balls. The values obtained were approximately 20 percent of the standard values (Fig 4. 21a, b).

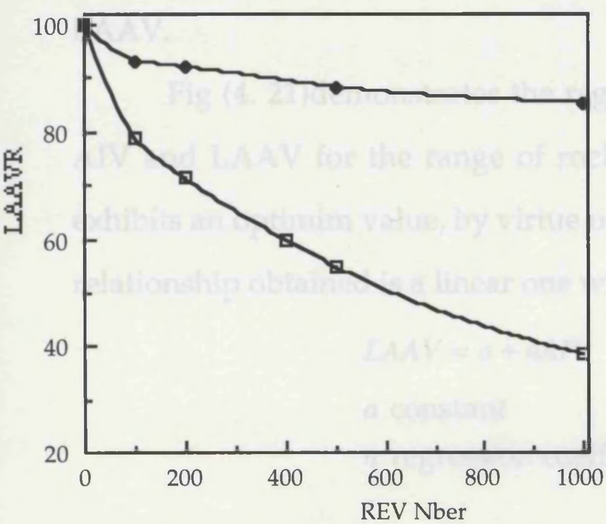


Fig (4. 21a) The pattern and magnitude of Abrasion and Impact in the LAAV (microporpheritic basalt)

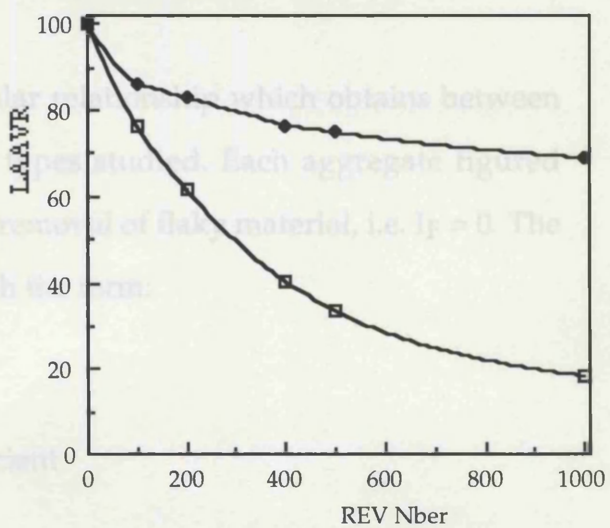


Fig (4. 21b) The pattren and magnitude of Abrasion and Impact in the LAAV (macroporpheritic basalt)

The granulation by impact in the Los Angeles Abrasion Test is the sum of three processes, the impact of the steel balls, the mutual collision between the aggregate particles and the falling of the aggregate from the internal edge on the internal surface of the cylinder. More than 80 percent of the communition is believed to be caused by impact. Mieninger (1978) suggested that the hard material in the test tends to shatter more than softer material which absorbs the impact forces better. On the other hand, the softer material will be more susceptible to abrasion, and the result of the degradation in this case may be more of dust rather than larger angular pieces produced by shattering of the hard pieces.

4 6- **Comparison with AIV:**

As mentioned earlier, impact loading is the more significant agent of the degradation process in the Los Angeles AbrasionTest (LAAT). Because of this and as a possible pointer to aggregate influences the LAAT was compared with the equivalent British test, the AIV. The extensive literature on the latter, including the factors which affect it, are found to affect in a similar manner the LAAV.

Fig (4. 21)demonstrates the regular relationship which obtains between AIV and LAAV for the range of rock types studied. Each aggregate figured exhibits an optimim value, by virtue of removal of flaky material, i.e. $I_F = 0$. The relationship obtained is a linear one with the form:

$$LAAV = a + nAIV$$

a constant

n regression coefficient

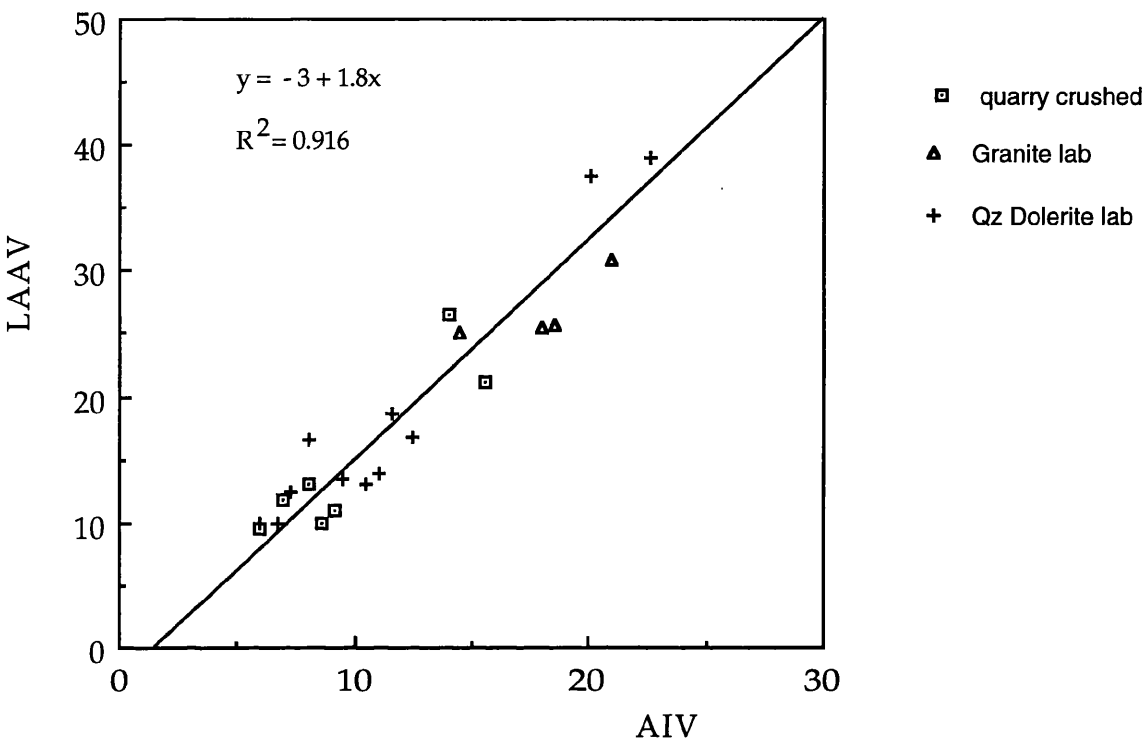


Fig 4. 21. The relationship between LAAV and AIV for the rocks studied

CHAPTER V

Engineering Properties of the Studied Intact Rocks

5 1- Introduction

Rock behaviour and strength characteristics in relation to their physico-geological conditions are the main subject of this chapter. Rock performance in different geotechnical environments is in fact the result of the contribution of several parameters such as grain size, texture, structural defects, mineral hardness, relative density, and weathering, acting individually or in concert. The quality variation of the rocks studied, from basic to acid, plutonic to volcanic, and from fresh to weathered, provided an opportunity to constrain and study individually the influence of the above parameters on rock strength. The tests used to study the engineering properties of the present rock suite are:

1- Uniaxial Compressive Strength

2- Point Load Strength

3- Schmidt Hammer

4- Ultrasonic Velocity

Other derived values such as dynamic Young's Modulus were also used. The results obtained are later correlated with aggregate strength values in order to establish relationships from which aggregate quality can be predicted from intact rock properties.

The Uniaxial Compressive Strength

5 2 1- Introduction

In the field of geomechanics three types of uniaxial test are generally considered for categorising strength and deformation characteristics of rock material. These are the uniaxial compressive strength, tensile and shear strength tests. Farmer (1983) states that there is a limited demand for shear testing of intact rock, and also for tensile test (Hawkes and Mellor 1970), whereas the uniaxial compressive strength test is the one widely used in engineering practice as a rock index property, and in research programmes (Paterson 1978). It has the advantage of being simple and requires minimal sophistication in equipment. The uniaxial compressive strength is recommended and standardised by (ISRM 1978).

Many authors have studied the unconfined compressive strength and presented a range of definitions of the test. Fox (1923) defined it as the ability of the rock to withstand crushing under pressure as in blocks and columns. Krynine and Judd (1955) describe the tests as the load required to break a loaded sample that is unconfined at the sides. It is calculated from the formula:

$$P = F / S$$

Where P is the uniaxial compressive strength

F is load at which the specimen fail

S the surface area under the applied load

Despite, the widespread use of the uniaxial compressive strength test, it is still primarily regarded as an index, especially for classification purposes and not as a design test Deer and Miller (1966), Stapledon (1968) and ISRM suggested methods (1978). Sigvaldason(1964) gave evidence that the results of uniaxial compression on identical specimens made in eight different laboratories displayed wide discrepancies of magnitude and variance, even though all the tests were performed on machines conforming to the norms required by British Standards and ISRM standards.

The mode of failure in uniaxial compressive strength has been the focus of much investigation (Jaeger. 1969, Hawks and Mellor. 1970). Three modes of failure have been identified, first, cataclasis which consists of a general internal crumbling by formation of multiple cracks in the direction of loading. Generally when the specimen collapses two conical end fragment are left together with long slivers of rock from around the periphery. The second mode of failure is axial cleavage exhibited by a vertical splitting in which one or more major cracks split the sample along the loading direction, i.e. in a principal plane of stress .The third mode of failure is shear fracturing of the specimen along a single plane oblique to the principal stress. In practice it is some times difficult to distinguish these three different modes of failure in a failed specimen, and occasionally all three modes may appear to be present.

Ideal brittle materials under uniaxial compressive loading have been shown to behave elastically until sudden failure occurs (Fig 5. 1a,b). The stress-strain curve Fig (5. 2), for a typical rock material, divides into four stages: In stage 1 (OA) the curve is slightly convex upwards due mainly to crack and pore closure (Brace, 1965; Walsh,1965; Walsh and Brace, 1966). In stage 2 (AB) the curve is linear and represents elastic strain in the material. Up to this stage loading and unloading does not produce irreversible changes. In stage 3 (BC) the curve deflects to become concave downwards, due to the formation of microcracks, which progressively destroy the integrity of the original fabric and the load bearing capacity so that irreversible changes are induced in the rock. From B the slope of the curve decreases progressively until it approaches zero at C and the uniaxial compressive strength of the rock is reached. In conventional soft compression machines a violent failure with a complete structural collapse occurs when the point C is reached due to the sudden release of the strain energy stored in the machine. In "stiff" testing machines, however, the sample does not fail suddenly at the point C but continues to strain under diminishing load by slip on internal cracks along CD as shown (stage 4).

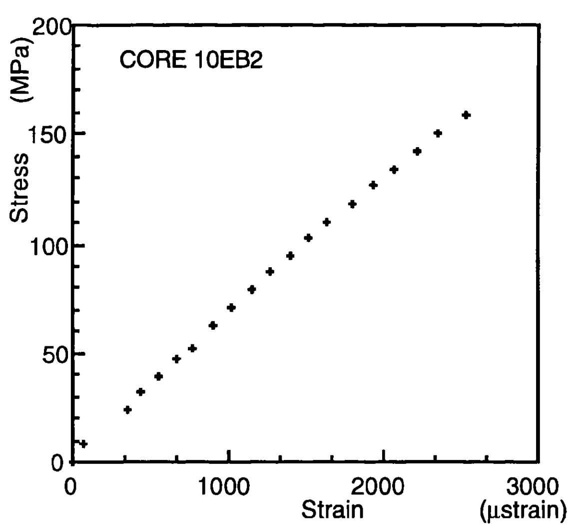


Fig (5.1a) Stress - Strain curve for basalt

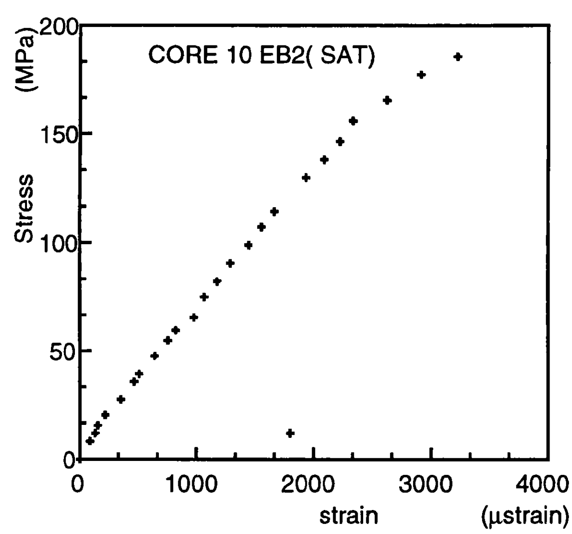


Fig (5.1b) Stress - Strain curve for basalt

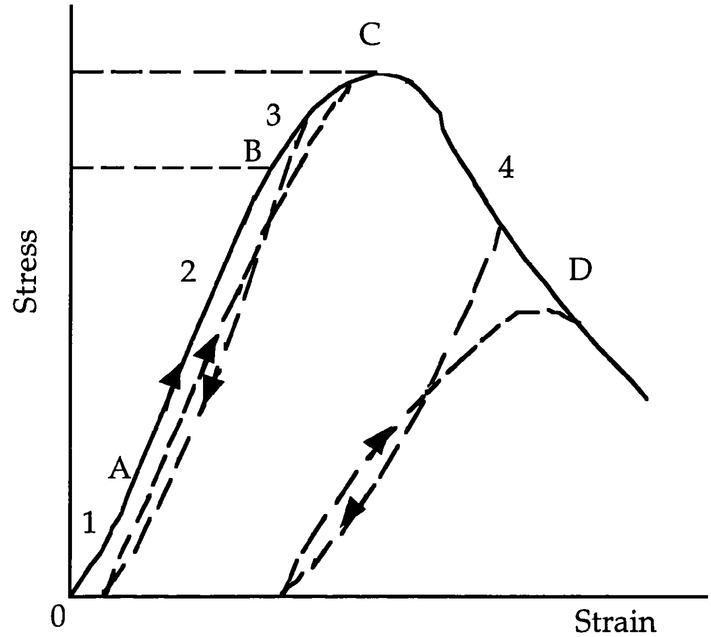


Fig 5. 2 The complete stress- strain curve
after (Jaeger and Cook 1969)

5 2 2- **Factors affecting uniaxial compressive strength**

The concept of strength as applied to rock material is in theory extremely simple and the behaviour of brittle material is assumed to be elastic up to the moment of failure, in practice things are not so simple, and the uniaxial compressive strength is influenced by many factors geological and methodological, and misleading results may be obtained if these are unconstrained or ignored.

a- **Methodological factors**:

a 1- Specimen aspect ratio:

The specimen aspect ratio (length/diameter) can have a significant effect on the apparent compressive strength of the rock material. Beniawski (1968) demonstrated that as this ratio increases the uniaxial compressive strength decreases up to a length/diameter ratio of 1.5, after which the compressive strength becomes independent (Fig 5. 3).

An aspect ratio of one has been proposed by Hardy (1959) but the theoretical and the experimental findings show that for this ratio the measured strength of the sample will be influenced by end effects. Hawks and Mellor (1970) using Bellas data, showed that for high aspect ratio a development of deviatoric stress at the corners of the specimen affects its measured strength. Attwell and Farmer (1976), ISRM suggested methods (1978), and Farmer (1983) adopted the ratio 2.5 as a standard for current testing and consider the ratio 2 to be the minimum acceptable length to diameter ratio

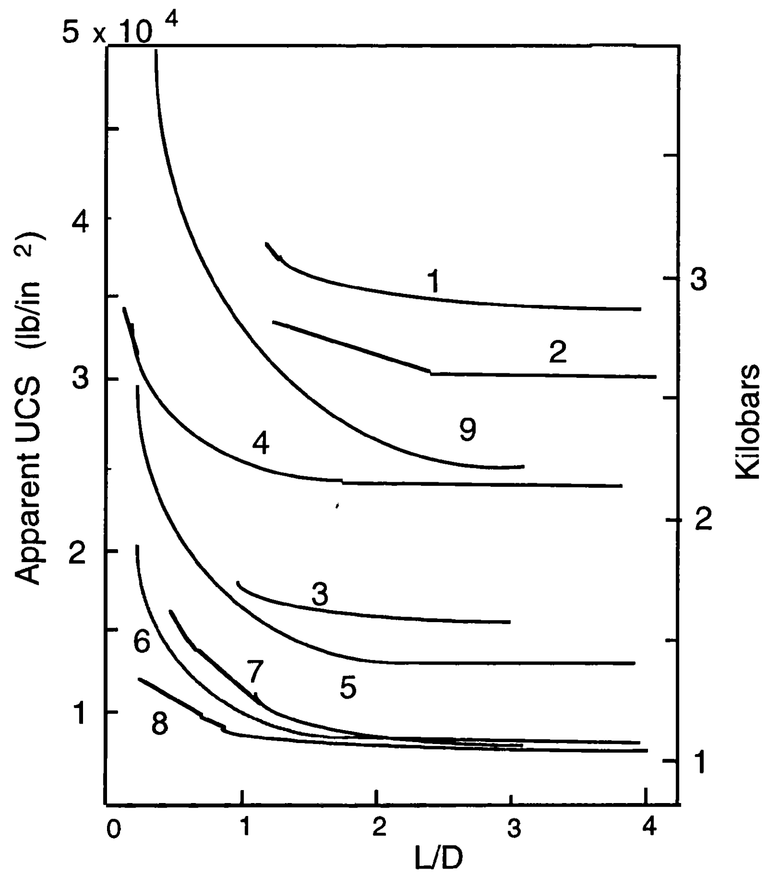


Fig 5.3 Influence of length/diameter ratio (L/D) on uniaxial compressive strength. 1- Westerly Granite; 2- Dunham Dolomite 3- Muzo Trachyte; 4- Pennant Sandstone; 5- Kirkby Siltstone 6- Ormonde Sandstone and Siltstone; 7- Darley Dale Sandstone; 8- Berea Sandstone; 9- Saturated Granite. (Hawks and Mellor 1970)

a 2- Rate of loading

Stagg and Zienkiewcz (1968), using sandstone and gabbro, demonstrated that in the uniaxial compressive test an increase in the loading to failure from 30 to 0.3 seconds results in a 50 to 30 percent increase in the measured strength of Berea Sandstone and gabbro respectively. Houpert (1970) Lama and vutukuri (1974) Atkinson (1989) also reported that in general the uniaxial compressive strength increases as the rate of loading increases. The ISRM committee 1978 suggested a standard loading rate of 1MPa/s.

Virgin material contains a distribution of submicroscopic flaws, with various sizes and orientations, called "Griffith cracks" which grow and propagate under load and their coallescence causes material failure. Griffith (1921) demonstrated that the energy required to propagate a crack is inversely proportional to the crack half length. Therefore, at low loading rates, only the largest flaws " critical cracks" start to propagate early causing material failure before the applied load is high enough to activate other flaws. This phenomenon is known as work softening and results in a low apparent threshold of the material. The specimen under test breaks down into relatively large fragments due to the smaller number of failure contributing flaws. When the load is applied more rapidly, a single crack which has a sub-critical growth velocity is not sufficient to relieve the increasing stress, therefore, a greater number of flaws participate, causing the fragment dimensions to be smaller and the apparent threshold for material failure to be higher. Again in the literature this phenomenon is some time refered to as work hardning. Fig (5. 4) shows the effect of loading rate on the apparent strength of rock material

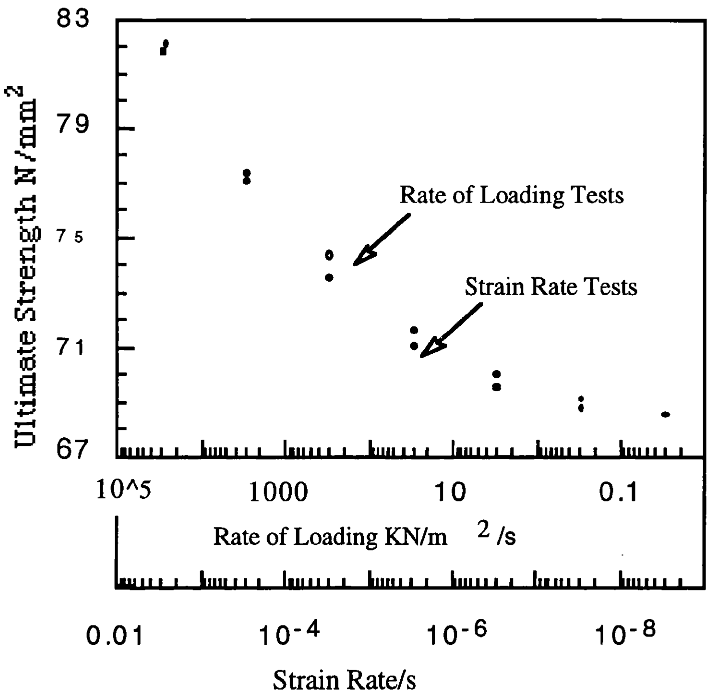


Fig 5. 4. Influence of strain rate and loading rate on the uniaxial compressive strength of Laurencekirk sandstone. From Sangha and Dhir (1972)

a 3- Specimen preparation:

For a uniform stress distribution at the platen-specimen contact, the surface of the specimen should be flat to the tolerance of 0.02 mm. The presence of asperities, foreign grains on the surface of the specimen, or departure from flatness can induce a premature failure of the specimen.

Hoskins and Horino (1968) using limestone and granite found that up to 0.002 inches surface roughness did not greatly influence the strength or the mode of failure. Above this, surface roughness of 0.003 inches causes a sample of granite to split axially.

The ISRM committee suggested method (1978) suggested that the end flatness should be to the tolerance of 0.02 mm.

a 4- End effect:

The mode of failure and the failure strength of the rock material is greatly influenced by the end conditions. The ISRM commission (1979), Hawks and Mellor (1979) and Stagg et al (1968) recommended the use of uncapped specimen since the capping material is generally weaker than the rock. Experience has shown that the capping material causes lateral tensile stress which lead to a premature failure of the specimen. The use of any kind of lubricant material is also not recommended since it's intrusion into the ends of the specimen is likely to set up a tensile stress and promote longitudinal splitting of the specimen.

b- **Physical factors:**

b 1- Relative Density and Porosity:

The relative density of a rock of volume (V) is the ratio of it's mass to the mass of an equal volume of water. The porosity is the volume of the porespaces in a rock expressed as a percentage of the total volume of the sample. Rock porosity is directly related to the degrees of interlocking or compaction, and microfracturing of the mineral constituents. The interrelationship between bulk density and porosity comes from the fact that low porosity rocks tends to have a bulk density equal or very near the mean density of their minerals constituents, while high porosity rocks on the other hand have relatively lower density than their mineral constituents, because of the presence of air or water filled voids. Attewell and Farmer (1976) mentioned that the overall magnitude of the grain bonding forces depends on the total area of the contact between individual particles which is inversely related to the amount of porespaces within the rock. Duncan (1969) stated that the nature and the extent of the voids within the rock material and the nature of the bond between the solid mineral aggregate have a strong influence on the strength of the whole rock. Judd and Hubber

(1962), Duncan (1969) and Irfan and Dearman (1978) demonstrated that within one given type of rock, strength increases with density increase. The negative exponential relationship between porosity and strength was reported by (Hochino 1974). Therefore, as a general rule when the porosity increases the intergranular contact decreases which leads to an overall decrease in the interlocking bond between the mineral grains, on one hand, and higher stress concentration zones on the other, resulting in a decrease in strength of the whole rock. In rocks porosity may be primary or secondary in origin. Thus in rocks considered to have low porosity, this property may still be an impotant factor influencing their deformational characteristics. In the present study of an igneous suite porosity is associated with and employed as an index of weathering.

b 2- Water content:

The presence of water within the porespaces and cracks of rocks influences their strength properties in two ways. The first is purely mechanical Hubbert and Rubey (1959) and the second is a complex chemical rock fluid interaction. Several authors have stressed that the strength of rock material decreases with increase in water content. Colbeck and Wild (1965) using samples of sandstone and shale at different moisture content have shown that the strength of a fully saturated sample is about 50 percent the strength of a completely dry one. Additionally, they demonstrated that the strength of a rock is extremely sensitive to the first small increase of the moisture content. Price (1966) stressed the pore water pressure and associated the effect of water with the shape and size of pores which are, in turn, a function of the dimensions, geometry, and packing of the grains. In fine grained rock even where porosity is low, water may fill a large proportion of the pore space, while in coarse grained rock it will occupy only a small percentage of the pore volume. The effect of water on sandstone of different porosities and saturation are presented in (Table 5. 1).

Table 5. 1. The influence of moisture content on rock strength.

Rock strength Porisity	Porosity (% vol.)	Air-dry pore water content (% pore vol)	Relative strength		
			Completely dry (%	Air-dry (%	Saturated (%
Pennant Sandstone	2.5	42 $\pm$3	100	51	45
Markham Sandstone	6	22 $\pm$ 5	100	57	45
Parkgate Rock	10	9 $\pm$ 1.55	100	68	-
Darly Dale Sandstone	19.5	3 $\pm$ 1	100	80	45

This data shows that the mechanical influence of water on rock strength is directly related to the degree of saturation. Choper and Paterson (1984) made the significant discovery that olivine in polycrystals containing less than 100 ppm H_2O exhibits mechanical behaviour identical to that of wet (more than 0.1 wt % H_2O) aggregates.

The mechanical effect of water results from the fact that when a rock is compressed in an undrained environment the pores tend to close, but being filled with an uncompressible fluid, hydraulic pressure (p) builds up in the pore spaces. This internal hydraulic pressure will cause a decrease in the failure strength of the rock as follows:

$$\sigma = P - p$$

σ Axial compressive strength

P Axial compressive stress

p pore water pressure

In an unjacketed condition, where free drainage operates, water will escape from the pore spaces and no internal fluid pressure will build up. Hawkes and Mellor (1970) stated that the presence of water on the internal surface of the rock produces a static fatigue which may involve a reduction of the surface energy, fracture energy, bond modification or interatomic shielding. It is suggested that at the crack tips in silicate minerals a hydration reaction occurs. Replacement of Si-O-Si links acros the crack tip by Si-OH: HO-Si links degrades the strength at the crack tip, and then cracks require just the weak hydroxyl bonds to be broken in order to propagate. In quartzitic sandstone Colbeck and Wiid (1965) suggested that the influence of the immersion liquid such as water is to reduce the surface-free energy of the rock and hence its strength. Den Brack (1991) in a very detailed study on quartzite showed that a small amount of added water can considerably reduce the strength of the rock in comparison with its strength in the dry state.

The influence of water on the uniaxial compressive strength of the rocks studied in this programme is summarized in (Table 2.4).

Table 5. 2 Influence of water on the uniaxial compressive strength of the studied rocks.

Sample Nber	R d (sat)	W ab%	UCS (dry) MPa	UCS (sat) MPa
Granite				
G3	2.66	0.305	178	158
G4	2.62	0.65	107	75
G5	2.61- 63	0.31-33	175	143
G8	2.59	0.95	78	49
Basalt B				
1E	2.5	5.4	21 - 39	25
6N	2.8	1.3	207	171
3N	2.83	1.4	196	147
4N			96	35
2S	2.52	5.2	63	37
10EB		2.8	90	59
8EB		3.67	38	17
8N	2.83	1.1	177	-
Basalt O				
O1	2.94	0.33	346	351
O2	2.96	0.3	-	299
Basalt L				
Ln3	-	-	388	-
Ln4	2.98	0.2	399	323
Dacite				
Ba2	2.52	1.2	250	174
Ba6	2.55	0.6	335	-
Ba8	2.47	3.14	218	175
Ba9	2.47	2.74	214	136
Qz dolerite				
Str2	2.93	0.75	182	170
St4	2.67	0.85	171-160	72
St9	2.89	0.48	166 - 175	186

b 3- Temperature:

The strength and deformability of rock is directly affected by temperature. The effect of temperature on strength is to a great extent similar to the effect of water (Zhong et al 1993). Generally an increase in temperature causes strength to decrease and deformability to increase. Hawkes and Mellor (1970) suggested that the decreasing strength is due to a differential thermal strain of the constituent grains leading to an intergranular displacement and intragranular strain. The change in the equilibrium water content (absorbed water) attending changes in temperature was also found to affect the rock strength. The defects induced as a result of these thermal fluctuations are the seeds of the future microcracks. Mellor and Ranney (1968-69), and Houpert (1969) showed that sub-freezing temperatures cause a dramatic increase in strength. Kumar (1968) Using basalt and granite found that at subfreezing temperature of -120 ^{o}C an increase of 50 MPa in UCS occurs. Handin (1967) has shown that at a confining pressure of 100 MPa, the maximun differential stress decreases from 950 MPa at room temperature to 440 MPa at 400 ^{o}C.

c- **Geological factors**:

c 1- Influence of mineralogy and fabric:

In order to study the influence of mineralogy and fabric on the mechanical properties of rock one must work progressively from the scale of a single crystal to that of the rock mass Friedman (1966). The physical and mechanical properties of a single crystal are determined by it's chemical composition, lattice structure, and lattice defects such as dislocations and vacancies. However, on account of the concentration of structural defects and imperfect bonding along the grain boundaries, where the least energy is required to propagate a crack, the mechanical properties of mineral grains as individual components become less important in the strength characterisation of polycrystalline aggregates. The way in which these minerals are assembled together (texture), bonding or cementation, mineral orientation, grain shape, and structural defects are, in fact, the important parameters upon which the strength of rocks largely depends.

Price (1966) in a study of a suite of Coal Measure rocks showed that for the same quartz content, a sandstone having a clay matrix is weaker than a calcite cemented one. He also suggested that in general strength was related to the quartz content of the rocks. In igneous and granoblastic metamorphic rocks, the strong nature of the bonding between the constituent silicate minerals and the interlocking nature of grain contact, is reflected in higher values for strength. Their strength, for the reasons stated earlier, cannot be characterised on a mineralogical basis, rather on texture, grain size, grain orientation, grain shape, and structural defects (Friedman. 1966, Mariam. 1970, Dayre et al 1986, Dyke et al 1991, and Hawkins et al 1991). Mineral alteration is another factor which should be considered, since slight grain boundary alteration greatly affect the intergranular bonds and consequently lowers the strength. Therefore, engineering classification of rocks on mineralogy and petrology alone can be misleading and even dangerous (Farmer 1983). This factor, alteration, has been discussed more fully in a discussion of weathering.

c 2- Grain size

Grain size is a factor known to influence the fracture strength of brittle materials. Experimental evidence from Paterson (1978), Brace (1961), (1964), Jaeger and cook (1969) Farmer (1983), Stagg and Zienkiewcz (1968), Hawkes and Mellor (1970), Bratlli et al (1992), and Atkinson (1987) all show that as the grain size decreases the ultimate strength of the material increases. Explanations for this have been based on Griffith Crack Theory which states that the fracture strength of brittle materials is governed by the initial presence of small cracks known as "Griffith Cracks". Skinner (1959), and Brace (1961), (1964) have identified Griffith Cracks with the same length as the maximum grain size. Later Spunt and Brace (1974) using the Scanning electron microscope, showed that long narrow and sharp-ended cracks typically occur at the grain boundaries, but when intragranular they are often sited along cleavage planes. Many authors have stated that the presence of such cracks within rock material affect the stress distribution in the vicinity of the void, and the stresses that should be carried by the material within the void are actually deflected round the margin of the void (Brace 1965, Hoek and Brown 1980 and Hoek and Bieniawski 1965). In consequence, a stress concentration develops at the vicinity of the crack tips. Griffith postulated that cracks would start to propagate under tensile stress when the stress at the crack tips reaches a critical value given by the following

Formula: $\sigma = \sqrt{\beta \frac{EY}{C}}$

Where E is Young modulus

Y is the surface energy

ß is a constant

C is the crack half length

From this it is clear that as the crack length increases the required critical stress value at the crack tip to initiate a crack decreases. Since the crack length to width ratio is directly related to the constituent mineral grain size, an increase in the latter would directly cause a decrease in the strength of the material. Simmon and Richter (1976) explain this behaviour by the fact that the effect of the structural defects such as porosity, secondary phase minerals, and imperfect bonding between the mineral grains is greater for larger grains than for small ones, and consequently the fracture energy required to propagate a crack in coarse grained rock sample will be less than for a fine grained one. Moreover, as the stress concentration at the crack tip is inversly proportional to the crack length, it is however easier, at an early stage, to stop a crack from propagating by an adjacent grain in fine grained rock than in coarse grained one. In this study the influence of grain size on the uniaxial compressive strength is summarized in (Fig 5. 5).

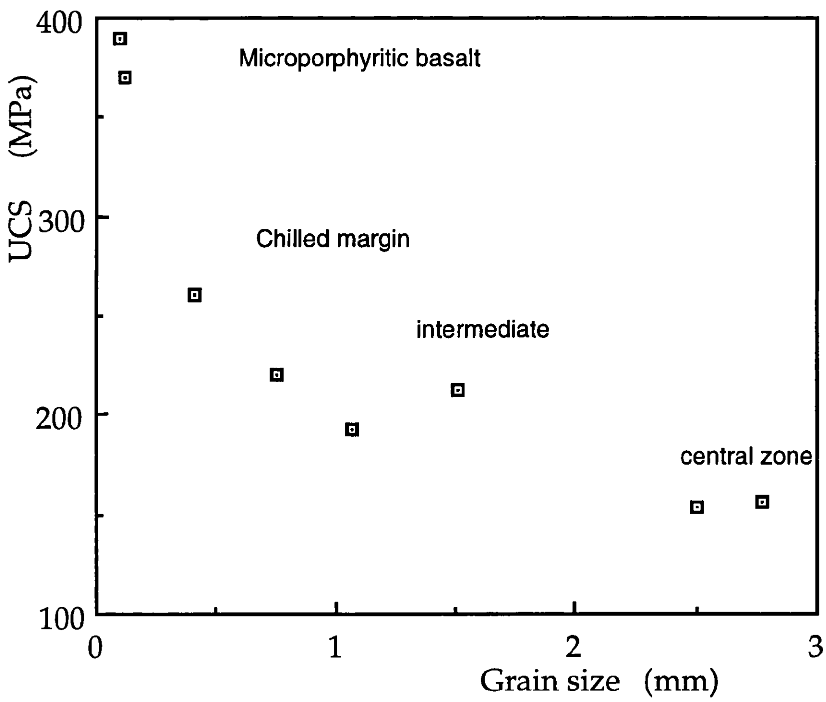


Fig 5. 5 Influence of grain size on the uniaxial compressive strength for basic rocks

c 3- Weathering:

Mineral alteration, crack formation and opening, void creation by leaching and the subsequent bond strength weakening and loss of cohesion upon weathering, discussed earlier in chap III , have a great influence on the resistance to compression in rocks. The change in the state of strength as weathering progresses has been studied using several weathering indices, i.e. porosity, water absorption, and density, (Sarafim. 1966, Hamrol. 1961, Irfan and Dearman. 1978, and Ghosh 1980). In all cases strength was found to decrease dramatically as weathering progresses, especially in the initial stages. Similar results were obtained when cracks and/or secondary minerals were used as a weathering index (Mendes. 1965, Onodera. 1974, Irfan and Dearman. 1978).

In this study, weathering was characterized by both physical and petrographical indices, i.e. porosity, water absorption, and the volume of secondary minerals and cracks. The uniaxial compressive strength decreases sharply as the volume of secondary minerals and cracks increases, especially in the early stages. This behaviour is clearly demonstrated by the power law function relating the volume of secondary minerals and cracks and the uniaxial compressive strength (Fig 5. 8). A similar relationship was obtained when the UCS is plotted againt either porosity or water absorption (Fig 5. 7). The strength response to weathering in such a manner suggested that a small amount of alteration on the grain boundary can cause a significant reduction in strength.

The mode of failure of weathered samples up to the weakened stage usually fail in a very quiet and gentle manner with a shear mode of failure or crumbling under load. Fresh samples, on the other hand, exhibit high strength and usually fail cataclastically in a violent manner.

c 4- Anisotropy:

A medium whose physical and mechanical properties are not similar in all directions is termed anisotropic. Rocks with a preferred orientation of their constituent grains such as sedimentary and metamorphic rocks are by definition anisotropic. This primary anisotropy (Friedman1967), on a macroscopic scale is manifest in pervasive bedding, schistosity, and cleavage planes and generally characterized by low shear and tensile strength. The secondary anisotropy (Habib and Bernaix, 1966),, Friedman,1966 is created by fracture and cracks opening as a result of geologic deformation . In both cases the resulting variation of compressive strength according to the variation of the loading direction is called strength anisotropy (Goodman. 1989).

The effect of anisotropy on the strength characteristics of the rock material has been extensively studied by Donath (1964) who found the ratio of minimum to maximum unconfined compressive strength of Martinburgh slate to be equal 0.17. El-jassar et al (1979) using sandstone from Bristol area, Bastiken (1985) on Scottish limestones, and Goodman (1989) found that the compressive strength parallel to the bedding to be always less than the stress measured perpendicular to bedding. Although, the rocks studied in the present programme are isotropic, the discussion of anisotropy here is just to show the different possibilities that may affect the strength of rocks.

5 2 3- **Results and discussion**:

In the course of the present investigation uniaxial compressive strength tests were undertaken according to the recommendation of the ISRM committee suggested methods (1978). Specimens of 25.4 mm diameter with an aspect ratio of two have been prepared from the available rock spectrum and uniaxially loaded to failure using an ELE 2000 kn digital compression machine.

The tests results within one rock type and between different rock types show a systematic variation compatible with the physical and geological factors discussed earlier, i.e. water content, the state of weathering, the grain size, and texture, i.e. the proportion of phenocrysts to ground mass.

a- Basalt:

The basaltic rocks tested are of two types, Markle and Hillhouse.The Markle basalt is a macroporphyritic alkali-olivine basalt ranging in texture from fine to coarse grained, vesicular to non vesicular, and fresh to highly weathered. The Hillhouse type is a homogeneous microporphyritic alkali olivine basalt.

Both Langside and Orrock basalts (Hillhouse) display very high values of unconfined uniaxial compressive strength, varying between 346 MPa and 400 MPa. The macroporphyritic alkali-olivine basalt, however, displays a wide variation in the uniaxial compressive strength between the lower and the upper portion of the same unit, i.e. 202 MPa for the dense basalt of the lower portion to as low as 20 MPa for the most slaggy and vesicular weathered basalt of the upper portion.

The high strength of both Langside and Orrock microporphyritic basalts reflects their physical properties and geological nature. They are characterised by high density of the order 2.96 - 2.98 and low water absorption values ranging between 0.1 - 0.56 %. They are amygdale and vesicle free and contain only a very small proportion of secondary minerals (less than 5 %). No geomechanical discontinuities have been observed either on the scale of 25.4 to 50.8 mm cores or on the thin section. Mineralogically the presence of olivine and augite phynocrysts embedded in the ground mass contributed to the strength increase of the rock.

The basalts of Loanhead quarry, with their great textural variation within single flows, exhibit a wide variation in the values of uniaxial compressive strength. The material from the base of the flow is characterised by up to 40 % phenocrysts of plagioclase set within a fine grained ground mass ranging from 0.1 to 0.3 mm in size. In addition 1 to 2 and rarely 4 % of amygdales and/or vesicles are present together with 10 to 12 % of secondary minerals. The uniaxial compressive strength of this portion varies between 177 and 202 MPa and indicates that the rock has a relatively high strength. Similar material with a lower proportion of phenocrysts, and higher amygdales and staining, exhibit a strength ranging from 70 to 96 MPa. The highly amygdaloidal and vesicular portion of the lava with coarser grained ground mass has a lower percentage of phenocrysts but higher secondary mineral content. Strength ranges from low to moderately high, i.e. 21 to 104 MPa. Finally the upper part of the lava which is weathered and completely rotten (Plate 3. 7.), on which the UCS could not be directely determined, but was evaluated from the mean value of Point Load Strength of 2.61 MPa at about 80 MPa. It is thought that this moderate strength is the result of burial compaction of the weathered top caused by later lava flows.

In quantifying the influence of secondary constituents, i.e. products of weathering and/or alteration on the strength of the rock, both physical and petrographical indices have been correlated with the UCS. Fig (5. 6) shows that the UCS increases with increase in density, and decreases in a power law manner as the porosity increases (Fig 5. 7). There is also hyperbolic relationship between UCS and the proportion of defects and weak components such as microcracks and secondary minerals, amygdales, and vesicles (Fig 5. 8).

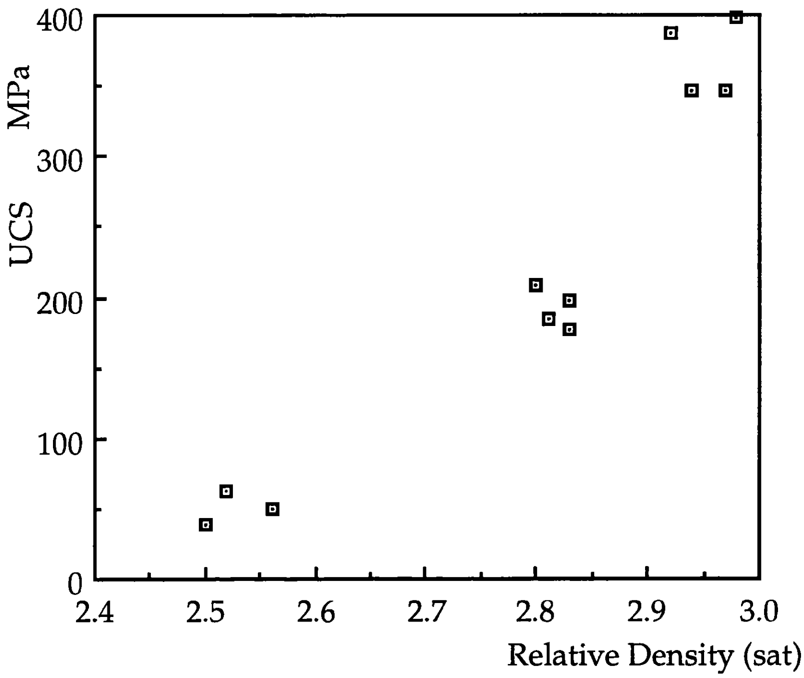


Fig 5. 6 The relationship between UCS and the relative density for saturated basalt

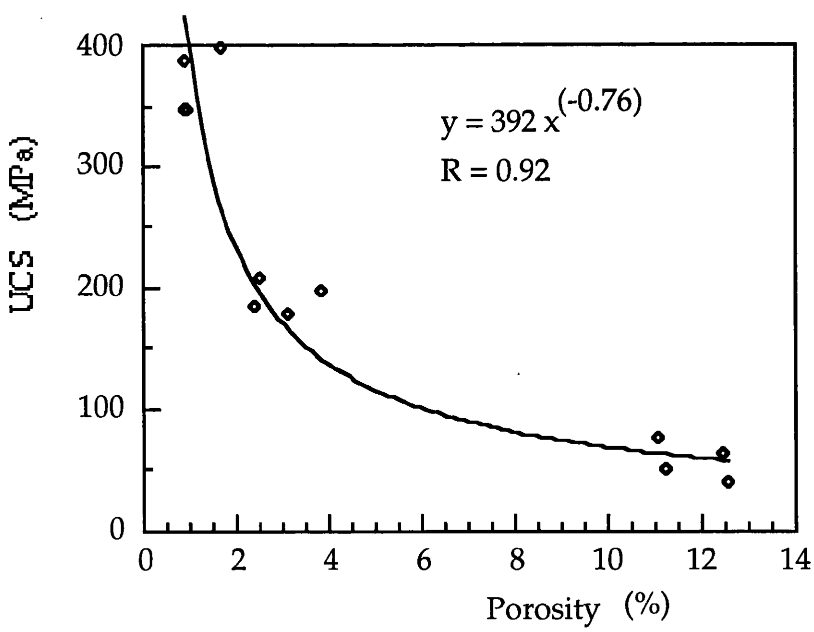


Fig 5. 7 The relationship between UCS and porosity for basalt

Different modes of failure were observed in the course of testing of these materials. The Langside and Orrock materials show a violent and sudden failure after which the core is shattered to small pieces. Loanhead basalt, on the other hand, exhibited a less violent conical failure pattern for the fine grained lower portion and very quiet failure where the specimen crumbled under load for the amygdaloidal and rotten basalt.

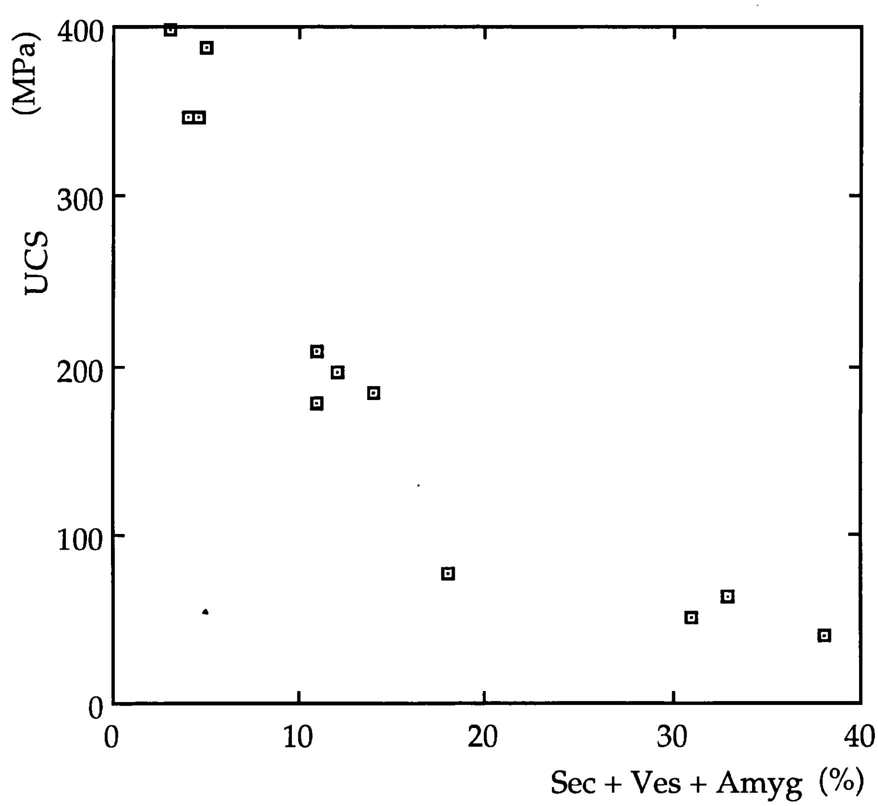


Fig 5. 8 Graph showing the influence of secondary minerals + Vesicles and amygdales on the UCS of basalt

b- Quartz dolerite:

The uniaxial compressive strength of the quartz dolerite displays a wide spectrum of values ranging from 220 to 120 MPa for the fresh to the slightly weathered or altered rocks. For the highly weathered material it was not possible to obtain cores of material due to the machining operation difficulties, therefore, the strength of this weak material was assessed indirectly from both PLS and R (Schmidt rebound number). With this rock, the significant factors are dominantly the grain size, texture and the weathering state as expressed by secondary mineral content and geomechanical discontinuities.

As revealed by the macroscopic and the microscopic study and stated by several authors such as Read (1956), Robertson et al (1937), Francis (1982), Walker (1952) the Midland Valley Sill has chilled margins with a homogeneous medium grained central part towards the top of which there is a zone of coarse and irregular crystallisation.

The chilled margin material is characterised by a very fine grain size ranging from 0.41 to 0.75 mm. This exhibits a very high strength varying between 220 and 266 MPa. As the grain size increases into the range 0.75 - 2.77 mm the strength of this medium to coarse grained central part, however, decreases to 150 - 210 MPa. This central zone consists of a dark grey to black material containing some patches of pink k-feldspar, and a clear whitish, coarse grained material comprising a high proportion of k-feldspar and micropegmatite. Test results revealed that the whitish material has a lower UCS than the dark material and does not exceed 176 MPa. In some instances, however, where samples were collected from locations close to shear zones strength fall to 120 MPa despite the fact that the rock looks fresh. Under the microscope the rock displays a series of filled intragranular microcracks perpendicular to the longest axis of the plagioclase laths.

The detrimental effect of secondary constituents such as secondary minerals (weathering and/or alteration products), cracks, and voids on the mechanical properties of the rocks is clearly demonstrated by plotting the micropetrographic index (Ip) against the UCS (Fig 5. 9). Relative to increase in the amount of secondary minerals + cracks UCS decreases in a negative linear manner (Fig 5. 10).

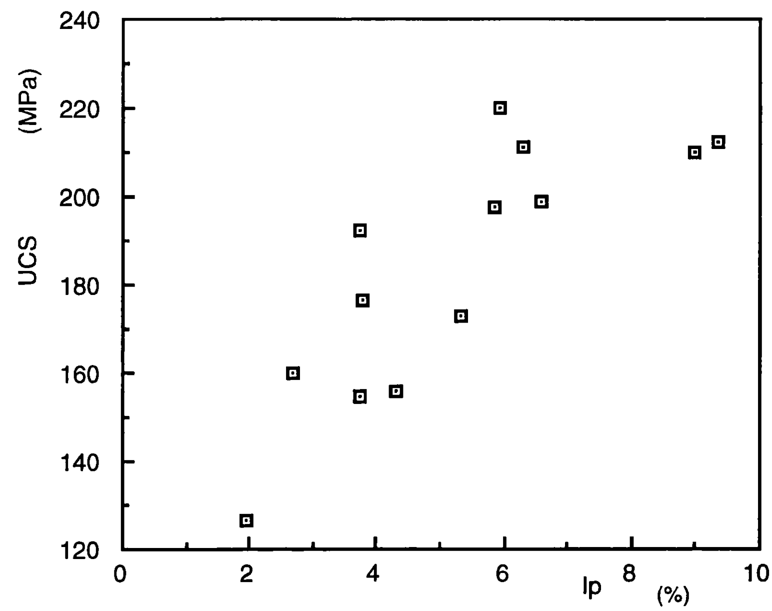


Fig 5.9 The relationship between UCS and the micro-petrographic index for quartz dolerite

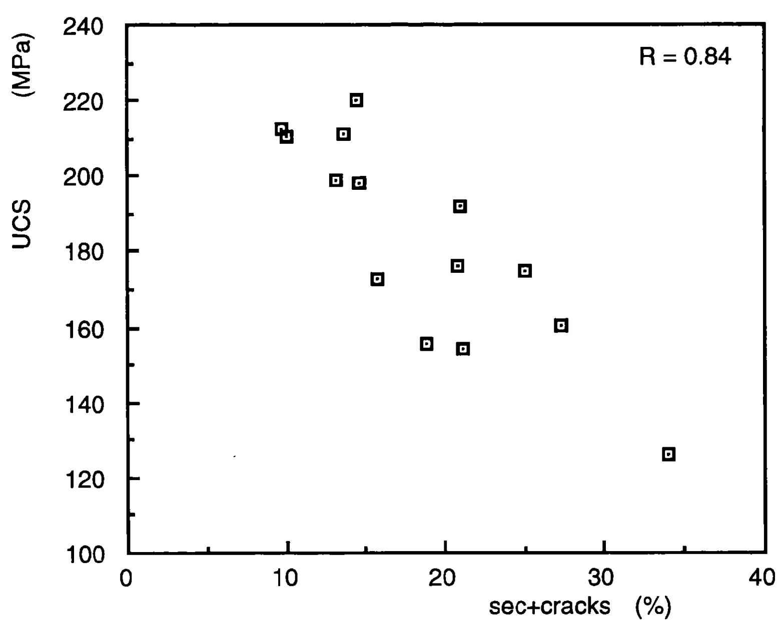


Fig 5.10 The relationship betwen UCS and the volume of secondary minerals + cracks in quartz dolerite

The dominant mode of failure in the quartz dolerite is the combined cataclasis/cleavage mode which is characteristic of the stronger varieties of rock, and shear failure which is quite frequent in samples of high strength (120 - 150 MPa). A violent sudden failure was common among all the specimen tested, but the stronger the specimen the more violent the failure.

c- Granite:

The granite tested in this programme comes from three different quarries. Craignaire Quarry in Dalbeattie (Criffel pluton), Bruce Plant Quarry in Peterhead and Craigenlow Quarry in Dunecht (Aberdeenshire). The samples collected from the quarries cover a wide range of weathering states and grain size, and consequently show a wide variation in their physical and mechanical properties. For Peterhead, Dalbeattie, and Dunecht the UCS varies in the range 69 - 175, 75 - 257, and 78 - 237 MPa respectively. The pink granite from Peterhead has the coarsest grain size, greater microfracturing and a more granular texture with less interlocking between the mineral grains than the two other granites, and exhibits the lowest range of strength. Dalbeattie and Dunecht grey granite exhibit a grain size in the range 0.63 to 2.6 mm, and 1.12 to 1.5 mm respectively, and a granular texture with more interlocking between the grains, resulting in higher ranges of strength. The influence of grain size and texture on the strength of these granites is shown in (Table 4.3).

The variation of density and porosity as they are indices of the state of soundness and freshness (including secondary mineral contents and amount of microcracks and voids), relates to strength variation in a similar fashion (Table 5. 3). In Fig (appendix VI) strength is plotted against density, and shows that as density increases, and the rock becomes fresher, the strength increases in a linear manner. With the porosity, however, the strength decreases exponentially as the porosity increases (Fig 5. 11).

Table 5.3. The influence of Grain size on UCS for granite

Sample	UCS (MPa)	Mean Grain Size (mm)	Density (sat)
Gd1	181	2.7 ± 1.54	2.67
Gd3	178	2.42 ± 1.36	2.66
Gd10	257	0.63 ± 0.23	-
Gd5	175	2.39 ± 1.53	2.62
Gd7	195	1.22 ± 0.59	2.63
Gd9	237	the same as Gd7	2.64

The effect of secondary mineral content and the volume of pores and microcracks is to a great extent similar to the effect of porosity and density since they are closely related. For all the granite studied the graphical representation of the strength vs weathering and alteration products show an inverse relationship. When strength is measured against the petrographic index a logaritmic relationship is obtained (Fig 5. 12).

The effect of pore water content on the UCS is summarised in Table (5. 2) where it is clearly shown that an increase in water content is accompanied by a systematic decrease in strength.

In uniaxial compression testing of granitic rocks, different modes of failure have been observed. Cataclastic failure, where the sample collapses violently, and shatters into tiny pieces has been observed to occur with very high strength specimens having a UCS greater than 230 MPa. The combined cataclasis/cleavage is the more frequent mode of failure observed. When the specimen collapses, conical end fragments are left, with long slivers of rocks from around the periphery. It has also been observed that cracks which tend to split the specimen develop in this mixed mode of failure, especially in the coarser grained variety of the granite. Shear failure, in which a specimen

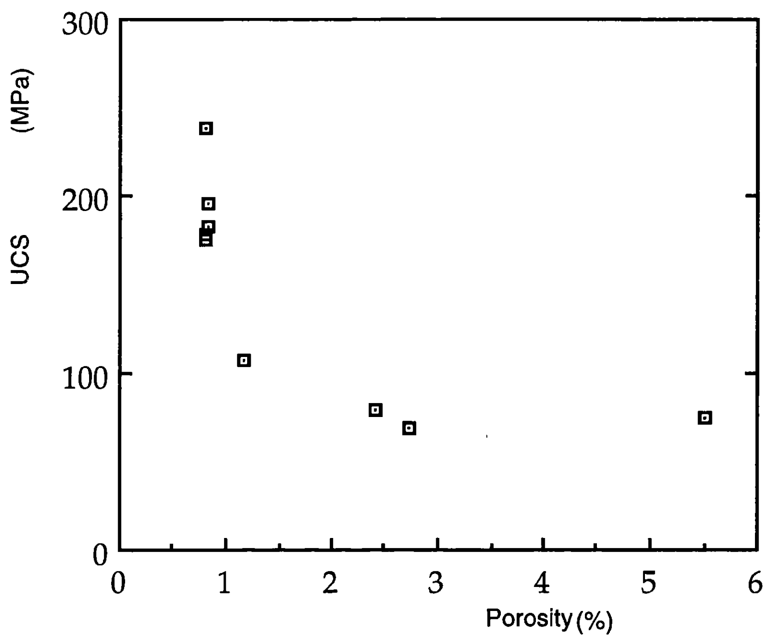


Fig 5.11 The relationship between uniaxial compressive strength and porosity for granite

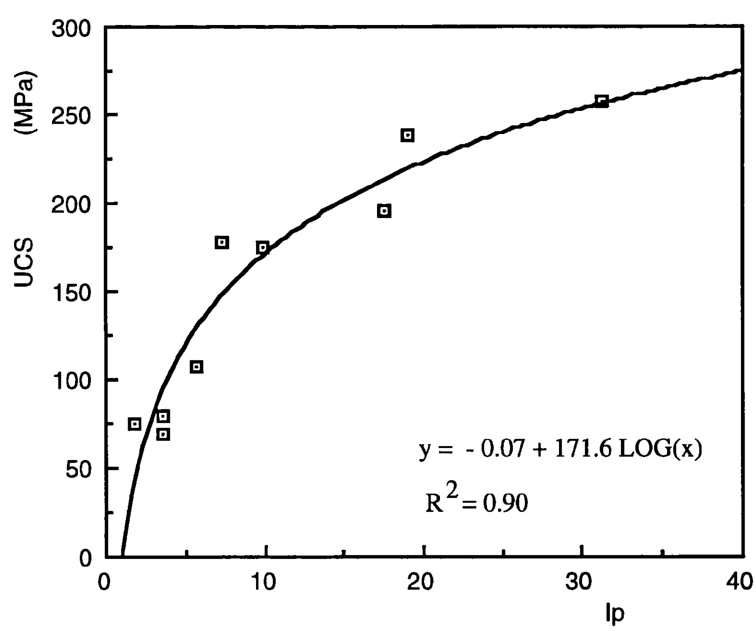


Fig 5.12 The relationship between the UCS and the micropetrographic index for granite

fails along a distinct single shear plane are also common, especially among moderate strength granite samples. Low strength samples, at failure, collapse by a general internal crumbling and become more or less friable. Contrary to high and extremly high strength granite, the low strength granites fail in a very quiet manner.

d- Dacite:

The Dacite investigated in this study comes from the Lucklaw Hill intrusion in north Fife. It is a very fine grained rock with phenocrysts of plagioclase-feldspar (andesine) and biotite embeded in the dominantly quartzo-feldspathic ground mass, with zircon as an accessory mineral.

The uniaxial compressive strength varies between 205 and 335 MPa signifying that this rock is an extremely strong material. From microscopic and scanning electron microscope observations the weaker samples are characterised by completely altered phenocrysts (Plate 5. 1), On the other hand, the strong varieties have less weathered phenocrysts. The grain size of the ground mass varies from cryptocrystalline to very fine grained and seems to have no influence on the strength since samples having both grain sizes and the same degree of alteration of their phenocrysts exhibit the same order of strength. The variation of the physical indices such as porosity and density with strength suggest that weathering and/or alteration are the main strength controlling factors for this material (Table 4. 2).

Two modes of failure have been observed during testing, the combined cataclasis/ cleavage and cataclasis. In the cataclastic mode, specimens fail by a general internal crumbling and shatter into tiny pieces. This mode is characteristic of the stronger varieties with a UCS generally more than 250 MPa. The combined cataclasis/ cleavage mode, however, is characteristic of specimens of strength lower than 250 MPa.

Plate 5. 1 Photomicrograph showing altered plagioclase phenocryst in dacite

4 2 4- **Conclusion**

The present programme showed that strength of rocks is affected by both primary and secondary factors. Grain size and texture as primary factor have been demonstrated to affect uniaxial compressive strength. the influence of grain size is best demonstrated when comparing strength of fresh basalt, fine grained quartz dolerite from the chilled margin, coarse grained quartz dolerite from the central part of the sill, and the medium grained quartz dolerite. This is actually the general pattern, but where the difference in grain size is small higher strength is associated with specimens with a more interlocking texture. The influence of texture, can be easily deduced from the fact that for similar secondary mineral content and cracks, quartz dolerite exhibits higher strength than granite, due to it's ophitic texture. Weathering as a secondary factor quantified by index properties such as porosity, and water absorption, volume of secondary minerals and cracks is also demonstrated to markedly affect the strength of rocks. The mineralogy, however, is not of great input in the strength characterisation of rocks.

Point Load Strength Test

5 3 1- Introduction:

The Point Load Strength Test is a tensile test from which the uniaxial compressive strength (UCS) as well as the Modulus of Elasticity can be empirically derived. It is regarded as a convenient index property for strength classification of rock material since it can be carried out easily in the field without any specimen preparation (Franklin. 1971, Broch and Franklin. 1972 and Beniawski. 1975).

The Point Load equipment developed at Imperial College by Franklin (1971) was used for field work. it comprises a small hydraulic pump and a ram with adjustable loading frame able to test core or rock lumps of different sizes .The tested specimen is compressed between two conical plattens and failure is by tensile splitting.

The calculation of the point load strength requires two quantities to be measured , the distance "D" between the conical platen points and the load "*P*" at which the specimen split up . It is then obtained using the following formula :

$Is = P/D^2$

P. the load at which the specimen break up

D. the distance between the two platen contact points

Other formulae have been suggested to overcome both the size and the shape influence on the Point Load Strength. Sundae (1974) studied the effect of varying the volume of a disc shaped specimen on the Point Load Strength index and suggested the following formula :

$Is = P/Dt$

P. the load at which the specimen fail

D . the diameter of the disc

t . the thickness of the disc

Broch and Franklin (1972) presented a size correlation chart for the Point Load Test to overcome the size of the core and recommended the 50 mm core size as a reference for correlation. Beniawski (1975) also studied the effect of specimen diameter on the Point Load Strength Index. He recommended that the strength classification should be based on the uniaxial compressive strength, and proposed a chart for index to strength conversion, based on the core diameter. He found a factor of 24 to be the most convenient when using NX cores. Al Jasser et al (1979) in their study of the Carboniferous Limestone in Bristol area found that the best correlation between the Point Load Strength and the Uniaxial Compressive Strength occurs when cores of 76 mm diameter and lumps of 70 mm thickness are used. the correlation factor suggested was 24 to 30, i.e. UCS = 24 Is

Since the Point Load Test can be carried out on irregular lumps, this qualified it as a very important one in the study of weathered rocks which are sometimes very difficult to be machined into regular shaped test specimens. Although, it is well demonstrated that the diametral Point Load is more convenient and more repeatable than on irregular shaped specimens, the lower repeatability can be offset by testing a large number of specimens.

The effect of physical and geological factors on the Point Load Strength is similar to their effect on the Uniaxial Compressive Strength.

In this study a large number of cores and lumps from different locations and different grades of weathering have been tested. The cores used have a diameter of 50 mm , the lumps are about 50 to 60 mm in thickness. The number of cores tested are at least two for each sample, while for the lumps the number exceeded 15 in general.

5 3 2- **Results and discussion:**

In this study Point Load Strength tests was carried out according to the requirement of the "suggested method for determining the point load strength index" (ISRM 1973) using an ELE lt apparatus. The calculations of the point load strength index were done using the formula,

$PLS = P/D^{2.}$

The investigation covered different rock types, (i.e. Basalt, quartz dolerite, dacite, and granite), each of which displays a wide spectrum of weathering (from grade 1B to IV). Fresh basalt, dolerite, dacite, and granite all display a strength value > 7 MPa, while their highly weathered varieties display values as low as 0.95 and 1.24 MPa. The range of variation between different rock types and within rock types is displayed (Table 5. 4).

No systematic variation of the Point Load Strength with mineralogy appears to operate, e.g. fresh basalt and dacite are characterized by values of the same order of magnitude, and similarly quartz dolerite and granite. However, within one rock type, as weathering increases the Point Load displays a systematic decrease. The relationship between several weathering indices, e.g. porosity, water absorption, density, and secondary minerals + cracks, and PLS is one of systematic fall in strength as weathering increases. When porosity and increases and density deceases the Point Load Strength exhibits a dramatic linear decrease (Fig 5. 13 & 14). The latter feature can be observed with both the volume of secondary constituents Fig (5. 14) or the micropetrographic index (Fig 5. 15).

Table 5. 4 Showing some Point Load Strength data from the studied rocks.

Sample	Pls (MPa)	UCS (MPa)	Porosity (%)
1N	6.2	184	2.89
2N	2.6	-	-
3N (lw)	6.2	197	3.8
4N	4.2	96	10.34
1E	1.3	40	12.56
O1	10.3	346	0.9
Ba1	7.5	223	2.8
Ba2	7.6	205	-
Ba3	9	276	3
Ba9	8.2	214	-
Sz2	7.8	113	-
St3	10.2	211	2.5
St6	11.2	213	1.68
St12	10	160	2.25
st22	9.5	176	2.06
St30	1.17	-	9.13
St31	1.39	-	7.71
St32	2	-	8.74
Gd4	3.9	107	1.17
Gd5	8	175	0.813
Gd6	1.8	69	2.74
Gd9	10.5	237	0.809

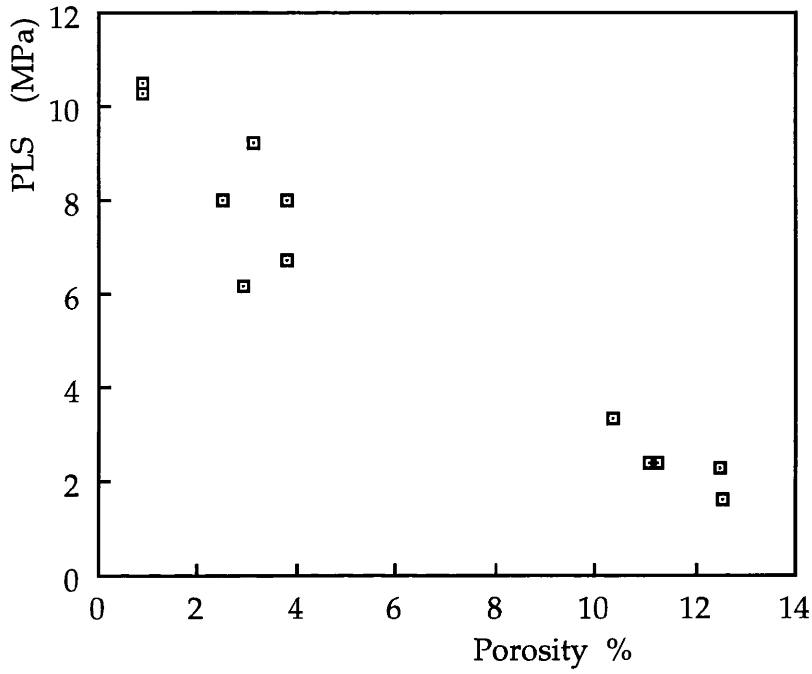


Fig 5. 13 The relationship between Point Load Strength and porosity for basalt

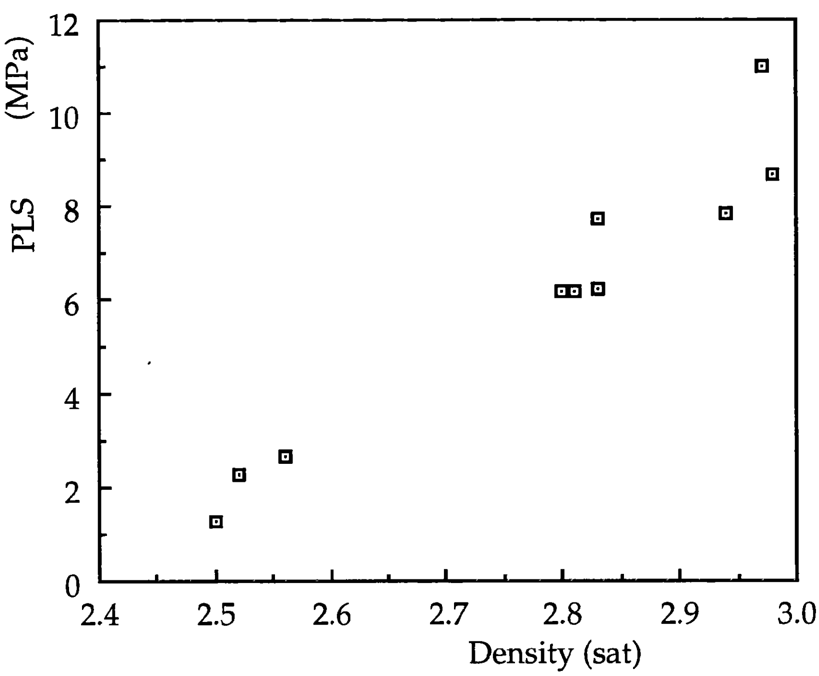


Fig 5. 14 The relationship between Point Load Strength and density for basalt

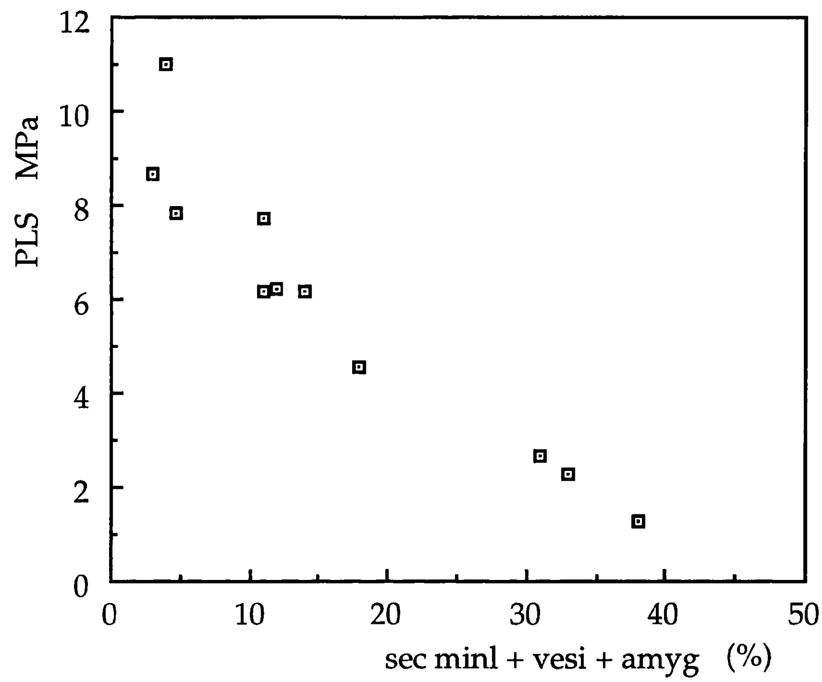


Fig 5. 15 The influence of secondary mineral, vesicles and amygdales content on the Point Load Strength for basalt

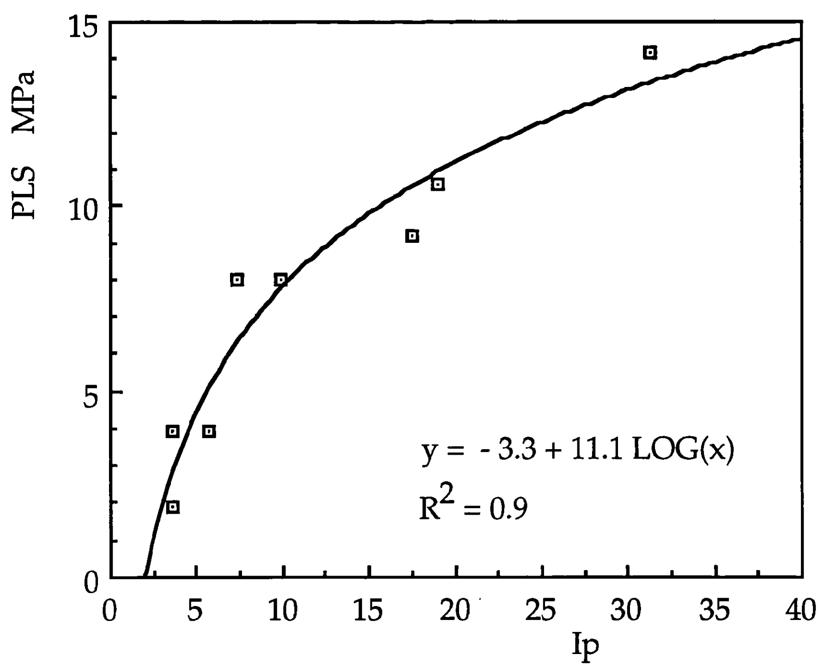


Fig 5. 16 The relationship between the point load strength and the micropetrographic index for granite

The Point Load Strength Index has been used in the mechanical characterisation of weathered rocks (Fookes et al 1971), i.e. as weathering grade increases from I to V the PLS falls sharply to less than 1 MPa. Lumb (1983) in a study of weathered rocks from Hong Kong found that Point Load Strength was a very useful test for discriminating between slightly and moderately decomposed rocks where a big change in strength occurs. The point load strength index for this discrimination was 2.5 MPa.

In the present study it has been demonstrated that values below 2.6 MPa are characterized by high water absorption, strong weathering and staining and a high volume of secondary constituent while the UCS is less than 70 MPa.

The correlation between Point Load Strength and the UCS exhibit conversion factors varying from one rock type to another within the interval of 17 - 30 for granite, dacite and basalt (Fig 5. 17a, b). Quartz dolerite on the other hand exhibits a conversion factor of 15 with very low coefficient of correlation.

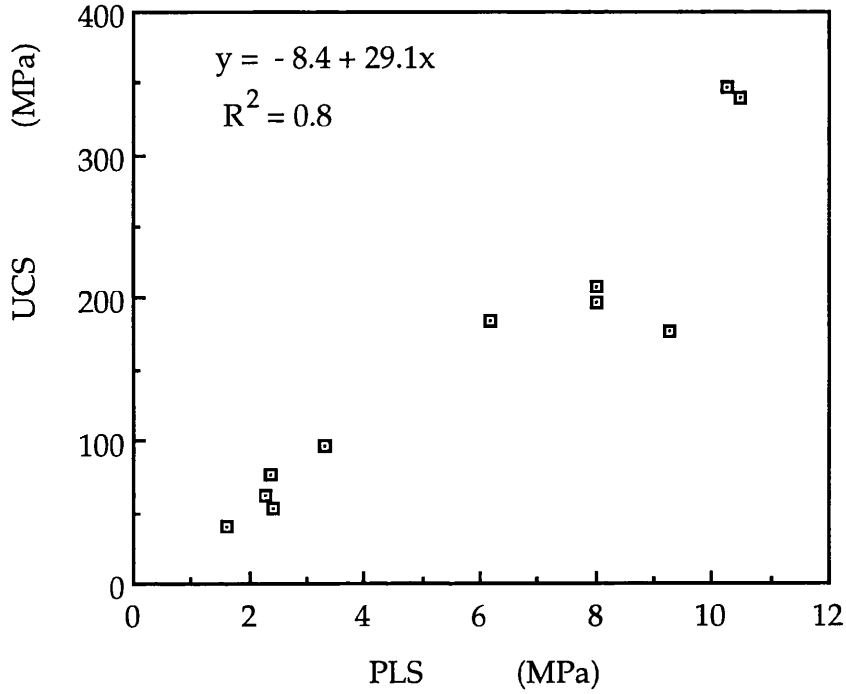


Fig 5. 17a The relationship between
UCS and PLS for basalt

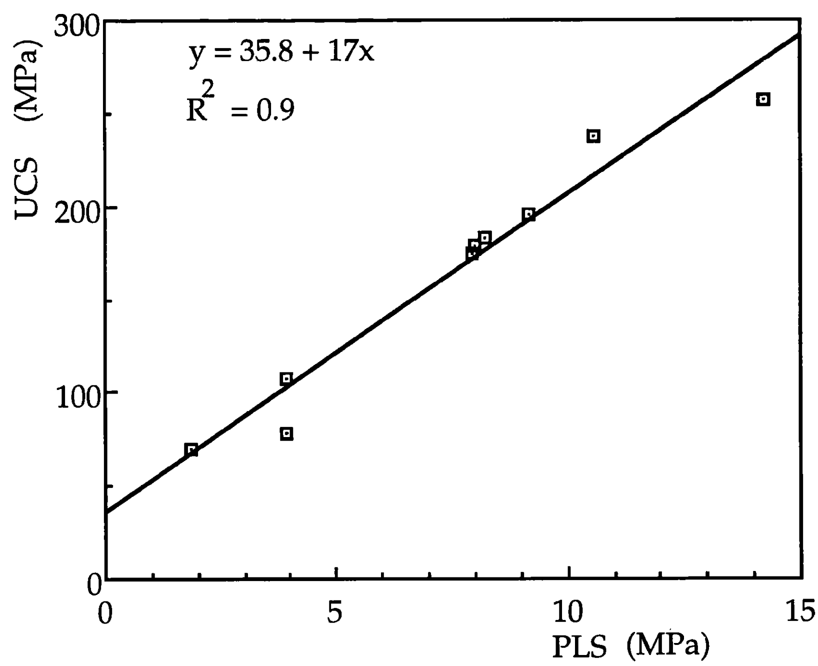


Fig 5. 17b The relationship between
UCS and Pls for granite

Fookes (1971), Broch and Franklin (1972) and Beniawski (1974) proposed the Point Load Strength as a replacement for to the uniaxial compressive strength and later Beniawski (1975), for strength classification purposes, recommended the use of the uniaxial compressive strength calculated from the point Load Strength index since the strength ranges for the former is internationally known while those of the later are not.

The lower reproducibility of the Point Load Strength values as compared to the UCS results from the fact that in the Point Load Tests the energy which is build up in the specimen is released only along one surface, unlike the uniaxial compressive test where the energy is released along many surfaces.

Schmidt Hammer

5 3 1- **Introduction**:

Originally the Schmidt Hammer was designed by Schmidt (1951) as a non-destructive test for assessing the strength of concrete. Subsequently, it was found to be a useful and convenient instrument for rock strength assessment, as it can be used in both the field and the laboratory. In addition the Rebound Number (R) can be an index property for Uniaxial Compressive Strength, Point Load Strength and Modulus of Elasticity (Deer and Miller 1966, Aufmuth 1974, Carter and Mills 1976, Irfan and Dearman 1978, Al jassar et al 1979, and Johnson and Degraff 1988). The Schmidt Hammer operates by pressing a plunger against the rock surface which causes a spring loaded hammer to fall, applying a known amount of impact energy to the rock. The rebound of the hammer, as a percentage of the forward travel, is indicated on a scale and taken as a measure of the rock hardness and known as the Rebound Number (R). It is proportional to the strength and elasticity of the material. Ramsay et al (1974) described it as a simple test for quantitative assessment of the toughness, elasticity and the state of freshness of the rock material .

The strong correlation between the R and the Uniaxial Compressive Strength for a variety of rock materials enabled Deer and Miller (1966) to establish a correlation chart (Fig 5. 20) from which the uniaxial compressive strength can be derived if it's dry density is known. The chart has been recommended by many authors such as Duncan (1969), Carter and Sneddon (1969) and Dearman (1974), but the Geological Society (1977) found that there is only 75 % probability that the laboratory determined uniaxial compressive strength would lie within 50 % of the strength derived from the Schmidt Hammer results using the correlation chart of (Deer and Miller 1966). They suggested, however, that the Uniaxial Compressive Strength can be obtained by multiplying the R by the dry unit weight of the material. In the author's experience, based on the results obtained in the present study, uniaxial compressive strength values derived from R using Deer and Miller's correlation chart are quite realistic if the effect of grain size is taken in consideration. Microporphyritic basalt and dolerite display realtively small differences in their R values, 63 - 65 and 58 - 60 respectively, while their UCS values are in the range of 346 - 399 for microporphyritic basalt and around 190 - 210 MPa for fresh medium grained quartz dolerite with a maxium of 260 for the chilled margin material. Similar behaviour was observed between granite and dacite, where the latter displays higer UCS and lower R values than the former. Moreover, Deer and Miller (1966) found a good relationship between R and Young's Modulus. Irfan and Dearman (1978) confirmed this, and suggested the use of R as an index property for Young's Modulus estimation.

A detailed statistical study by Poole and Farmer (1980) has shown good repeatability and representativness of the test if a minimum of 5 impacts are made at each measurement point and selecting the peak rebound value at each point was selected and taken as R of the material. The ISRM committee (1978) recommended 15 impacts with the mean of the highest readings be taken as R.

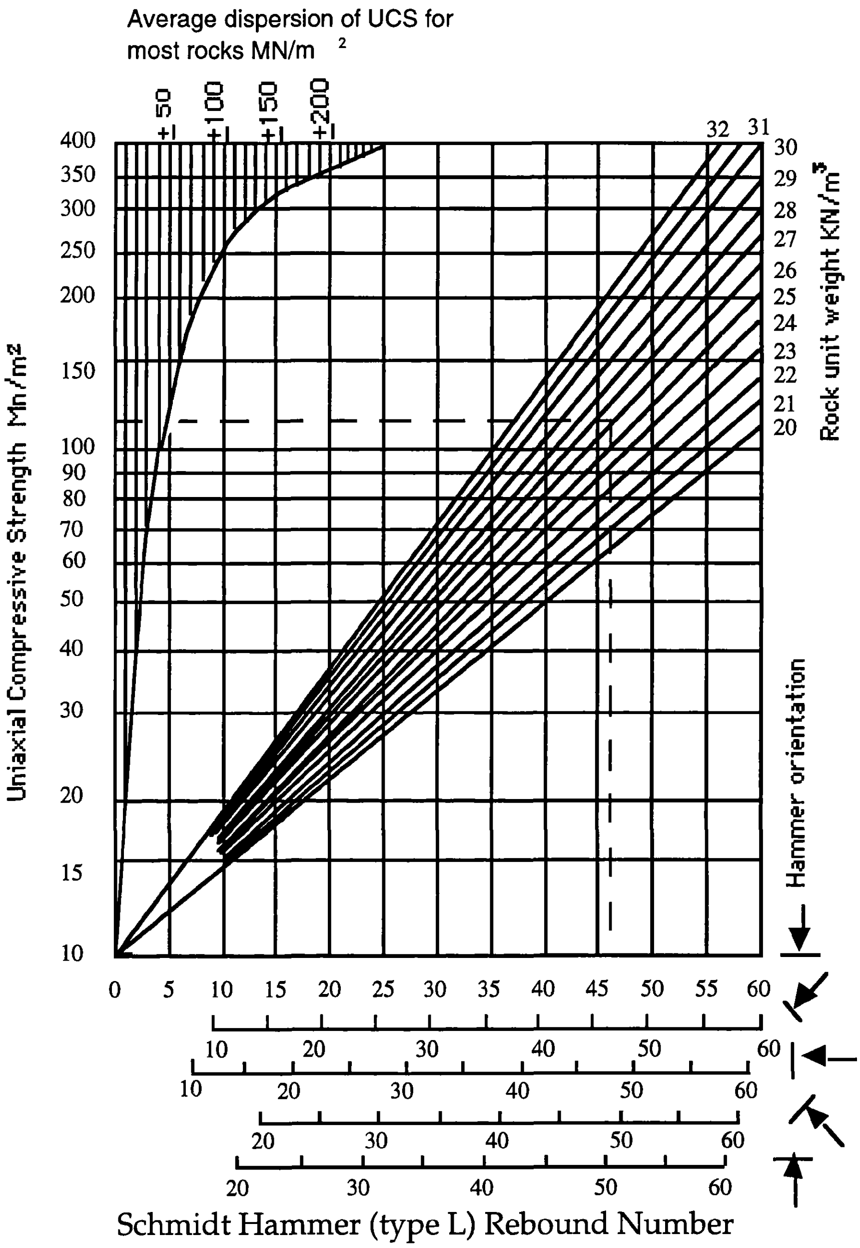


Fig 5. 18 The relationship between Schmidt hammer reboundnumber and the uniaxial compressive strength (after Deere and Miller 1966)

5 3 2- **Factors Affecting the Rebound Number**

The consistency and the representativeness of the Rebound Number depends on one the hand on some technical factors related to the state and function of the hammer, and on the other hand on the state of the rock face. These factors are:

1- The plunger of the hammer must be in good condition. It was observed when the plunger is worn the hammer can give variable energy of impact which leads to a variable R (Poole and Farmer 1980).

2- The plunger should be tightly held perpendicular to the rock face in a vertical plane, unless corrected .

3- The surface of the specimen should be flat and smooth at least over the area covered by the plunger.

4- If the specimen under test is in a wet condition the results can be unreliable especially if the material is weak .

5- It was found that the size of the specimen under test affects the rebound number (Carter and Mills 1976). The ISRM (1978) recommended that the area under the plunger should be free from cracks or any localised discontinuities at least to a depth of 6 centimetres beneath the spot.

5 3 3- **Results and discussion**:

The average of 15 readings have been taken both in the field and in the laboratory for different rock types and different grades of weathering. The samples tested in the laboratory had average dimensions of 20 x 30 x 20 cm, and were tested with the same instrument used in the field. The instrument is a Schmidt Hammer type NR which corresponds substantially to type N but is fitted with a special recording device. The impact energy of this hammer is 0.225 mkg. For all samples tested, the laboratory readings were systematically lower than those taken in the field.

The results obtained in the present study Table (5. 5) from several rock types with different weathering states display a significant range in R. Among fresh rock types, (basalt, granite, dacite, quartz dolerite), variation is strictly related to their densities. Fresh basalt from Langside and Orrock having the higher densities 2.96 - 2.98 exhibit the higher rebound numbers 62 - 67. Dacite although it has higher strength than quartz dolerite displays lower rebound values on accounts of it's lower density. Within one rock type, however, the variation in the rebound number is mainly a reflection of the state of freshness of the rock, e.g. fresh basalt is characterized by a rebound number usually higher than 62 while weathered basalt from Beith exhibits values as low as 30. Similar trends were observed for granite and dolerite where heavily weathered samples have no rebound at all. The influence of weathering on R can be determined from any of the index properties such as the volume of secondary minerals and cracks, density, water absorption, and porosity. Fig (5. 21a, b) shows that as the volume of secondary minerals and cracks increase R decreases linearly. Similar relationships are obtained from water absorption (Fig 4. 22a, b) as well as porosity. Duncan (1969) demonstrated a similar relationship with saturation moisture content, the decrease in the Rebound Number during the early increase in the saturation moisture content is more dramatic. In the present study, it is shown that as the water absorption capacity increases the Schmidt Rebound Number decreases and the dramatic decrease occurs in the early stages before it reaches 3 percent.

Table 5. 5 Showing the density influence on Schmidt Rebound Number among different rock types at their fresh state.

Sample	Sch R No	Density (sat)
O*1	62 - 66	2.96
O*1	64 - 65	2.94
Ln*1	64 - 67	2.98
1N*	50 - 53	2.81
2N*	28 - 31	-
1E*	29 - 31	2.48
3N*(lw)	50 - 54	2.83
8N*	52 - 56	2.83
St6+	59 - 52	2.87
St8+	58 - 60	2.91
St16+	56 - 59	2.88
St19+	62 - 64	2.91
St24+	54 - 57	2.79
St31+	31 - 33	2.69
Ba1	54 - 58	2.52
Ba2	54 - 56	2.51
Ba3	55 - 58	2.54
Gd1	59 - 62	2.67
Gd3	61 - 64	2.66
Gd4	42 - 45	2.62
Gd7	58 - 59	2.64
Gd9	62 - 64	2.64

* = basalt, + = quartz dolerite, Ba = dacite, Gd = granite

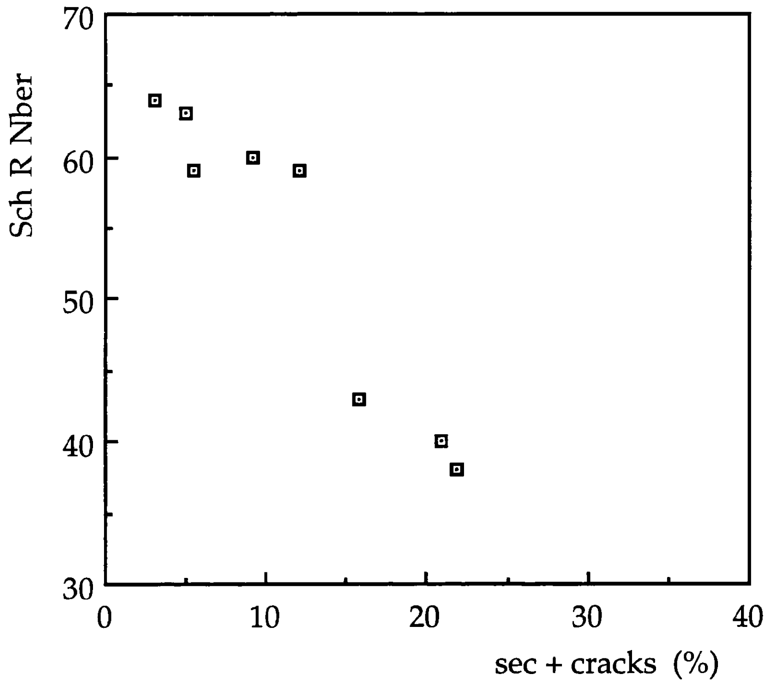


Fig 5. 19a The relationship between Schmidt Rebound Number and the amount of secondary minerals and cracks for granite

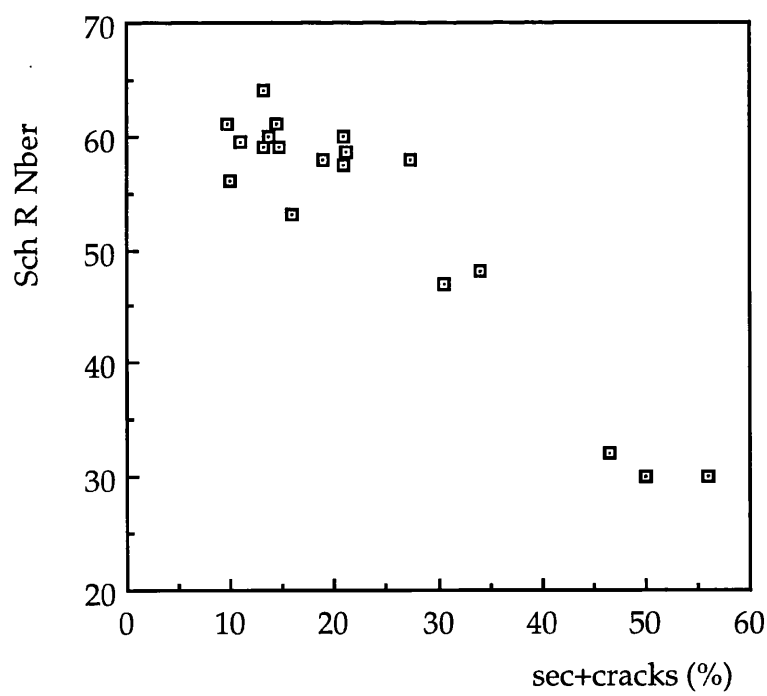


Fig 5. 19b The relationship between Schmidt Rebound Number and the amount of secondary minerals and cracks for quartz dolerite

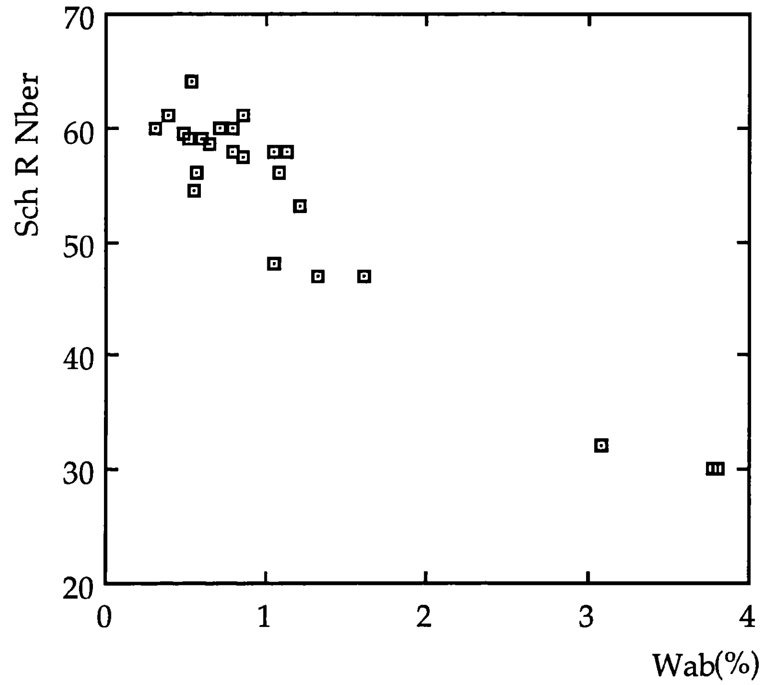


Fig 5. 20a The relationship between Schmidt Rebound Number and Water Absorption for quartz dolerite

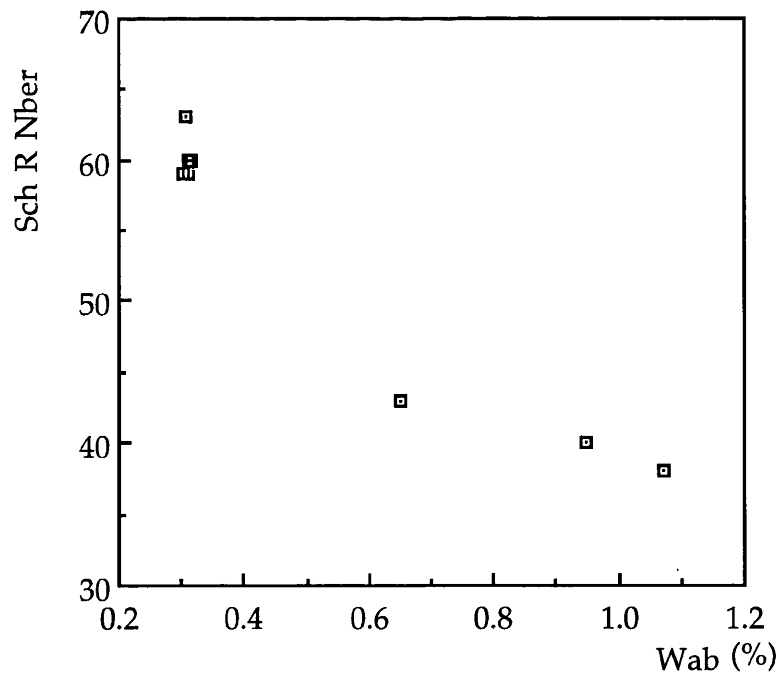


Fig 5. 20b The relationship between Schmidt Rebound Number and water absorption for granite

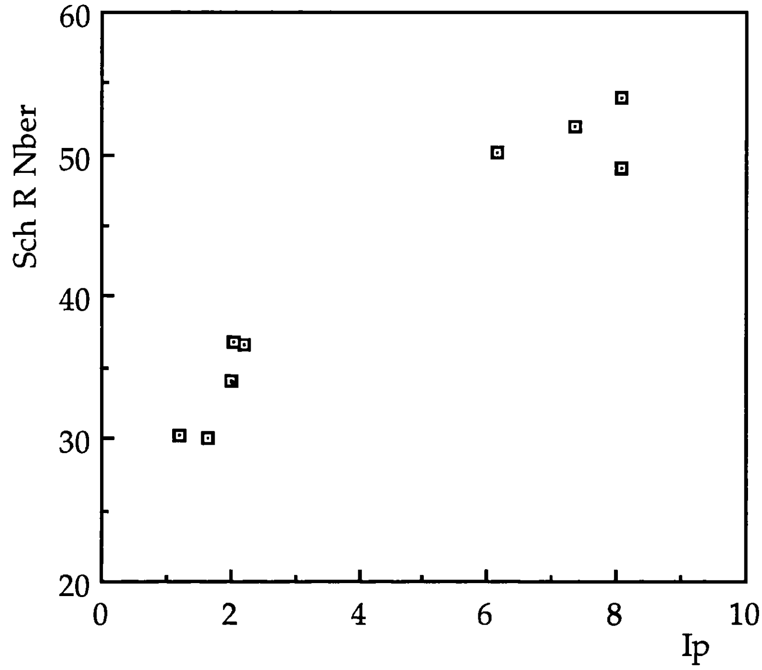


Fig 5. 21 The relationship between Schmidt Rebound Number and the Micropetrographic index for Beith basalt

From the present study it appears that the Schmidt Hammer is insensitive to change beyond value of 3 % for water absorption. The reason behind the variation in R that, in the fresh state, as the rock density increases it's elasticity increases resulting in high Rebound Number. When rocks undergo weathering, the development of secondary minerals and cracks and the weakening of intergranular bonds significantly reduces their elasticity.

Carter and Snedon (1977), El Jassar et al (1979) and Bastikan (1985) compared Rebound Numbers with Uniaxial Compressive Strength, and observed a linear relationship. Duncan (1969), however, demonstrated that R was linear up to a value of 50 for a range of rock types, but above this the relationship became non linear. Irfan and Dearman (1978) in a study of a variety of weathered granite from Cornwall showed that R exhibits a linear relationship with UCS at values above 40. Below this, the relationship becomes curvilinear. For the whole range of results of both these authors the use of a power law or logarithmic function provides the best curve-fit in this relationship.

In Fig(5. 24a, b) on samples with different grades of weathering. Schmidt Rebound Number appears to show a good linear relationship when plotted against the uniaxial compressive strength for the values up to SHV=40-50. At higher values the slope of the curve flattens suggesting a decrease in the sensitivity of the hammer as the rock becomes more elastic and fresher.

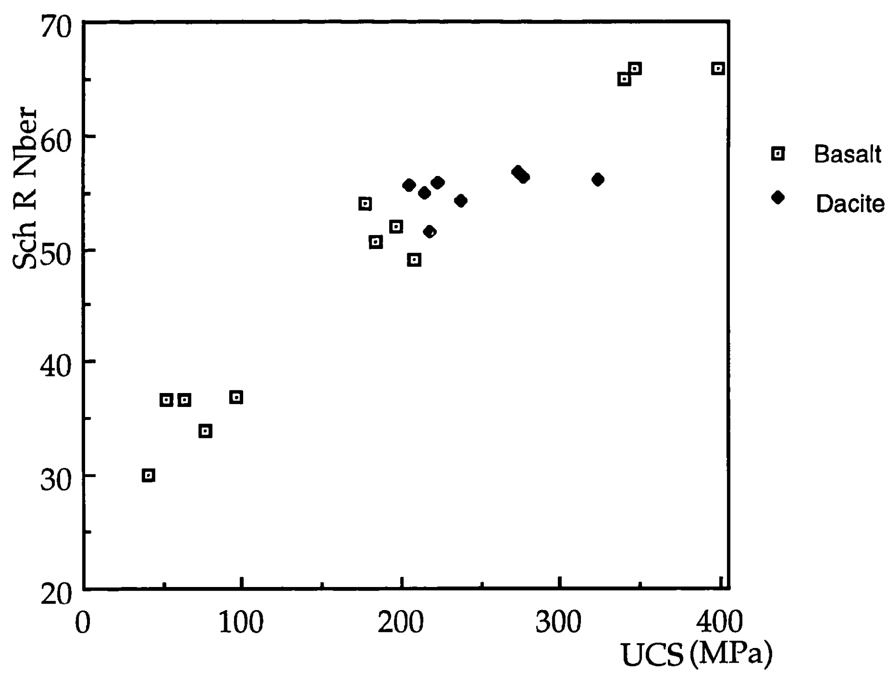


Fig 5. 22a The relationship between Schmidt Rebound Number and Uniaxial Compressive Strength for basalt and dacite

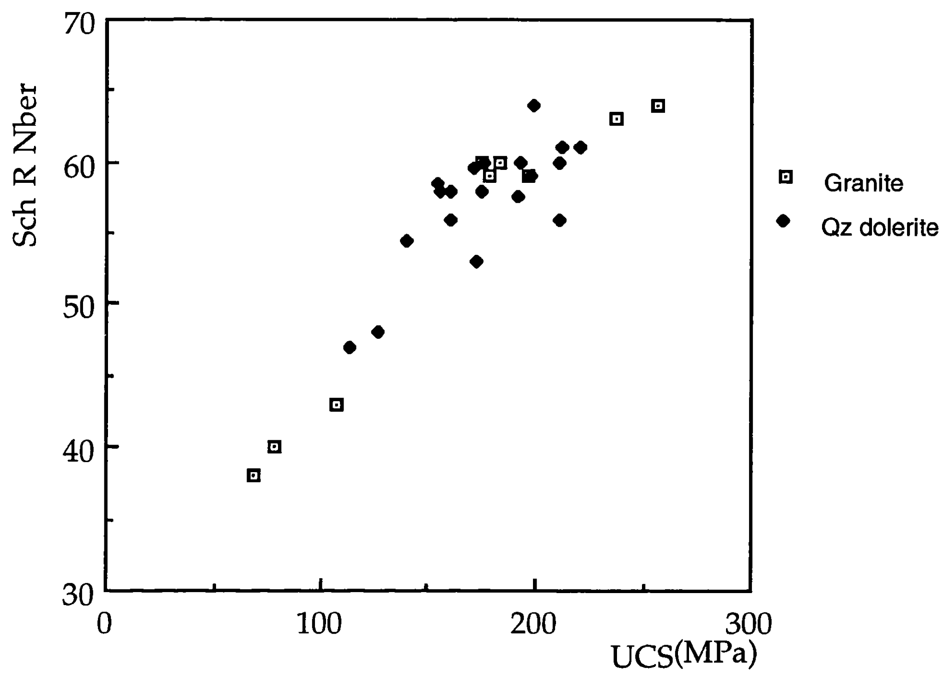


Fig 5. 22b The relationship between Schmidt Rebound Number and the Uniaxial Compressive strength for granite and quartz dolerite

Al Jasser et al (1979) in their study of the geotechnical properties of the carboniferous limestone of the Bristol area, found a good linear relationship between R and the Point Load Strength values both in the field and in the laboratory. In fact Al jasser et al (1979) were dealing with material having schmidt rebound number always above 40. In the present study, Fig (5. 25a, b) the relationship between Schmidt hammer and point load is best described using a logarithmic best fit curve of the type:

$$SRN = a + b \log Pls$$

a, b are constants

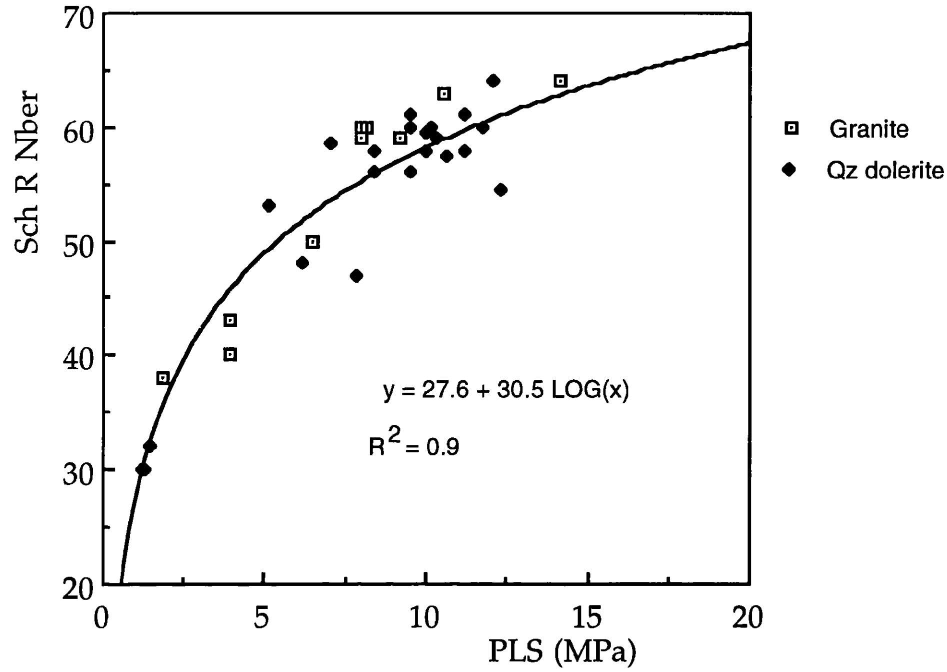


Fig 5. 23a The relationship between Schmidt Rebound Number and Point Load Strength for granite and quartz dolerite

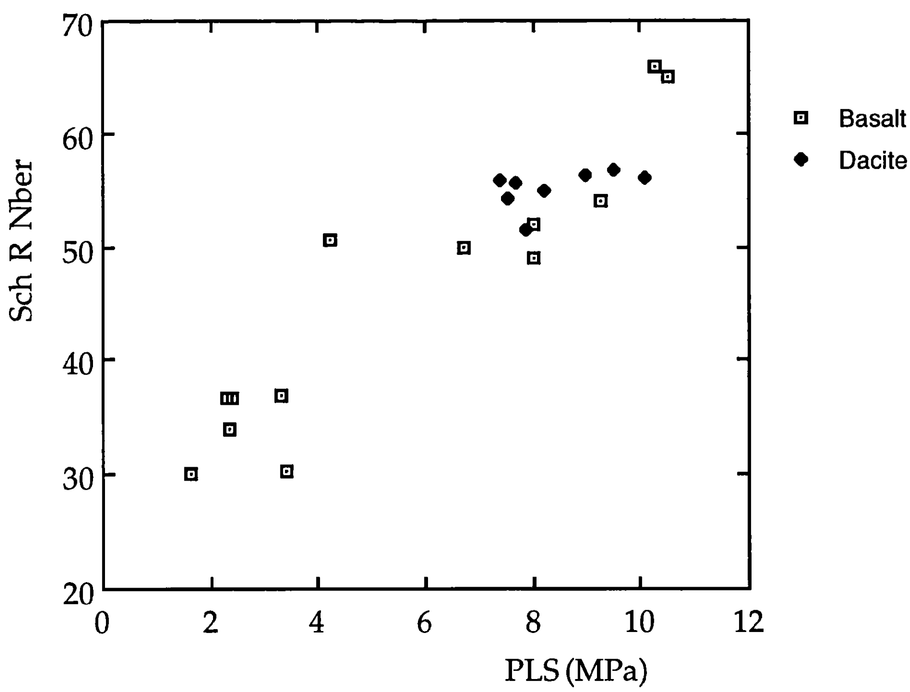


Fig 5. 23b The relationship between Schmidt Rebound Number and Point Load Strength for basalt and dacite

This logarithmic relationship between Rebound Number and Point Load Strength results from the fact that at high strength, the Schmidt Hammer becomes less sensitive to the strength variation of rocks while the sensitivity of the Pls is relatively not affected.

From the previous it appears that while Schmidt Hammer may not give a totally reliable estimate of the material strength it can differentiate between different rock groups in term of strength, elasticity, toughness, and especially freshness. In addition it has a limited use in the case of weak and very hard material. However, this may be overcome with the use of a type L Hammer in which energy input is significantly lower.

Ultrasonic Pulse Velocity

5 5 1- Introduction:

The measurement of elastic properties has often been used to provide information on the structural properties of rocks. These elastic properties are determined by either static or dynamic methods. In the latter case, seismic waves velocities are commonly measured using resonance or pulse methods.

Among the early works of note are the pioneering experiments of Bancroft (1940), Hughes et al (1950). Several other authors have used the technique to study the dynamic properties of rocks, such as Birch (1961), D'andrea et al (1964), Yeghishe and Leonard (1972), Youash (1970). For concrete, where the technique is routinely used, the first known report on the measurement of the ultrasonic velocity was by Obert (1940). Later Long et al (1945) developed the first modern type of apparatus to measure the ultrasonic pulse velocity in concrete slabs. Several other authors such as Andersen et al (1952), Kesler and Chang (1957) have studied the dynamic properties of concrete using the ultrasonic pulse velocity.

The ultrasonic pulse method, used in the present study, consisted of the determination of the travel time of elastic waves in rock cylinders of known length. This travel time is then used to calculate the velocity using the formula:

$$v = \frac{l}{t}$$

v= velocity of the elastic waves in m/s

l= length of the cylinder specimen in meter (m)

t= elastic wave travel time in second (s)

The dynamic Modulus of Elasticity was then calculated using the expression proposed by Jaeger (1969).

$$Edy = \rho v^2$$

ρ material density

v ultrasonic wave velocity through the rock

The ultrasonic velocity through the specimen is sensitive to the mineralogical assemblage, and is affected by the shapes, distribution, and preferred crystallographic orientations of the rock components. Moreover, it is affected to a great extent by the presence, size and orientation of defects such as pores and cracks and the presence or absence of a fluid phase.

4 5 2- **Factor affecting Ultrasonic Velocity**:

a- Mineralogy:

In general terms, compact and dense rocks have higher elastic wave velocities than less dense rocks even if they are compact(Table 5. 6). The fact is that velocity increases as the mean atomic weight of the material increases, i.e. basalt and gabbro which contain dense ferromagnesian minerals exhibit higher velocities than granite and rhyolite. In this study dacite, although it has higher strength than quartz dolerite exhibits lower velocity than dolerite due to it's lower density. In granite the quartz content was found to greatly influence the velocity, for instance the Dunecht granite although it has higher strength than Dalbeattie it exhibits lower velocity, because of it's higher quartz content. The quartz content of Dunecht and Dalbeattie granite is about 35 and 25 % respectively. Ramana and Venkatanarayana (1973) studied the effect of mineralogical composition on longitudinal wave velocity for Kolar rocks and found that velocity increases with the hornblende content of the rock, while an inverse relationship was obtained for quartz. This is true under high pressure conditions. Under atmospheric pressure, since the velocity of intact rock is not just dependent on the rock type rather than texture, porosity, density, presence or absence of fluid, and the degree of fracturation, considerable overlapping in the range within which rock type velocities vary, usually occurs.

Table 5. 6. Typical values of longitudinal velocity for rocks (after Formiantreau 1976)

Rock type	Velocity (m/s)
Gabbro	7000
Basalt	6500 - 7000
Granite	5500 - 6000
Limestone	6000 - 6500
Dolomite	6500 - 7000
Sandstone quartzite	6000

b- Texture:

Elastic wave velocity in compact, non-fractured rocks is related to the velocity of it's mineral components. If the mineral grains are randomly oriented the rock velocity is usually the average velocity of it's constituent minerals (Brace 1965), and can be calculated using Birch (1961) formula:

$$V = \frac{1}{\sum_{i=1}^{n} \frac{Xi}{Vi}}$$

$V =$ Velocity in the rock

$Xi =$ Proportion by volume of the ith mineral.

$Vi =$ Velocity of the ith mineral.

Preferred orientation of minerals in the rock usually give different velocities in different directions. In amphibolite and dunite the directions of high velocity were found to coincide with directions of preferred orientation of the high velocity directions in hornblende and olivine respectively (Brace 1965). Grain boundaries also influence the velocity, being greater in finer grained rocks than in coarse grained ones (Lama and Vutukuri 1974).

c- Density:

A principal factor determinng the propagation velocity of elastic waves is the density. Birch (1961) showed that the velocity increases linearly as the material density increases. Youash (1970) also showed that the dynamic modulus of elasticity, computed by ultrasonic pulse method, increases linearly as the material density increases. Similar results were obtained in this study on different weathering grades (Fig 5. 24). The plot of velocity versus density for different type of rock shows clearly the velocity dependence on the material density. In Table (5. 7), the results were obtained from fresh cores of granite, dacite, quartz dolerite, and basalt and demonstrate clearly the dependence of velocity on the material density.

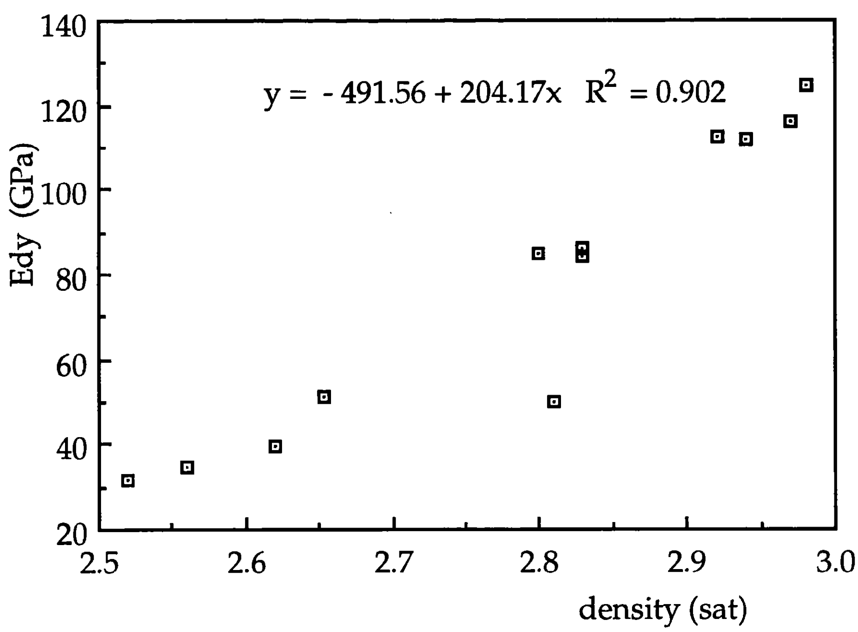


Fig 5. 24 The relationship between dynamic modulus of elasticity and density for basalt

d- Porosity:

In general, longitudinal wave velocity is influenced considerably by the rock porosity. However, there are two different type of porosity, intergranular porosity and intragranular porosity. The former type of porosity is very often due to fracturing or dissolution of minerals at their grain boundary, due to weathering, leaving intergranular spaces. The latter usually results from vugs or kernel alteration of minerals. The influence of intergranular porosity on the elastic wave velocity is much greater than that of intragranular porosity. Secondary porosity in dolomite has no effect on the velocity (Lama and Vutukuri 1974), while the velocity is considerably reduced in a fractured medium (Paterson 1978). The influence of porosity on the velocity of elastic waves stems from the fact that when waves propagate in solid rock at a given velocity, an open crack or void in it's path causes delay in the transit time of the waves, which will be reflected in a lower velocity for the whole specimen.

e- Water content:

The presence of a fluid phase in the rock porespaces leads to a change in the longitudinal elastic wave velocity. In the dry state the porespaces and cracks are filled with air with a velocity of 100 m/s. Having in mind that elastic waves in a rock travel through minerals and pores their transit time will then be:

$$t = t_m + tp$$

and their velocity will be:

$$1/v = n/v_p + (1-n)/v_m$$

t elastic wave transit time in the rock

tm elastic wave transit time in minerals

tp elastic wave transit time in the porespaces v Velocity of the waves through the rock

vp Velocity of the waves through the pores

v m Velocity of the waves through minerals

n porosity

When wet, pores and cracks will be filled with water with a velocity five times greater than the velocity of air. Therefore, the longitudinal wave velocity in the rock will increase as a result of saturation. Moreover, the higher the velocity of the porefilling material the greater the velocity of the rock. Lama and Vutukuri (1974) demonstrated that rock velocity increases as the amount of water in the porespaces increases, until the rock is fully saturated.

5 5 3- **Results and discussion:**

In this study an ELE ultrasonic testing apparatus called PUNDIT (portable ultrasonic non destructive indicating digital tester) was used. The whole system consists of a pulse generator and timing unit connected by two cables to two transducers placed at each end of the rock core to be tested. The transducers must be in perfect contact with the perfectly flat and grease smeared rock core ends. The pulses generated are converted in the transmitting transducer to a mechanical motion which travels trough the core to the receiving transducer where it will be converted to an electrical signal. The transit time through the sample, or from one transducer to the other, is then displayed on an electronic display. The velocity then is calculated as shown previously.

Representative cores 4 inches long by 2 inches diameter with perfectly flat ends were prepared from all the rock types studied, using a precision grinding machine, then tested at room temperature conditions. It has been observed that every rock type displays a range of velocities, e.g. the velocity in the basalt ranged from 3150 to 6292 m/s, in the quartz dolerite 1416 to 6300 m/s, 4660 to 5400 m/s for dacite, and 2700 to 5400 m/s for granite Table (5. 7). These intervals of variation and overlapping are influenced by several factors such as porosity, density, degree of weathering (secondary mineral content), texture and grain size. Comparison between the studied rock types in their fresh state indicates that density is the major variable upon which the velocity depends. Basalt and dolerite having higher densities exhibit higher ultrasonic velocities. Selected cores of quartz dolerite and fresh basalt with densities of 2.88 to 2.91 and > 2.92 display velocities of 5700 to 6292 m/s and 6200 to 6300 m/s respectively. Fresh granite and dacite, on the other hand, with densities in the range of 2.64 - 2.67 and 2.48 - 2.54 exhibit longitudinal wave velocities of 4400 to 5300 m/s and 4600 to 5400 m/s respectively. The density-velocity relationship Fig (5.25) indicates that as the density increases, the rock becomes fresher, the velocity subsequently increases. This means a decrease in the amount of secondary minerals and especially structural discontinuities, i.e. cracks and voids, allows the ultrasonic elastic waves to travel faster through the rock. Thus in comparing samples of the same rock type Vp is an index property for the rock state, which can be used in conjunction with other indicators, e.g. porosity, specific gravity, water absorption etc (appendix 5).

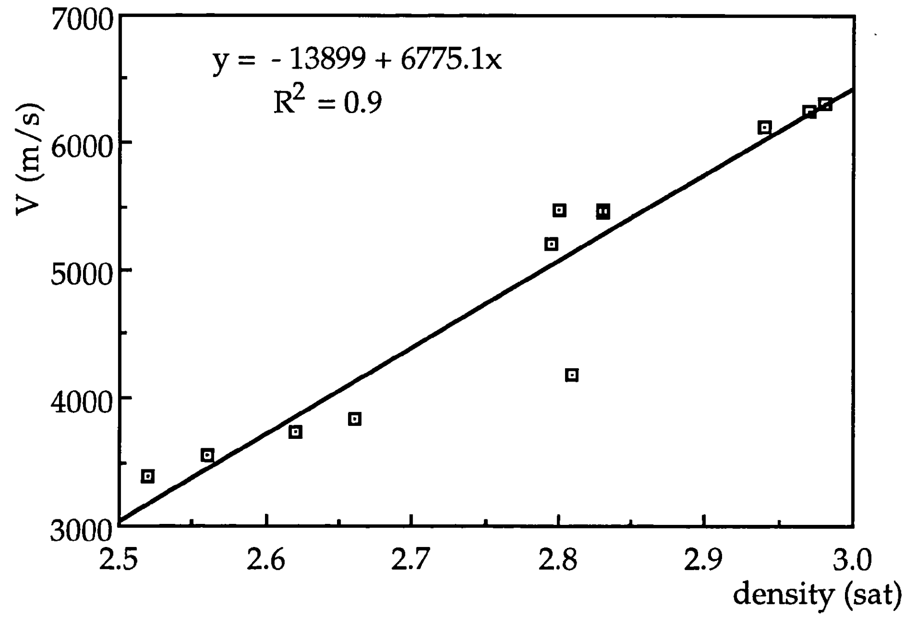


Fig 5. 25 Graph showing the velocity-density relationship for basalt

Duncan (1969), and Irfan and Dearman (1978) and turk and Dearman (1986) have all studied the relationship between density and velocity within one rock type, and have shown that as the rock density falls the velocity of the rock also decreases linearly. For instance, fresh basalt with a density of > 2.9 exhibits a velocity of > 6000 m/s while a heavily weathered one has a density of 2.52 - 2.56 and a velocity of 3300 - 3600 m/s. For other rock types see (Table 4. 7). The other factors to which variation in the longitudinal wave velocity are related is porosity and water absorption. Several authors such as Iliev (1965), Duncan (1969), Onodora (1974),and Turk and Dearman (1986) found a good relationship between the longitudinal wave velocity and both porosity and water absorption. For the rocks studied, the author has confirmed that longitudinal wave velocity decreases linearly with increase in porosity (Fig 5. 26). Exactly the same trend of relationship was obtained with the water absorption (Fig 5. 27).

Secondary minerals development and cracks also reduce significantly the elastic wave velocity through the rock and thereby the dynamic modulus of elasticity (Fig 5. 28, 29).

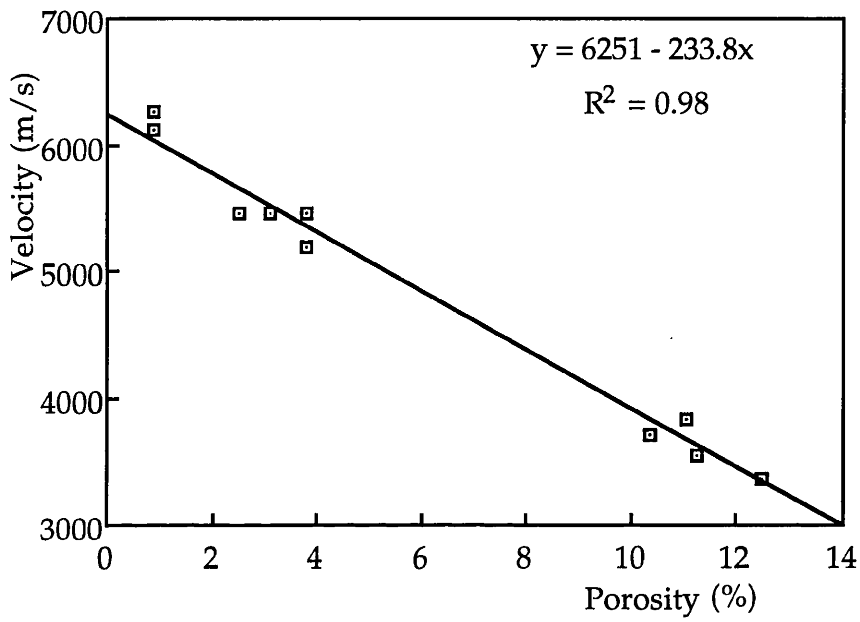


Fig 5. 26 The relationship between velocity and porosity for basalt

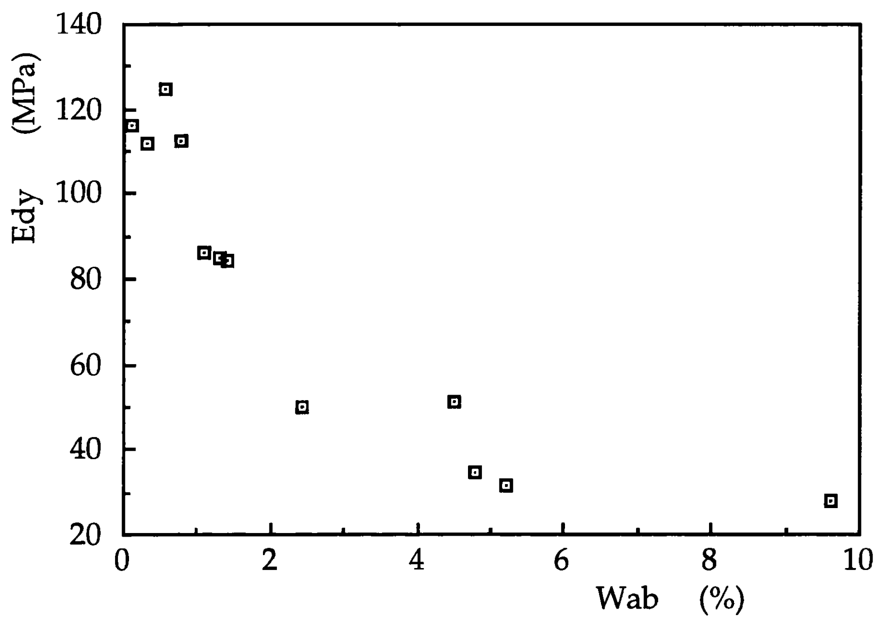


Fig 5. 27 The rrelationship between material water content and it's Dynamic Modulus of Elasticity for basalt

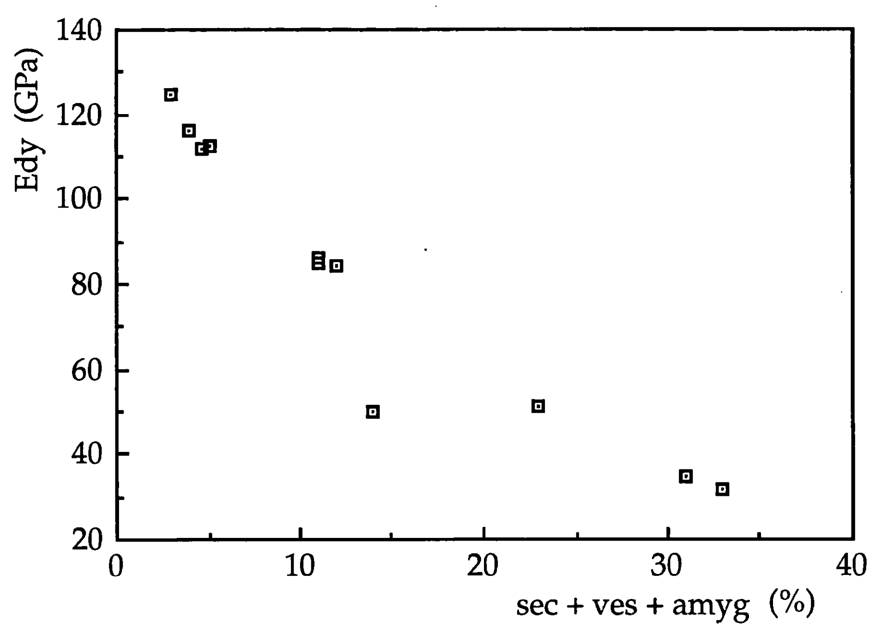


Fig 5. 28 The influence of secondary constituents on the Dynamic Modulus of Elasticity for basalt

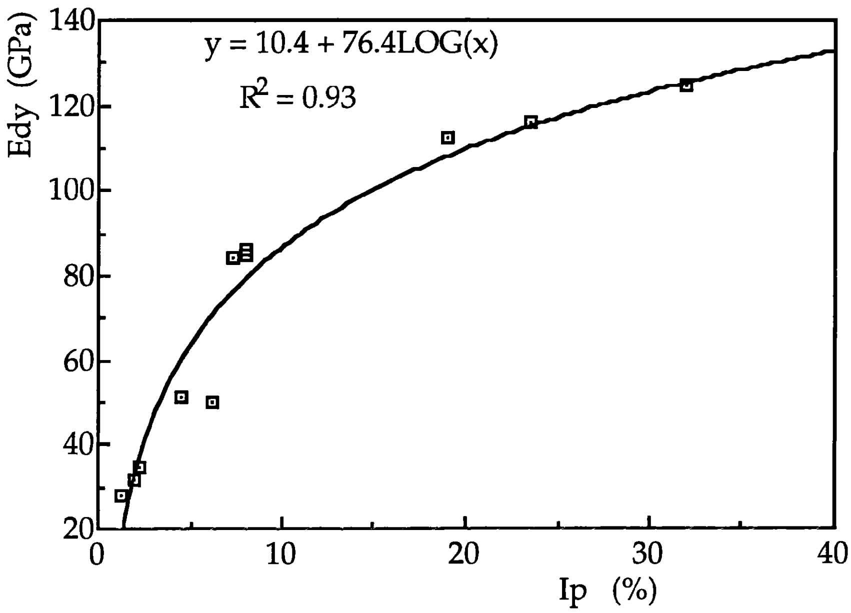


Fig 5. 29 The relationship between the Modulus of Elasticity and Ip for basalt

Table 5. 7 The effect of water saturation on the ultrasonic velocity of rocks.

Rock sample	Velocity (m/s)	Velocity (sat) (m/s)	Density
4N	3790	3807	2.62
1N	4008	4138	2.81
2N	3423	3094	-
1S	3795	4019	2.56
2S	3369	3531	2.52
8N	5745	5674	2.83
O4	6253	6253	2.97
Ln4	6445	6306	2.98
Ba2	4518	4481	2.51
Ba4	4528	4552	2.50
Ba8	4659	4712	2.47
Ba9	4541	4588	2.47
Gd1	5299	5145 (oven dry)	2.67
Gd2	4137	-	2.54
Gd5	4974	4595 (oven dry)	2.61
Gd6	2893	2452 (oven dry)	2.59
Gd7	4410	4419 (oven dry)	2.64
Gd8	2738	4358 (oven dry)	2.59
Gd9	4399	4358 (oven dry)	2.64
STr1	5028	5385	2.90
Str2	5226	5860	2.94
Str3	5314	5282	2.92
St19	6292	-	2.91
St11	6111	-	2.83
Strw	4980	4901	2.69

On saturation the ultrasonic wave velocity has been observed to increase, especially for materials of higher porosity and water absorption. The fact is that highly porous samples have greater void content which causes a delay in the elastic waves transit time. When saturated these voids are filled with relatively higher velocity material (water) which diminishes at least five times the previous delays and results in higher wave velocity. Although, this is the general behaviour, instances occured where a decrease in velocity after saturation was observed, mainly in low porosity rocks.

The correlations between the ultrasonic velocity and the engineering indices such as UCS, PLS, and R usually give a relationship suggesting that higher velocity equates with higher strength. The relationship between the UCS, PLS, R and the Ultrasonic Velocity, when the parameters affecting each group are constrained, is linear. For the UCS (Fig 5. 30), it is clearly shown that as the UCS increases the Velocity also increases in a linear manner. The position of the quartz dolerite on the previous graph is mainly due to the influence of it's grain size on the UCS on one hand, and density on it's velocity. The point Load Strength when correlated with the Ultrasonic velocity exhibits a relationship similar to the UCS vs Ultrasonic velocity (Appendix V). A similar correlation was obtained with the R (Appendix V).

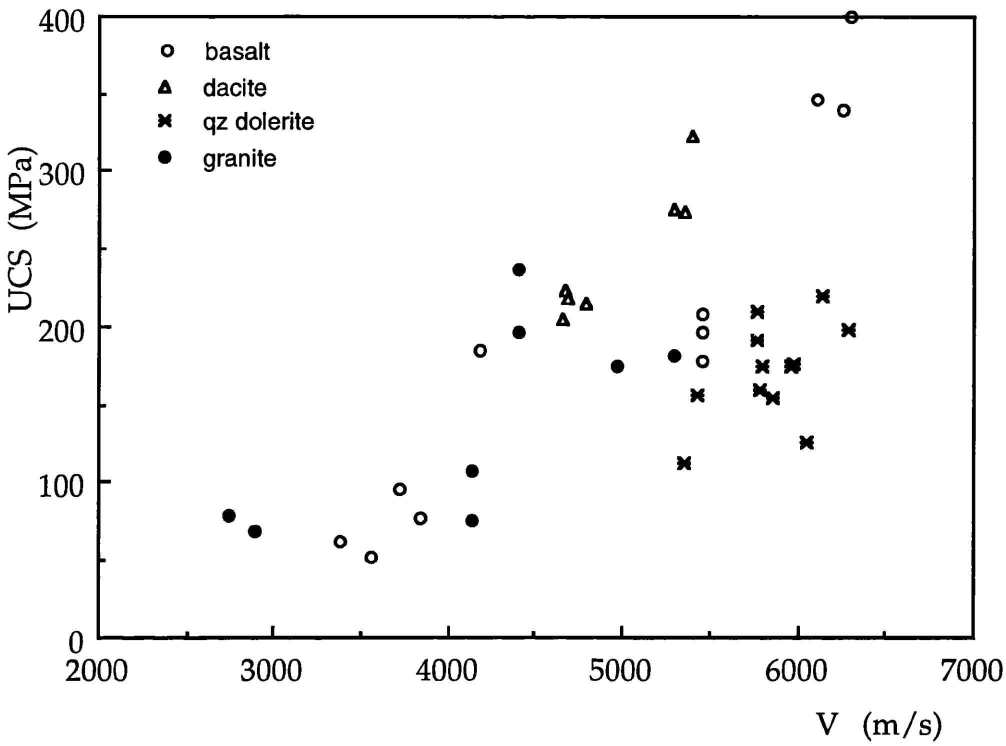


Fig 5. 30 The relationship between uniaxial compressive strength and ultrasonic velocity for all the rock studied

CHAPTER VI

Engineering Properties of the Studied Crushed Rock Aggregate

6 1- Introduction

Aggregate refers to natural or crushed rock particles which when brought together in bound or unbound conditions form part or whole of a building or civil engineering structure. Aggregates are used in varied and widespread environments ranging from highway and airport pavements, dams, buildings, and tunnel linings to railways and filter media...etc. They constitute about 60 to 80 percent by volume of concrete and bitumen mixes. Their influence, therefore, on the performance of the structures of which they are part is beyond any doubt. In these circumstances they are required to be hard, durable, clean, and free from any harmful substances that might have an adverse effect on the strength and durability of concrete and bitumen mixes (Collis and Fox 1985).

Particular aggregate properties have greater significance in some contexts than others, and some of these desirable qualities may be partially or completely conflicting, e.g. hard minerals are frequently brittle, hence high resistance to abrasion may be accompanied by low resistance to impact and crushing e.g. quartz, flint, and chert. Skid and abrasion resistance are also conflicting aggregate properties. Hard material although durable wears uniformly and slowly in consequence of which a highly polished surface will result. In this context Lees (1975) stated that the term quality, when used in respect of aggregates, is relative to the usage to which the material is to be put. There is no such a thing as a "high quality roadstone" until one has specified the context of it's use.

In Britain the selection of aggregates is generally based on some standard physical, mechanical, and chemical tests in addition to experience and reputation of the aggregate materials and their performance. Toughness and strength of aggregate is mainly gauged by the Aggregate Impact and Crushing Tests and the Ten Percent Fine Value (BSI 882), These can be suplemented by some non-standard values, i.e Aggregate Impact and Crushing Residue Values Ramsay (1965) and Dhir et al (1971), Modified Aggregate Impact Value (Hosking and Tubey 1969).

Hardness and durability are evaluated by the Aggregate Abrasion Value (AAV), while soundness is assessed by a chemical index test such as the Magnesium Sulphate Soundness (BSI 812. 1975). Other foreign standard aggregate tests include the Los Angeles Abrasion Test (ASTM C131. 1976) and the Methylene Dye Absorption (Tran Ngoc Lan 1980). Physical characteristics such as density, porosity, and water absorption (BSI 812. 1975) are also measured, on account of their use as indicators of material soundness and their frequent use in concrete mix design.

The variability of the factors which control the mechanical performance of an engineering material makes "in service" prediction of the material very difficult and some times practically impossible. The only variable that can be indicated with any accuracy is the rock quality. However Cox (1973) summarised the importance of aggregate and rock testing as follows:

1- To asses the quality usefulness or otherwise of the new source of stone

2- To ascertain whether the type of stone from a given source is changing significantly

3- To compare samples from a given source to ascertain that the quality remains relatively constant.

4- To compare the quality of stones from different sources.

5- To ascertain whether the characteristics of the stone satisfy specification requirements

In the present study factors affecting the strength, hardness, and soundness of a suite of widely used crushed rock aggregate has been investigated through the use of index properties such as AIV, ACV, AAV, MAIV, 10% fines value, Magnesium Sulphate Soundness, density, water absorption (BS 812) and non standard values such as AIVR and ACVR. The Los Angeles Abrasion Value (ASTM C131) and a non standard Los Angeles Residue Value have also been incorporated. In addition to the properties of fresh aggregate, the effects of weathering and alteration have also been investigated.

6 2- **Aggregate properties**:

6 2 1- physical properties

a- Relative Density and Water Absorption:

The Relative Density is defined as the ratio of the mass of material to the mass of an equal volume of water. The specific gravity of a rock depends largely on the unit mass of the constituent grains, freshness, porosity and the pore water content.

Water absorption is the ratio of water absorbed by the rock to the mass of the rock. It is an indirect measure of the hydraulic conductivity, reflecting the presence of an interconnecting network of secondary minerals and microfractures which are especially common in weathered rock material

(Fookes et al. 1988). This value has a significant influence on other physical characteristics such as mechanical strength, shrinkage, soundness and general durability (Fookes et al 1980, Collis and Fox. 1985 and Clifford 1991).

Relative Density and water absorption together have been found to be useful indicators of material quality (Goureley. 1986, Gribble. 1990, Fookes et al. 1988, Cawsey and Massey 1988). Smith et al (1970) have successfully used water absorption in conjunction with other index properties to derive a rock durability indicator for slope protecting rip rap, which they called the Durability Absorption Ratio (DAR).

DAR = durability index/1+water absorption

where durability index is a wet abrasion test. Later Fookes et al (1988) incorporated specific gravity and water absorption into two rock durability indicators, one static and the other dynamic.

$$RDIs = \frac{Is(50) \ - 0.1(SST + 5\,WA)}{SGssd}$$

$$RDId = \frac{0.1(MAIV + 5WA)}{SGssd}$$

Where RDIs the static rock durability indicator

RDId	the dynamic rock durability indicator
SST	the Magnesium Sulphate Soundness
SGssd	the Relative Density (saturated and surface dry)
$Is_{(50)}$	Point Load Strength
MAIV	Modified Aggregate Impact Value
W A	Water Absorption

b- Results and discussion:

Specific gravity or relative density and water absorption were carried out according to the test methods outlined in BS 812 (1975). The results obtained were used to calculate four physical quantities, apparent relative density, relative density on oven dry basis, relative density on saturated and surface dry basis, and water absorption as follows.

Apparent relative density = $\dfrac{D}{D - B}$

Relative density (on oven dry basis) = $\dfrac{D}{A - B}$

Relative density (on saturated and surface dry basis) = $\dfrac{A}{A - B}$

Water absorption = $\dfrac{A - D}{D} \times 100$

Where A is the mass of saturated surface-dry sample in air (g)

B is the mass of sample suspanded in water (g)

D is the mass of the sample oven dry (g)

In this investigation porosity, relative density, and water absorption have been determined for all the different rock types of the suite at different grades of weathering (Table 6. 1, 2, 3). For all of the rock types, high relative density and low water absorption and porosity are characteristic of fresh material, while the opposite is typical for weathered samples, e.g. weathered basalt displayed a relatively low relative density of 2.52, high water absorption and porosity of 4 - 6 and 11 - 13.5 respectively, while fresh basalt has a relative density as high as 2.97 and water absorption and porosity of 0.31 and 0.9 respectively. The other rock types quartz dolerite and granite displayed similar pattern in physical properties.

Table 6. 1 Some aggregate index values for quartz dolerite

Sample	AIV	AIVR	AAV	Wab	Specific G	Grain size
Sz2	10.5	53	8.57	1.32	2.86	1.51
St2	9	54	-	1.2	-	1.21
St3	8	57	4.84	0.98	2.87	-
St4	12.46	46	4.18	1.2	2.66	2.77
St6	9.46		5.56	0.85	2.87	-
St8	8.75	58	-	0.51	2.91	-
St11	6.5	64	-	0.58	2.83	0.41
St12	8.7	51.9	-	0.79	2.88	1.11
St13	10.69	48	2.85	0.56	-	-
St16	10.46	55.87	-	5.93	0.85	1.07
St18	8.25	60	3.95	1.05	2.86	0.92
St19	7	56	2.98	0.53	2.86	0.75
St22	11.16	52	-	0.71		1.55
St24	11	44	3.78	1.08	2.79	1.66
St25	12.5	40	3.36	0.82	2.73	-
St26 up	6.4	56	4	0.38	2.86	0.65
St261	11.62	49	6.25	0.65	2.86	2.4
St31	20.1	29.13	8.36	3.09	2.58	-
St32	22	32	12.5	3.77	2.45	-

All samples are tested at flakiness index = 0

Table 6. 2 Some aggregate index values for basalt

Sample	AIV	AIVR	MAIV	AAV	Wab	Specific G
1N	8	57.5	12.6	5.31	2.42	2.81
2N	18.5	32	29	25.4	9.45	2.27
3Nlw	7	62	9.22	4.4	1.46	2.83
6N	7.5	56	10.4	-	1.35	2.8
8N	8	59	10.5	4.54	1.1	2.83
1E	19	31	29.5	22.75	5.53	2.5
1S	14.17	43.5	28	-	4.8	2.56
2S	15.5	43	28	-	5.21	2.52
3S	11	48	17.5	11.37	4.43	2.53
O1	6 - 7	58 -59	7	2.18	0.31	2.94
O4	6 - 7	58 - 59	7	2.57	0.3	2.95
Ln4	5.5	65.4	-	2.26	0.56	2.97

All samples are tested at flakiness index = 0

Table 6.3. Aggregate strength index values on quarry crushed aggregates

Rock type	AIV	AIVR	ACV	ACVR	LAAV	LAAVR	AAV	10 % fines (kN)
Basalt B	15	16	23.5-24.5	31 - 32.3	19.6	40.25	7.25	150 - 170
Basalt L	5.5	71	10.43	40.64	9.55	55.64	2.4	380
Basalt O	6	59	12.44	38.7	10.5	59	2.67	-
Qz dolerite	7 - 8	58 - 63	14 - 15.5	37 - 38	11.67	59	4.53	260
Granite N	14	28 - 31	22.2	31.35	26.53	28.35	3.6	180
Aplite N	8	66	13	-	10.95	52.6	-	-
Dacite	8.8	50.23	12.8	36.7	9.92	53.9	3	-

B = Beith, L = Langside, O = Orrock, N = Nairn

All aggregates are tested at flakiness index = 0

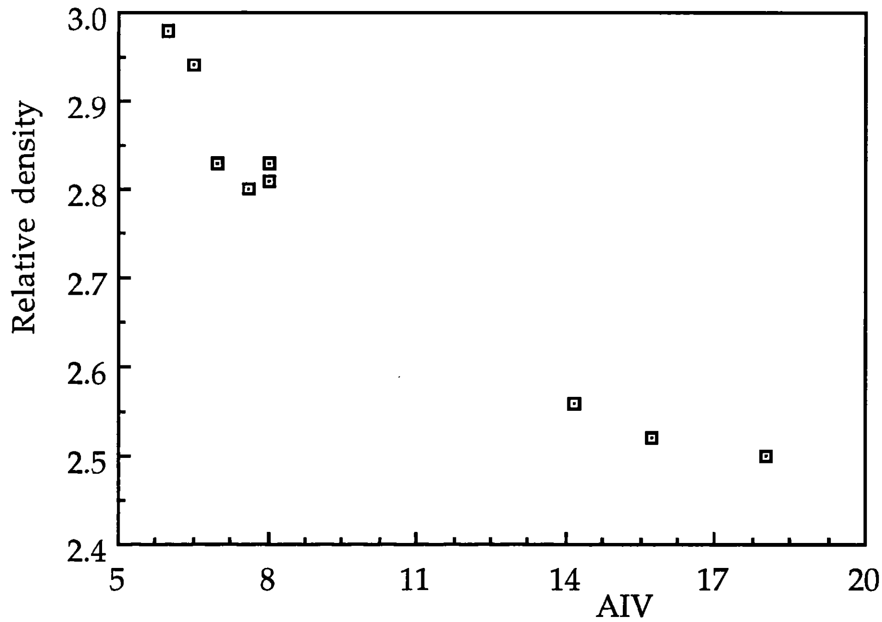


Fig 6.1 The relationship between Aggregate Impact Value and the relative density on saturated basis for basalt

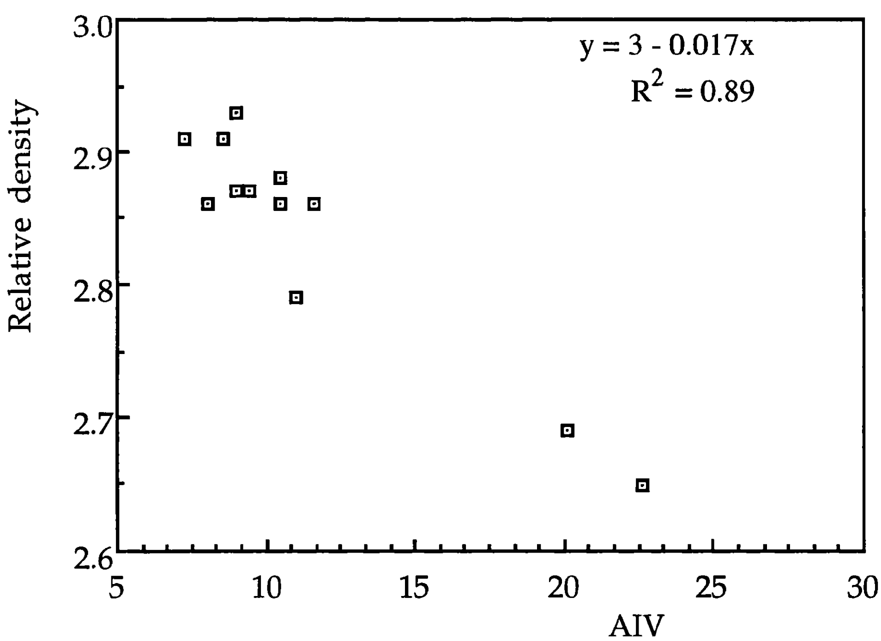


Fig 6.2 Relationship between Aggregate Impact Value and the Relative density on saturated basis for quartz dolerite

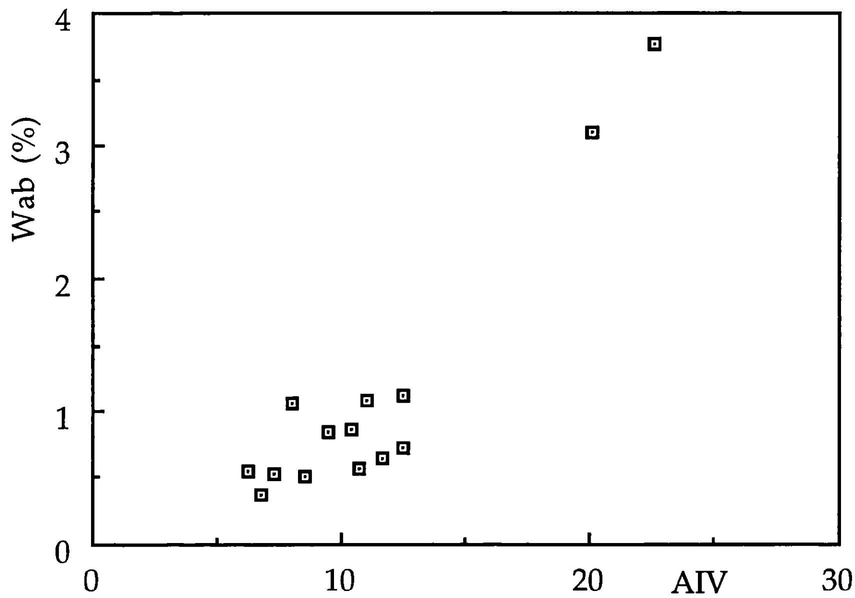


Fig 6.3 The relationship between AIV and water absorption for quartz dolerite

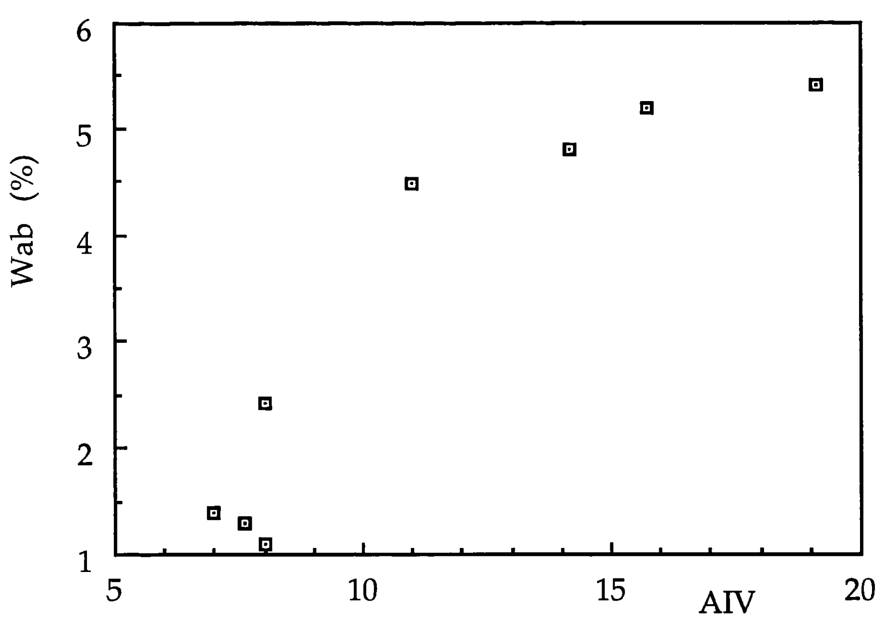


Fig 6. 4 The relationship between AIV and water absorption for basalt

6 2 2- Mechanical properties:

a- Strength:

The importance of aggregate strength in the selection process comes from the fact that in-service they are subjected to different kinds of stress. During the laying of pavements the aggregate comes under a heavy crushing stress applied by the rollers, and later in-service they experience impact and abrasion stresses from the traffic. The magnitude of the impact forces applied by a moving vehicle is of the order of 25% of it's static weight (Krynine and Judd 1957). The aggregate is also required to carry and distribute the load over the entire pavement and transmit it to the underlying subgrade. In order to withstand the rigours of service without pulverisation or breakdown, aggregate must have adequate strength. As mentioned earlier, aggregate strength tests used in British practice are, Aggregate Impact Test (AIV), Aggregate Crushing Test (ACV), and theTen Percent Fines. The Los Angeles Abrasion Test because it involves a high conponent of impact can also be used.

a1- Aggregate Impact Value:

The Aggregate Impact Test is designed to simulate aggregate behaviour under conditions where repeated impact loading predominates, i.e, roads and railways. It gives, therefore, an insight into the resistance of aggregate to pulverisation under impact loading conditions. The apparatus comprises a steel base plate, two vertical guide runners and a cylindrical hammer whose fall is controlled by runners. The hammer is raised to a cross-bar and releases by a trigger mechanism to fall on the sample from a height of 381± 6mm. The standard test requires 15 blows delivered at intervals of not less than 1 second.

The broken sample is then sieved for at least 1 minute and the fraction passing 2.36 mm sieve is weighed and expressed as a percentage of the initial sample weight. This is the Aggregate Impact Value (AIV):

$$AIV = \frac{\text{weight of the fraction passing 2.36 mm}}{\text{weight of the original sample}} \times 100$$

In the interest of repeatability and reproducibility, the AIV should be the mean of two tests differing from each other by no more than 1%.

Tough and high quality aggregate should exhibit a high resistance to pulverisation reflected in a lower values of AIV. For satisfactory performance, however, the aggregate should resist breakage of any kind not just pulverisation to fines less than 2.36 mm. Ramsay (1965) indicated that the choice of the 2.36 mm as the cut off level for fines and the critical measure of aggregate toughness was an arbitrarily chosen value. The standard AIV takes no account of the coarser cataclastic products which are produced. A more realistic measure of aggregate toughness and resistance to breaking is the amount of material remaining in the original size range (14 - 10) after the test. This value, Aggregate Impact Value Residue (AIVR) can be obtained by little extra effort and should be quoted along with AIV. e.g.

$$AIVR = \frac{\text{weight of the fraction retained on 10 mm sieve}}{\text{weight of the original sample}} \times 100$$

a 2- Aggregate Crushing Value:

The Aggregate Crushing Test (BS 812) measures aggregate strength as a response to a steadily applied load. Unlike the impact test it is designed to simulate the behaviour of the granular material under conditions of steady loading. It is another measure of the aggregate resistance to pulverisation and reflects the toughness, tenacity and ability to distribute and transmit the load over the entire sample. In this test a given amount 14 - 10 mm nominal size aggregate is placed in a removable base steel cylinder, and a metal piston is placed on the top of the sample and the whole assembly is then placed in a compression machine. The piston is loaded steadily on the aggregate sample at a rate of 40 kN/min for ten minutes achieving a total load of 400 kN. After the test, the weight of the material passing 2.36 mm sieve is expressed as a percentage of the initial sample weight. This dimensionless value is the Aggregate Crushing Value (ACV):

$$ACV = \frac{\text{weight of the fraction passing 2.36 mm}}{\text{weight of the original sample}} \times 100$$

High resistance to pulverisation in this test yields a low ACV and indicates that the material has a high capacity for distributing and transmitting the load over the whole sample. Dhir et al (1971) proposed a non-standard Aggregate Crushing Residue Value (ACVR) which, as in the Impact Test, is a measure of the material remaining within the original size range after testing. The ACVR was proposed to give a real figure of the material which which sustain the loading without damage. This non-standard value is calculated as follows:

$$ACVR = \frac{\text{weight of the fraction retained on 10 mm sieve}}{\text{weight of the original sample}} \times 100$$

c- Factors Affecting Aggregate Strength:

c 1- Technical factors:

Two procedural factors have been identified which could affect the repeatability and reproducibility in the Aggregate Impact Test, namely the rigidity of the apparatus and the nature of the test floor (Ramsay et al 1973, 74, 77).

The impact apparatus must remain rigid throughout the test, as any loosening of upright supports retards the fall of the hammer and reduces the kinetic energy of the system. This fault is most likely to develop in the test machines whose guide runners are screwed into or bolted through the base-plate. The lubrication and cleanliness of the guide runners also affects the fall of the hammer.

The nature of the floor upon which the machine stands during the test significantly influences the impact value obtained. Ramsay et al (1973) have shown that tests on material from the same source performed on concrete floors of different thickness and composition yielded different and unpredictable Impact Values. The wide spread of values was considerably reduced when the apparatus was mounted on a wooden block. In the present programme the apparatus was mounted on a wooden block.

c 2- Geological Factors:

1- Aggregate shape:

In the British Standard specification the shape of aggregate particles is grouped into four categories, equidimensional or cuboidal, elongate, flaky, and flaky elongate. A flaky particle is one whose smallest dimension is less than 0.6 times the mean sieve size. An elongate particle is one whose maximum dimension is greater than 1.8 times the mean sieve size (BS 812 1975). The proportion of each shape fraction in the crushed rock aggregate is a function of intrinsic geological features like brittlness and fabric, crusher type and reduction ratio. Strongly anisotropic metamorphic rocks yield aggregate with a high proportion of flaky particles, while igneous rocks with the homogeneous isotropic fabric yield a lower percentage of flakes. Fine grained igneous rocks are stronger and more brittle than coarse grained and usually yield a higher flaky content. The weight percentage of flaky particles in an aggregate sample is called the flakiness index (I_F), and the percentage of elongate particles is called the elongation index (I_E).

The effect of particle shape on aggregate strength has been extensively studied by (Ramsay. 1965, Dhir et al. 1971, Ramsay et al. 1974, Spence et al 1974, and Spence 1979). Ramsay (1965) demonstrated that AIV increases linearly as the flakiness index increases.

$$AIV= C + nIf$$

C = AIV at I_F = 0, or the petrographic constant

n = regression coefficient

Dhir et al (1971) studied the ACV and found that a similar relationship obtains, i.e. ACV= C + nIf. The elongation index, however, was found to have little effect on aggregate strength (Spence. 1979).

The decrease in aggregate strength (AIV, ACV) with increasing flakiness is the result of the low mechanical strength of the flaky particles. These are thin slivers with low tensile strength in the direction parallel to their shortest dimension. Within the sample these particles are in point contact with more massive cuboidal ones, and when impact or slow compressive load is applied the flakes fail easily in responce to stress concentration under conditions remniscent of point load testing and are preferentially eliminated (Fig 6. 5).

a 1- Results and Discussion:

The AIV and ACV programme on the present rock suite investigated aggregate shape. In addition to the variability between quarry products I_F was varied artficially from 0 to 100 for each rock type. All the rock types tested display a similar linear relationship between aggregate strength and I_F Fig (6. 5, 6) although the gradients "n" and C intercepts varied from rock to rock. The difference in petrographic constant "C" and the coefficient of flakiness "n" from one rock type to another is a function of geological features, i.e grain size, fabric, and degree of weathering.

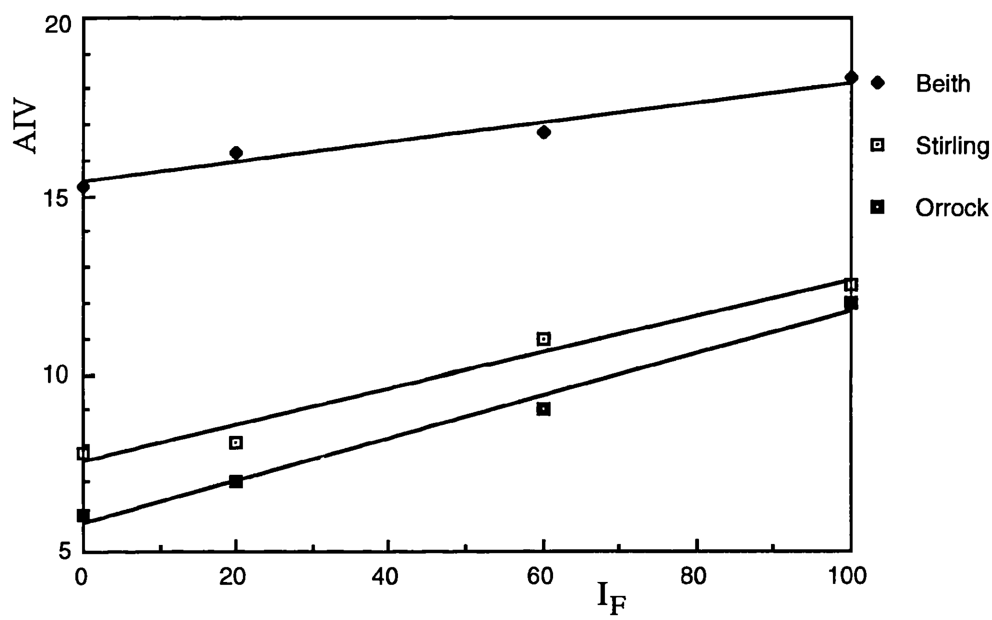


Fig 6. 5 Graph showing the influence of Flakiness on the aggregate impact value

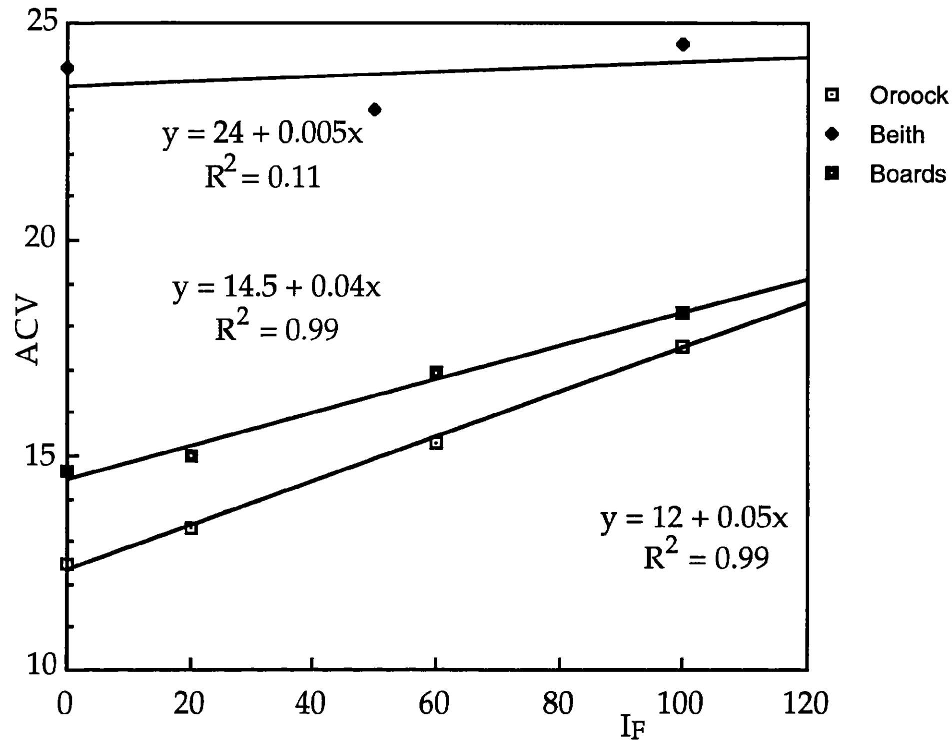


Fig 6. 6 Graph showing the influence of flakiness index on the aggregate crushing value

As seen from Fig (6.5, 6.6) the effect of I_F on the material strength is much more pronounced in stronger rocks such as microporphyritic basalt and dolerite than weaker material such as amygdaloidal basalt or coarse grained granite.

For the two non standard values AIVR and ACVR, a negative linear relationships with I_F (Fig 6. 7 and 6. 8) are obtained.

$$AIVR = C - nIf$$

and

$$ACVR = C - nIf$$

As with the standard Impact and crushing values, each rock type has it's own "C" intercept and slope of the curve. For microporphyritic and amygdaloidal basalt the influence of Flakiness Index on Aggregate Impact and Crushing Residue Values is quantified as following:

AIVR = 59 - 0.36 If

ACVR = 40 - 0.25 If

AIVR = 37 - 0.19 If

ACVR = 32 - 0.19 If

The steeper slope of the AIVR or ACVR - I_F curves reflect the increased sensitivity of the residue values to the proportion of flakes.

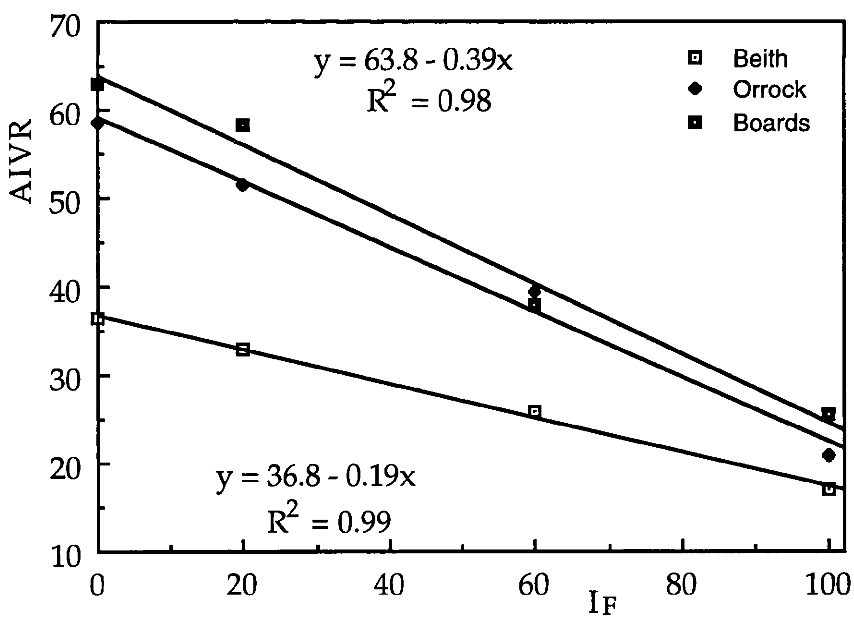


Fig 6.7 The influence of flakiness index on the Aggregate Impact Value Residue for amygdaloidal and microporphyritic basalts and quartz dolerite

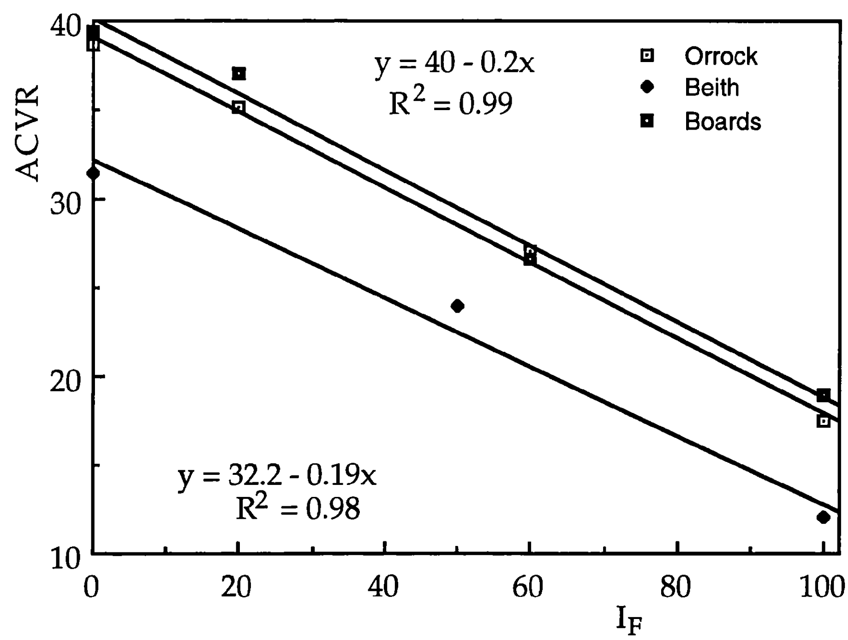


Fig 6.8 The influence of flakiness index on the ACVR for amygdaloidal and microporphyritic basalts and quartz dolerite

b- Grain size:

In order to study the effect of grain size on aggregate strength, other variable such as flakiness index and petrology have been eliminated. This was done by comparing strength values from aggregates of similar mineralogy with flakiness index set at 0. For basic rocks, fresh microporphyritic basalt aggregate displays a mean AIV of 6 and a mean ACV of 11. Medium grained quartz dolerite, however, has a mean AIV = 8.5 (range 6.5 - 11), and ACV = 14.5. The marked difference in strength is due to the differences in grain size (Fig 6. 9). For acidic rocks, however, granite has an AIV varying from 14 - 19 and a mean ACV of 22.5 while fine grained aplite has an AIV = 7 - 9 and an ACV = 12 - 13. Unlike the basic rocks, the strength difference between the coarse grained granite up to (2.7 mm) and the fine grained aplite (0.33 mm) is significant.

The strength decrease of the granular material as grain size increases can be explained in exactly the same manner as intact rock page (83).

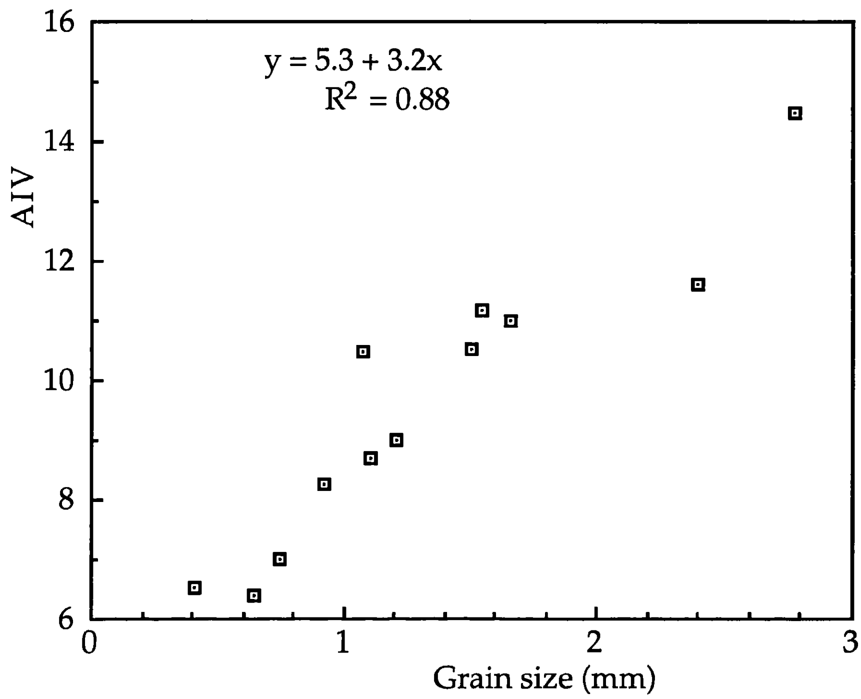


Fig 6.9 Graph showing the relationship between AIV and grain size for quartz dolerite

c- Mineralogy:

Fundamental differences in mineralogy and chemistry between rocks such as carbonates and silicates, are associated with different mineral hardness and different chemical bonding strength between mineral grains and consequently different rock strength. In studying the AIV, Spence (1979) indicated that despite similar interlocking texture and grain size carbonates and basic igneous rocks display very different resistance to pulverisation. The carbonates had a petrographic constant "C" varying in the interval 13.8 - 15.6 with a mean of 14.11, while "C" for the basic igneous rock varies between 7.6 and 11.9 with a mean of 9.49. But among fresh silicate rocks Ramsay et al (1974) and Spence (1979) showed that rock types ranging from acid to basic in composition have similar strength, and proposed a classification based on AIV and independant of rock petrology.

The results obtained from the present study show that despite the difference in the mineralogical composition between basic, intermediate, and acidic igneous rocks their aggregates have a similar resistance to pulverisation, i.e. the mean AIV and ACV for quartz dolerite, dacite, and aplite(Table 6. 1, 2, 3). The common factor between these rocks is they all have fine to medium grain size and an interlocking texture. The comparison of the results obtained from rocks of the same mineralogy such as granite and aplite, and basalt and quartz dolerite show big differences. Thus it can be concluded that the variation of aggregate resistance to pulverisation (AIV and ACV) among fresh silicate rocks is independent of rock chemistry but depends on texture, fabric and grain size of the rock.

This relationship between aggregate strength and grain size can be explained by the fact that within polymimerallic fresh silicate rock, the intergranular chemical bonds are of the same nature. These are not widely different from each other in terms of magnitude, and are greatly influenced by the grains boundary conditions, since the bonding at the grain boundary is less than the adjacent intergranular phase (Savanick and Johnson 1974). Among these conditions are the area of contact between minerals and the presence of microcracks and crack-like cavities, and impurities. In fact these imperfections tend to reduce the bonds which hold the rock together and subsequently weaken it. It is well estabished in the literature of the fracture mechanics of rock that failure, under both static and dynamic conditions, occurs when the preexisting cracks grow and coalesce. Therefore under loading, among fresh silicate rocks, the grain boundaries are sites from which cracks and flaws will more likely start to propagate, leading to failure, whatever the mineralogy of the rock. In rocks having interlocking texture, i.e. ophitic texture, there is more probability that a critical crack will be stopped by a neighbouring grain while this probability is very small in intergranular texture. Therefore, within a given rock group such as igneous rocks, the strength is strongly influenced by it's grain size and texture.

d- Weathering and Alteration:

In the discussion of intact rock it was demonstrated that strength was strongly affected by weathering and alteration. This present study sought to see whether this relationship carried over into crushed rock aggregate.

The rocks investigated display a systematic variation in AIV and AIVR with two widely used indicators of weathering namely, the content of secondary constituents and water absorption . Fig (6. 10) a plot of AIV against the volume of secondary mineral, amygdales, and vesicles in basalt displays a positive linear relationship, while AIVR decreases linearly. A similar pattern was obtained for quartz dolerite Fig (6. 11).

The ACV and ACVR are similarly affected by weathering. Fresh and weathered olivine basalt display a wide variation in values, e.g. fresh olivine basalt has an ACV of 11 - 12.5 and an ACVR of 38 - 40 , while for the weathered one these are 23.5 - 24.5 and 31 - 32.5 respectively. Similarly for fresh quartz dolerite ACV and ACVR are 14 and 41 - 42 respectively, while in the weathered material they are 18 - 20 and 34 - 30. The differences in the ACV and ACVR values between weathered and fresh material are to some extent reduced by a cushioning of the aggregate in the fines produced. Shergold and Hosking (1959) introduced the 10 % Fines Value (BS 812, 1975) especially for weak aggregate to overcome this cushioning effect. Weinert

(1964) showed that the difference in the 10 % Fines Value between aggregates of different states of weathering is greater than between their ACVs. Goureley (1986) also found that the 10 % Fines Value was useful in differentiating different weathering grades. In the present study fresh alkali olivine basalt exhibited a 10 % Fines Value of 380 kN while weathered amygdaloidal basalt it falls to 150 - 170 kN.

The Los Angeles Abrasion Value is also sensitive to the effect of weathering but discussion of this is presented in chapter V.

As seen earlier in chapter III, the reduction in strength of granular textures on weathering is also the result of intergranular bond weakening, crack opening, and the structurally weak nature of secondary minerals.

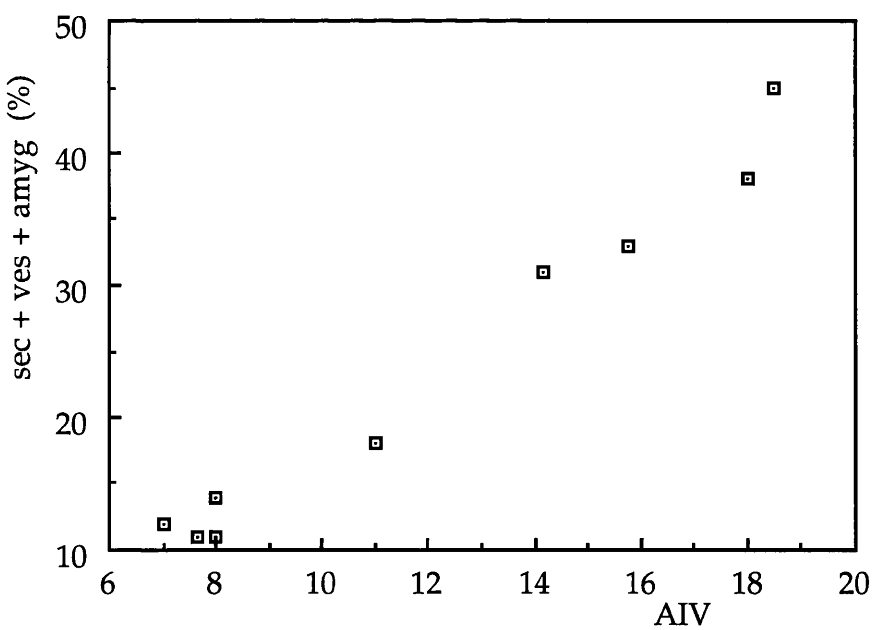


Fig 6.10 The influence of alteration product on Aggregate Impact Value for Basalt

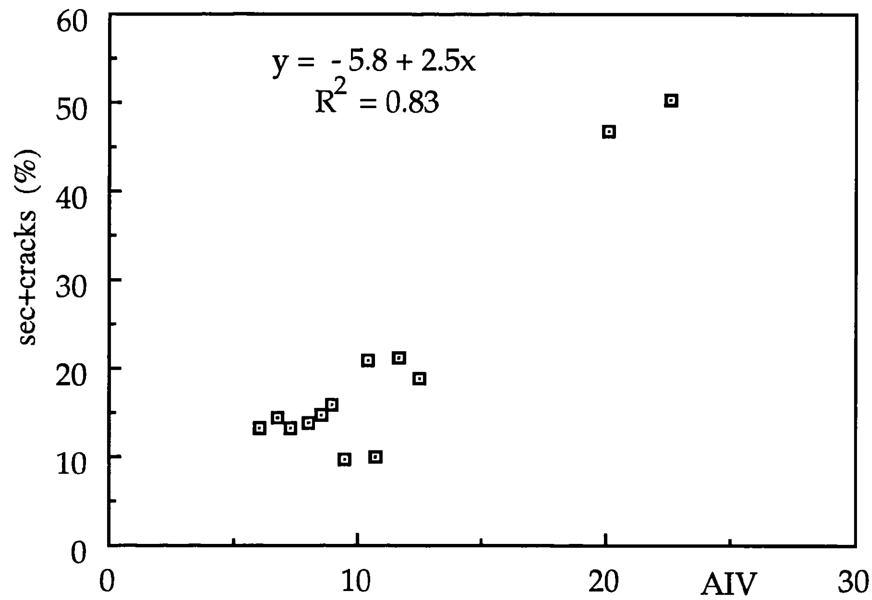


Fig 6.11 The relationship between AIV and the volume of secondary constituent for quartz doerite

e- Influence of Water:

Aggregates, like intact rocks, exhibits a decrease in strength, some times dramatic, when wet. Therefore, within an engineering structure, i.e, road, the location of aggregate material with respect to the water table has a significant effect on their durability and resitance to the imposed load. The availability of water when the material is subject to dynamic loading, i.e, impact, abrasion, and crushing, reduces material strength and in addition enhances the effectiveness of the physical weathering agencies to which the material will be subject. However, the quality and type of aggregate chosen is of considerable importance in these circumstances. Weathered and moisture suceptible aggregates may cause serious problems if located in a structure below the water table or at the reach of moisture, especially in unbound conditions.

In this study it has been observed that considerable decrease in strength occurs when aggregates are fully water saturated. Moreover, this decrease is greater in weathered rocks. Weathered basalt from Beith exhibited a considerable decrease in aggregate strength when wet, i.e, a saturated AIV of 20 to 21 as against 15 - 16 when air dry (Fig 6. 12). On the other hand, fresh basalt from Orrock and Lang Side and dacite show almost no decrease in aggregate strength when saturated. The quartz dolerite studied was only slightly weathered and exhibited a very slight decrease in strength, about 1% absolute value, when saturated. It should be noted that, although water dramatically influences the durability and resitance of aggregates to different types of loading, it is not as marked as in intact rocks.

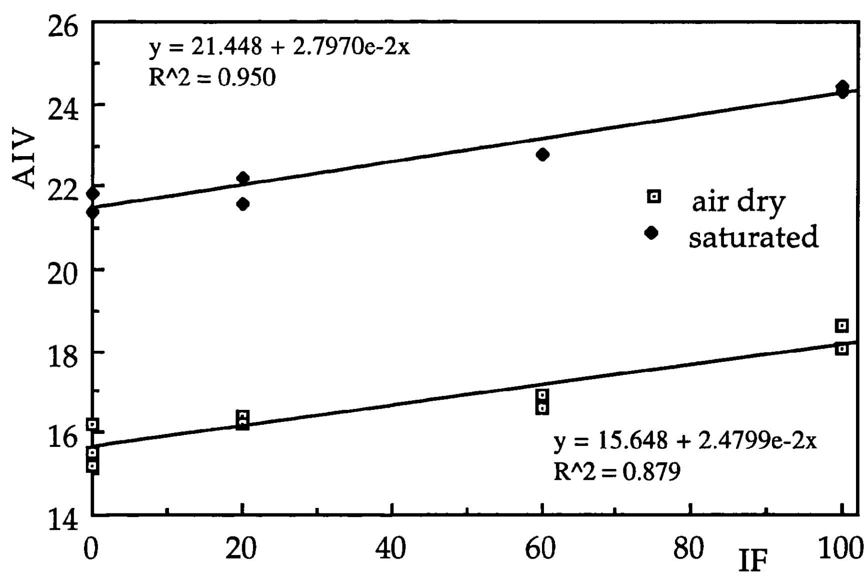


Fig 6. 12 Influence of water saturation
AIV for basalt

6 2 2 2- Soundness:

Soundness is the ability of aggregate to resist deterioration resulting from a change in the physical environment, such as wetting-drying and thermal changes at a temperature below zero often known as "freeze - thaw".

The freeze -thaw process commonly affects porous rocks. When the water freeze in the pore spaces it develops internal stresses which can overcome the tensile strength of the rock leading to bursting. Moisture suceptible minerals such as clay, swell in the presence of water and this leads to the build-up of internal pressures able to open cracks and cause deterioration of the rock. For this reason certain tests have been designed to simulate the effect of these destructive processes and allow the prediction of the likely "in service" performance of the material. Among such tests are the Magnesium Sulphate Soundness (BS 812, 1989), the Methylene dye absorption, and the Modified Aggregate Impact Value (BS 812, 1975).

a- Magnesium Sulfate Soundness:

Evaluation of aggregate durability using a sulphate solution was first proposed by (Brard 1828). The reasoning underlying the use of such a solution is that when Sodium Sulfate Solution (Na_2SO_4) crystallises, it simulates the effect of freezing water within the porespace of granular material.

The many factors which affect this test are, the temperature of the solution (Jackson, 1930), the solution concentration (Garrity et Al 1935,Walker et Al 1936; Wuerpel 1939; Mather, 1947), the temperature and the period of drying (Bloem, 1948; Woolf, 1956), the period of immersion in the solution (Woolf, 1956), the use of the magnesium sulphate solution as a replacement to the sodium sulphate solution (Ira, 1932; Walker et al, 1936; Dallaire, 1976), aggregate dimensions (Woolf, 1953), and the sieving operation for loss evaluation (Woolf, 1953).

Many reasons have led to the substitution of sodium sulphate by magnesium sulfate, among which is the lower sensitivity of the magnesium sulfate concentration to temperature variation. In addition, sodium sulfate solution crystallises in many and unpredictable forms leading to different amounts of disintegration, while magnesium sulphate crystallises in only one form (Hosking and Tubey, 1969). In the ASTM standard C88, 1990 it was shown that the magnesium sulphate test is more repeatable than the sodium sulphate test. For these reasons the former has become the recommended one for the evaluation of aggregate soundness.

For representative values, the size of the aggregate tested should not be less than 4 mm size. Gribble (pers comm) suggested that for small sized aggregate, one would be no longer testing the soundness of the rock, but rather the mineral grains.

a 1- The deterioration mechanism:

Theoretically, the mechanisms involved in the deterioration of aggregate by the use of magnesium sulphate solution are quite simple. On first immersion of the aggregate, the solution enters the pores, and in the course of subsequent drying, a salt of magnesium sulphate crystallises in the porespaces in a monohydrated form. During a second immersion, those monohydrate magnesium sulphate ($MgSO4\text{-}2H_2O$) crystals which do not dissolve, change to a hydrated form ($MgSO_4\text{-}7H_2O$) which have a larger volume. The transformation from the monohydrated to the hydrated form in a confined space, generates a pressure and leads gradually to a degradation of the material. According to this proposal, the degradation of the aggregate particles should start and continue at an increasing rate from the second cycle of immersion onwards. Theoretically, Bloem (1966) successfully demonstrated the existence of a limit prior to which no degradation would occur. This critical limit corresponds to the cycle when there is no free movement of the solution through the particle porespace. Despite this explanation, it was found in practice that some degradation did occur during the first cycle of immersion (Aubertin, 1982), suggesting that other factors must have been involved in the degradation process.

a 2- Results and discussion:

The magnesium sulphate soundness test was performed on 14-10 mm aggregate in accordance with the (BS. 1989). The test consists of five cycles of immersion in a concentrated solution of $MgSO_4$ and drying. Each cycle includes 17.5 hours immersion in a concentrated magnesium sulfate solution at a temperature of 25 ^{O}C. After this, the sample is removed from the solution and allowed to drain for about 15 min, and then removed and oven dried at a temperature of 105 to 110 ^{O}C until a constant mass is reached. After that, the sample should be allowed to dry in a dessicator (room temperature) for about 5h15 min. To complete the test, four more identical cycles should be performed to complete the five cycles required. The material remaining on the original sieve after the fifth cycle expressed as a percentage of the whole sample is the soundness of the material.

A limit of 80% was set as a discrimination value below which the material is considered as unsound. Many authors such as Weinert (1964), Hosking and Tubey (1969), Collis and Fox (1985) stated that the magnesium sulphate soundness results should not be taken as a definite criteria for acceptance or rejection of the aggregate material. On the other hand Hosking and Tubey (1969), in studying the cause of failure of a runway in Mauritius, found that the Sodium Sulphate Soundness test was very effective for discriminating between the sound and unsound material.

In testing the aggregate 14 - 10 mm lower values were characteristic of mechanically weak aggregate.

The amygdaloidal and vesicular basalt from Beith is characterized by high porosity and water absorption upto 9 %, a relatively high Aggregate Impact Value of 16 to 20, and a Los Angeles Abrasion Value (LAAV) of 21 to 24 , and shows a considerable loss in strength when saturated. The

Magnesium Sulphate Soundness values for this material are found to be in the range of 60 to 65.

In the fresh microporphyritic alkaline basalt from Orrock and Lang side, with low absorption values and good mechanical properties, the magnesium sulfate soundness tests was very satisfactory. The losses after five cycles of immersion in the concentrated Magnesium Sulphate solution were in the range of 4 to 2.5 percent for for I_F = 100 and 0 respectively. The other materials, i.e. dacite from Balmullo and the quartz dolerite from Stirling displayed very satisfactory Magnesium Sulphate Soundness results, 94.7 to 95.4 and 96 to 98 respectively.

The microscope examination of thin sections prepared from the tested material and the quick absorption tests (Hamrol 1961) revealed that higher Magnesium Sulphate Soundness values are associated with rocks having low or very low volume of secondary minerals and low water absorption values. On the other hand, material with high proportion of secondary mineral, vesicles, amygdales, and cracks, e.g Beith (45 %) suffers a heavy loss in the soundness test.

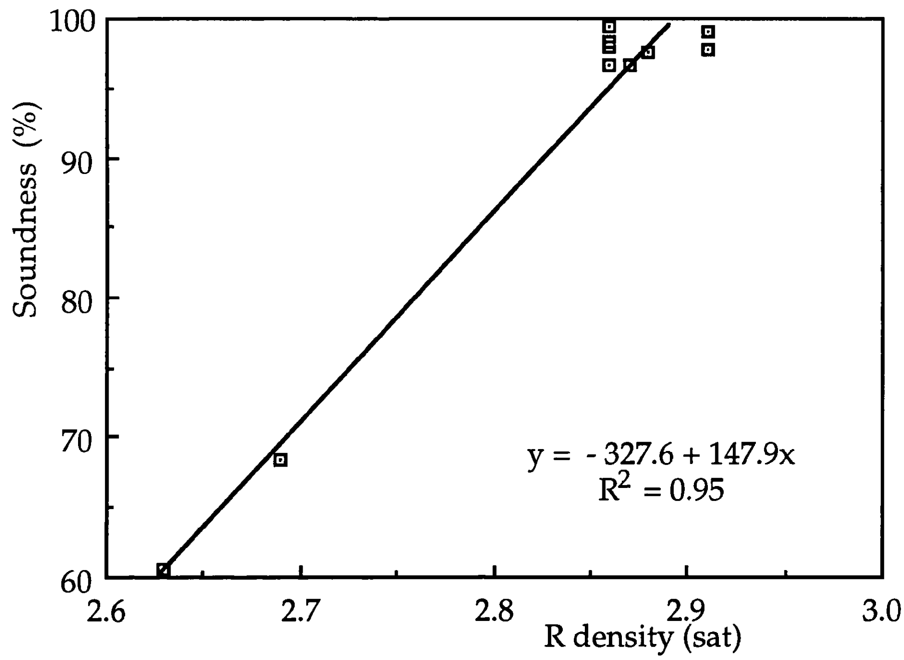


Fig 6. 13 The relationship between Magnesium Soundness and relative density on saturated and surface dry basis for quartz dolerite

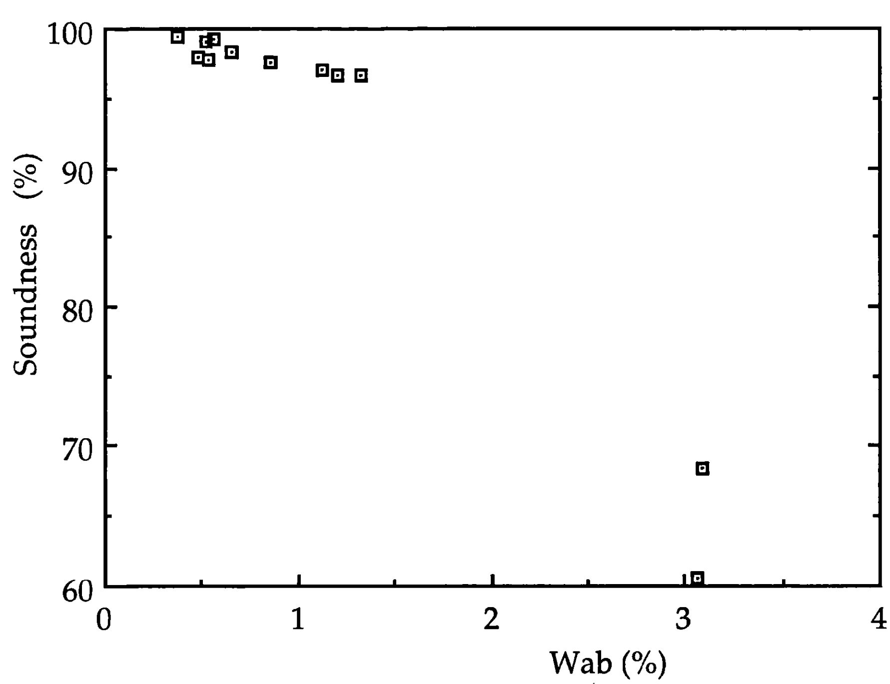


Fig 6.14a The relationship between Magnesium Soundness and water absorption for quartz dolerite

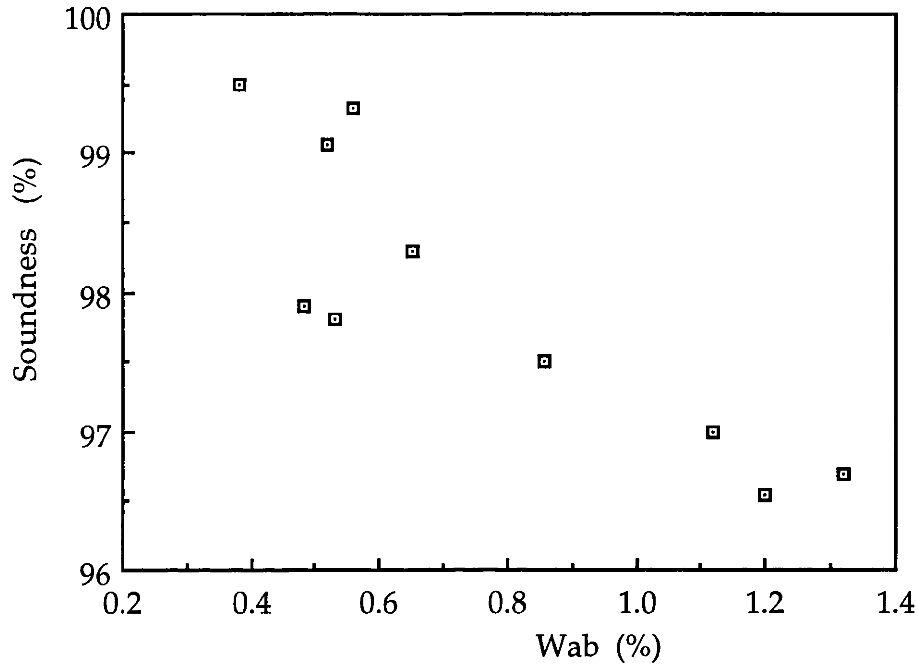


Fig 6.14b The relationship between Magnesium Sulphate Soundness and water absorption for quartz dolerite (for fresh and slightly weathered material)

b- Metylene Blue Dye Absorption:

The need for a rapid, cheap, and effective method of detecting lack of soundness in aggregate material has drawn attention to the methylene blue dye absorption test. The test was introduced in the 1950s by Fairbairn and Robertson (1957) and is now routinely used by highways researh labratories in Australia (Cole and Sandy, 1980). It is a standard test in Northern Ireland (Stewart and MaCulloch 1985) and France (Tran Ngoc Lan, 1980). Methylene blue is a dark green organic crystalline substance which forms a blue aqueous solution when disolved in water. In rock it is adsorbed by clay minerals which are the chief cause of premature failure of road aggregate. Swelling clay of the smectite family in basaltic aggregate has a very large specific surface area and therefore a very high adsorption capacity to methylene blue. The purpose of the test is to obtain a quantitative measure of the clay content of a sample of aggregate by measuring the amount of methylene blue solution adsorbed, often called the Methylene Blue Value (MBV).

Cole and Sandy (1980) found a good correlation between their secondary minerals rating index (Rsm) and the MBV for basaltic aggregate. The MBV was also found to give good correlation with the clay content (smectite) of aggregate while poor correlations were exhibited when aggregate contain mixed layer chlorite - smectite (Hills and Pettifer. 1985).

b 1- Test procedure:

A representative sample of aggregate is powdered to < 75 μm sieve size. One gramme of this powder is then taken and disperssed in 30 ml of distilled water. The suspension is then titrated by the methylene blue solution with a concentration 1 g/l by successively adding 0.5 ml increments. After each addition the suspension is agitated for one minute and a drop is removed by a glass rod and spotted on a filter paper. This process continues until a definite pale blue halo is seen to have formed on the paper when held up to the daylight. This is actually the end point and indicates that the particles are no longer able to adsorb any further methylene blue solution. The MBV expressed as the percentage mass of methylene blue adsorbed by the test portion, is then calculated using the formula:

$$MBV = \frac{0.1V}{M}$$

V(ml) volume of the methylene blue solution added to the suspension.

M(mg) mass of the test portion.

Based on experience with unsound basaltic aggregate, the department of the envirnment of Northern Irland has set up a limiting MBV = 1 for basalt, and 0.7 as a limiting value for gritstone.

In this study a few tests were performed on basalt and quartz dolerite aggregates. Amygdaloidal basalt aggregate yielded an unexpectedly low MBV. Although they are very weathered and unsound, or completely rotten, their MBV was around 0.4%. Stewart and MaCulloch (1985) found that vesicular basalt, despite the fact that they are friable, give very low MBV. The quartz dolerite yielded values varying between 0.5% and 1.66%. Again a highly weathered and fractured sample of quartz dolerite containing saponite clay yielded a value of 0.5% while sound and very high strength samples gave values of 1.5%.

c- The Modified Aggregate Impact Value (MAIV):

The Modified Aggregate Impact Value Hosking and Tubey 1969) is performed on saturated surface dry aggregate. The test procedure is the same as that for the standard AIV (BSI 812), the only difference being that the number of blows is reduced in order to yield between 5% and 20 % fines. The MAIV is then obtained using the following formula:

$$MAIV = m\frac{15}{x}$$

m = mass passing 2.36 mm sieve

x = number of blows

Hosking and Tubey (1969) investigating aggregate from Mauritius showed that after soaking for 20 days the MAIV indicated a considerable decrease in aggregate impact strength. In the dry state the Aggregate Impact Strength was 19, but with progressive increase in the period of soaking the MAIV revealed a systematic decrease in strength, and after 20 days it was 32 (Table 6. 4). This considerable decrease in strength upon saturation is most probably caused by the build up of internal pressure as a result of the swelling of the secondary minerals on one hand, and the elimination of bulking and cushioning effect by use of the MAIV on the other.

In the present programme a series of tests was performed on aggregate at different states of weathering and different secondary mineral content. The period of soaking in water was in each case 24h $\pm$ 30 min. The impact strength as measured by AIV and MAIV decreased systematicaly as the secondary mineral, and vesicle and amygdale content increased. Moreover the results show that as the MAIV increases the difference between the MAIV and it's corresponding AIV increases linearly. This means that as the proportion of unsound component in the rock increases the difference between AIV and MAIV increases (Fig 6.15). On the contrary, fresh basalt showed no decrease in strength when the Modified Aggregate Impact test is used. For the quartz dolerite aggregate, differences between AIV and MAIV do exist but not to the extent of those exhibited by the amygdaloidal basalt. Where the material is strong, i.e. AIV = 7 to 10, the difference is 1 to 2, and in one case where the material is weathered, (AIV = 16 on air dry basis at I_F = 0) the difference between AIV and MAIV was about 5 (Table 6. 2). It is thought that the small difference of 1 - 2 between AIV and MAIV is not a matter of soundness of the rock, rather it is the normal influence of water an aggregate strength.The difference of 5, however, does reflect unsoundness within the rock, and suggests a stress buildup within the rock as a result of the swelling of secondary minerals plus intergranular bond weakening on saturation. In addition the latter material exhibited a Magnesium Sulphate Soundness Value of 60 in contrast to the former which gave 96 to 99.

Table 6. 4 Results of Modified Aggregate impact tests on some Mauritius basic igneous rocks (after Hosking and Tubey. 1969)

Pretreatment	No. of Blows	MAIV
Dry strength	7	19
After 24 hours soaking	6	21
After 6 days soaking	6	24
After 20 days soaking	4	32
24 h soa+48h air drying + 8 h oven drying	6	21
24 h boiling in water	4	30

Table 6. 5 Some AIV and MAIV for quarry product aggregates set at zero flakiness index

Sample	No. of blows	AIV	MAIV
Basalt O	5	6	6.2
Basalt L	5	5.5	6.3
Basalt B	5	16	21
Quartz dolerite	5	7.66	8.1
Dacite	5	8.8	8.7
Granite N	5	14	16.8

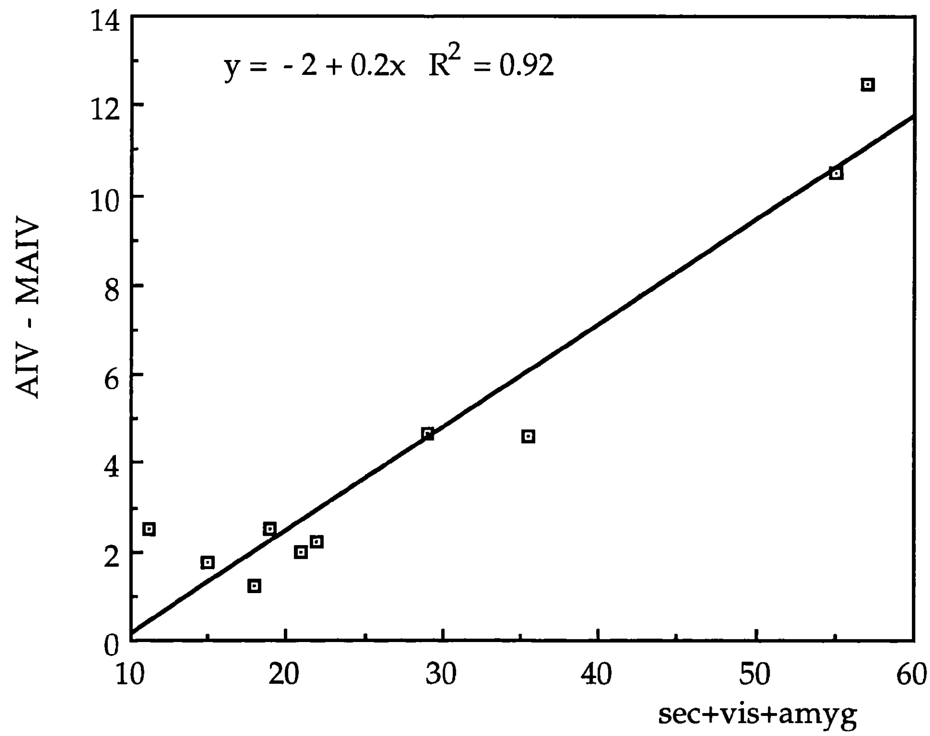


Fig 6.15 Graph showing the relationship between the rock aggregate secondary mineral content and the difference between AIVand MAIVfor basalt

6 2 2 3- Aggregate Toughness (Hardness):

Mineral toughness or hardness is the characteristic that controls aggregate wear. Minerals and rocks when subjected to abrasion, particularly in the presence of an abrasive, get worn due to scratching and pitting. In this process aggregates which contain hard minerals tend to be resistant. Several tests have been devised to measure mineral hardness such as Indentation Hardness, Rebound Hardness, Scratch Hardness, and Wear Abrasion Hardness known as Aggregate Abrasion Value (BS 812).

Hartley (1974) related resistance to abrasion to the petrology of the rock, in particular the proportion of hard minerals, the proportion and orientation of cleaved minerals, the nature of intergranular bonding and the state of alteration of the whole rock.

a- Aggregate Abrasion Value:

The Aggregate Abrasion Value (AAV) is a substitute for the abrasion value which, like the UCS, was obtained from cylindrical cores cut from selected samples. The AAV is a measure of the resistance of the aggregate to surface wear by abrasion and is based the loss of mass by abrasion from the test specimen.

In this test (BS 812) 33 cm^3 of clean 14 - 10 mm aggregate and retained on the 20 - 14 mm flake sorting sieve are set into polyester resin backing, allowing 6 mm protrusion of the aggregate particles (Tubey and Szafran 1970). The sample is firmly held against a lap rotating at 28 - 30 rev/min under a total load of 2 kg for 500 revolutions. Leighton Buzzard sand is fed in front of the samples at a rate of 0.7 to 0.9 kg/min. After 500 revolutions the loss in weight is determined and the AAV is calculated as follows:

$$AAV = \frac{3(A - B)}{d}$$

Where A - the mass of the specimen before the test

B - The mass of the specimen after the test

d - The relative density of the aggregate on saturated and surface dry basis

The Ministry of Transport require an AAV of 12 for normal heavily trafficked road sites and 10 for difficult sites such as sharp bends, steep hills and sections where abrasion is more severe. Inadequate resistance to abrasion means an early loss of surface texture of the road.

a 1- Results and discussion:

Following (BS 812. 1975) a series of tests was performed on the rock aggregate studied in this programme. Basalt, quartz dolerite, granite, and dacite were all tested (Table 6. 1, 2, 3). Acid igneous rock with high quartz content exhibited high resistance to abrasion, while basic igneous rocks were significantly lower.

Basalt aggregate displayed a wide variation in resistance to abrasion. AAV ranged from 2.5 for fresh material to 24 for the weathered and amygdaloidal variety. This variation reflects alteration and replacement of the primary hard minerals by soft secondary minerals such as chlorite, calcite, zeolite, clay minerals, and prehnite (Fig 6.16). Quartz dolerite also showed variation between 3 and 12 . Fresh samples are resistant and yielded values below 5 while coarse grained varieties, although they contain a considerable amount of secondary minerals, were resistant to abrasion, i.e. AAV= 4 - 6.5. AAV also increased with increase in fracture density, even when they have low secondary mineral content, i.e. adjacent to shear zones. In this case AAV is about 7 to 8. During the abrasion test wear is enhanced by disintegration of the mineral grains along the fracture planes and plucking. When the aggregate had both high secondary mineral content and fractures it exhibited the lowest resistance to abrasion (AAV of 12 to 13). Dacite aggregate exhibited a high resistance to abrasion (AAV = 3) while granite aggregate, despite the fact that it had lower strength, also exhibited a similar resistance (AAV = 3.6). In the case of granite the resistance is a function of the relatively high quartz content.

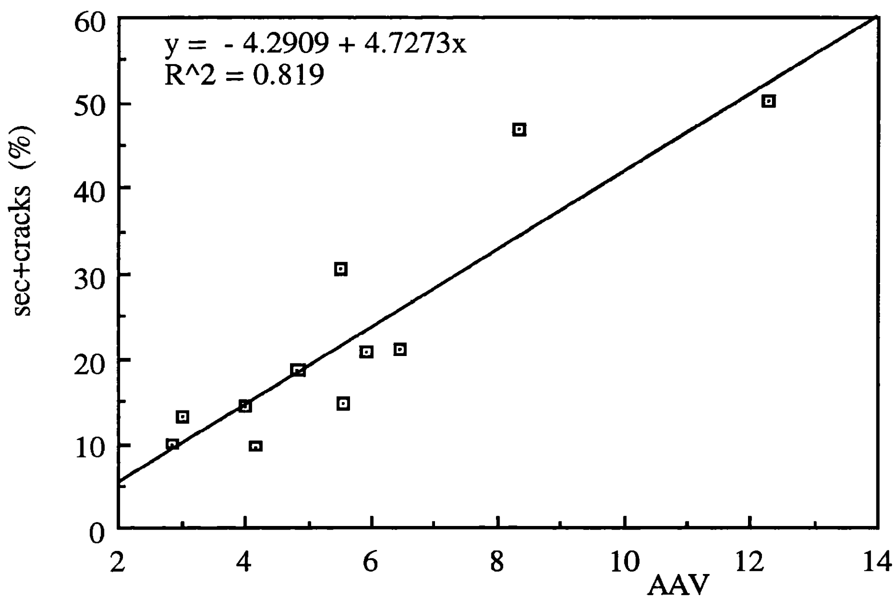


Fig 6. 16 The relationship between AAV and the volume of secondary minerals and cracks for quartz dolerite

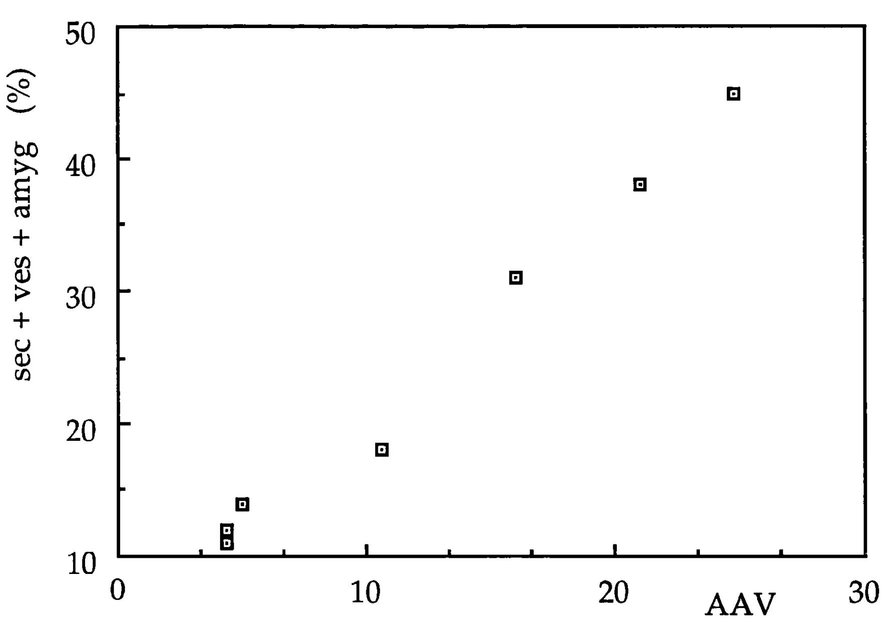


Fig 6.17 The influence of alteration product on the Aggregate Abrasion Value for basalt

b- The Los Angeles Abrasion Value:

The Los Angeles Abrasion Value as described previously (chap V) reflects the resistance of aggregate to both impact and abrasion. In our investigation it has been demonstrated that the degradation process involves about 20 % abrasion, and 80 % impact. Nevertheless, LAAV shows a high degree of correlation with the AAV (Fig 6.18). In the case of coarse granite aggregate the linear relationship between LAAV and AAV is impaired. This is mainly due to the high resistance of the quartz grains and the lower resistance to impact.

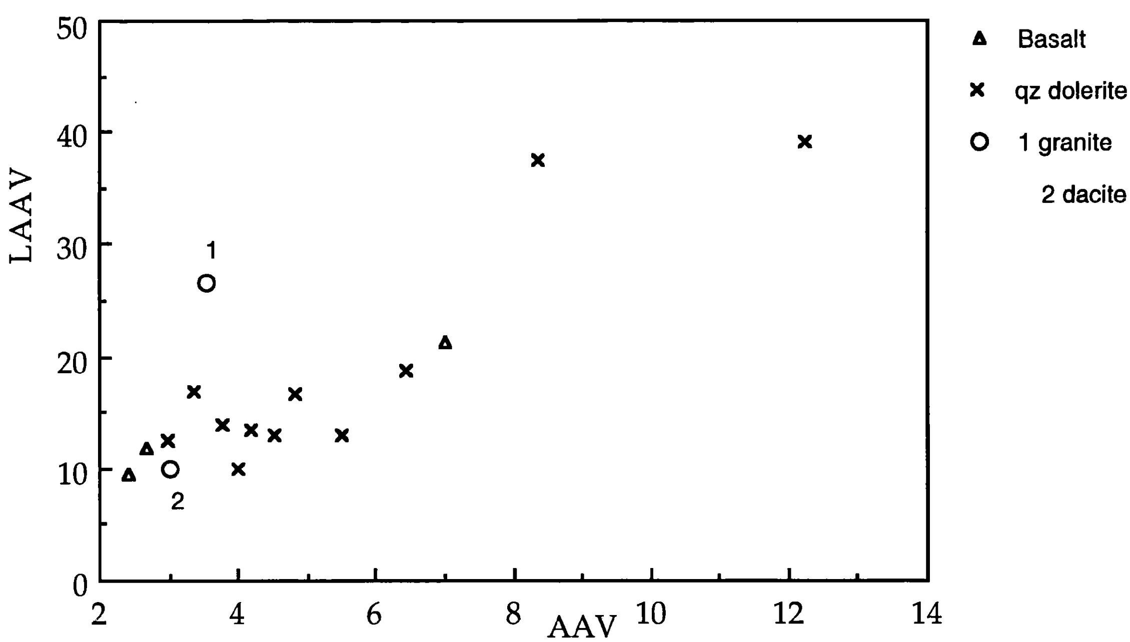


Fig 6.18 The relationship between LAAV and AAV

CHAPTER VII

Correlation between intact and aggregate rock strength

7 1- Introduction:

This investigation has revealed a general pattern in which the mechanical and physical properties of both intact rock and crushed rock aggregate are affected in a similar way by the same factors, i.e., grain size, texture, secondary mineral and crack content, porosity, and moisture content. Although it has been noticed that the magnitude to which they are affected differs, in some cases, from intact rock to aggregates, e.g. the UCS is usually more sensitive to the variation in grain size, texture, structural defects, and amount of secondary constituents than aggregate strength while the pattern of the relationship is similar. An identical relationship has been revealed by the LAAV and the newly proposed LAAVR.

The correlation between indices of aggregate and intact rock strength endorses the status of UCS, PLS, R, and Seismic velocity as index properties for aggregate quality.

7 2- **Correlations:**

UCS - AIV:

Despite the fact that Aggregate Impact Value reflects the resistance of rock aggregate to pulverisation rather than the ultimate strength or loss of bearing capacity recorded by the UCS, they correlate quite well if the range of strength is wide enough and aggregate used is set to a constant value of flakiness index, i.e $I_F = 0$. In Fig 7. 1 the relationship has the hyperbolic form so commomly exhibited between physical and mechanical rock properties. Since the geological variable is weathering, this hyperbolic relationship suggests that at an early stage of weathering, only small amounts of secondary minerals on the grain boundaries cause a significant decrease in intact rock ultimate strength, while aggregate resistance to breakdown and pulverisation remain high, or just slightly decrease. As weathering progresses, the amount of secondary mineral increases, the ultimate strength of the rock decreases, but not at the same rate as in the initial stages since the critical stages when it decreases dramatically is the development of the first weakness planes from which cracks start to propagate, while AIV increases dramatically due to the lower pulverisation resistance and the fines they produce upon weak secondary mineral brakdown and clay release. A similar but positive relationship is exhibited between UCS and AIVR (Fig 7. 2).

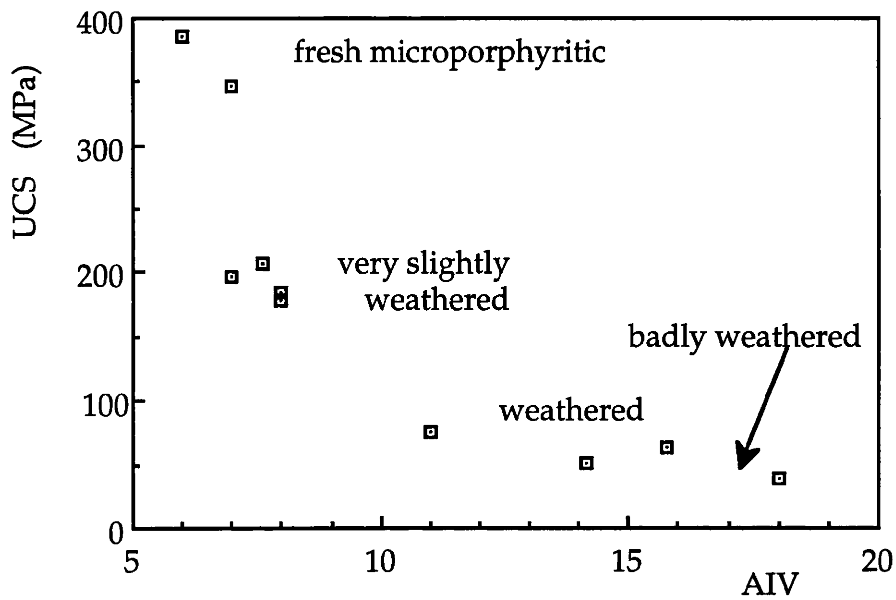


Fig 7. 1 The relationship between UCS and AIV for basalt at different weathering states

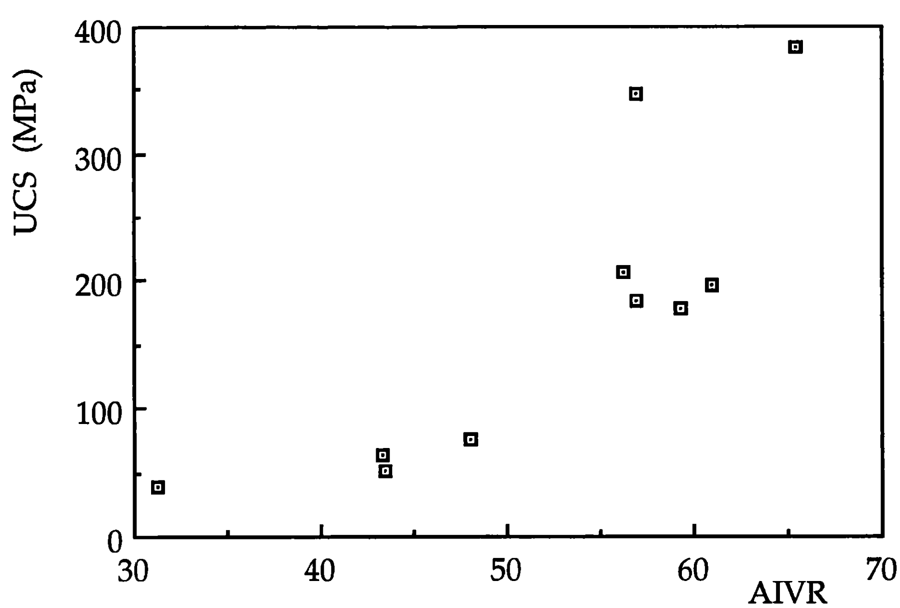


Fig 7. 2 The relationship between UCS and AIVR for basalt at different weathering states

UCS - LAAV:

The correlation between the UCS and both LAAV and LAAVR is a quite good linear one for quatrz dolerite and granite, i.e. a steady and progressive in LAAV with falling UCS (Fig 7. 3, 4).

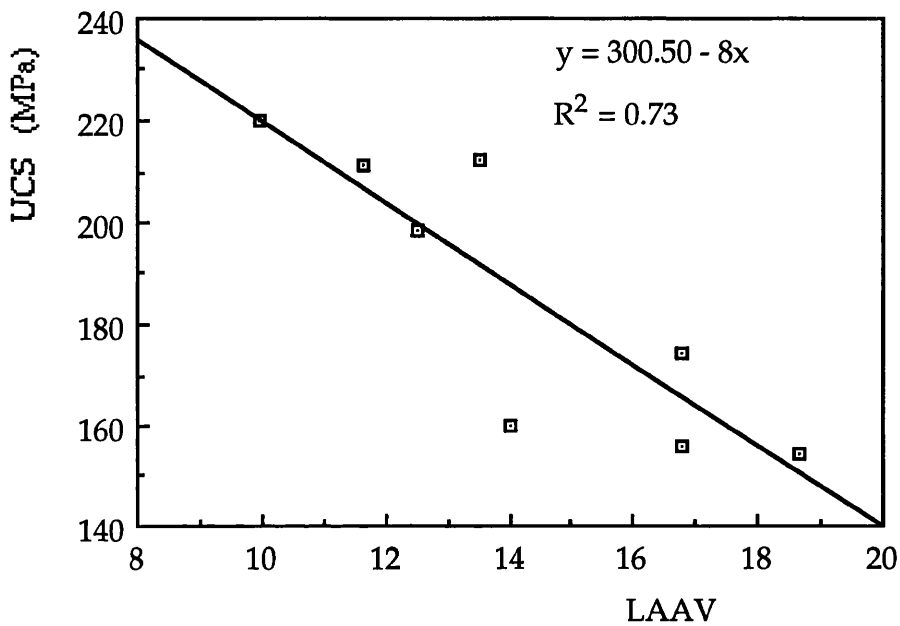


Fig 7. 3 The relationship between UCS and LAAV for quartz dolerite

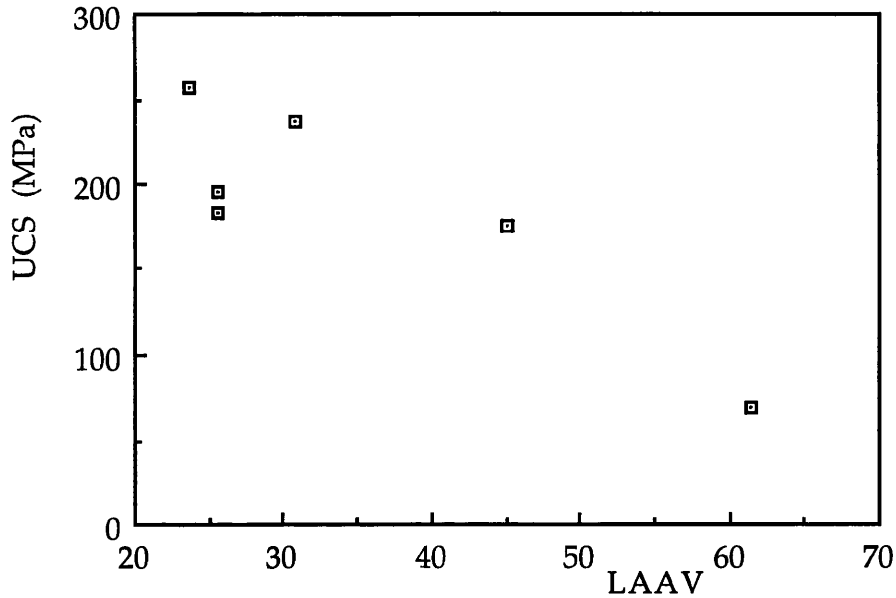


Fig 7. 4 The relationship between UCS LAAV for granite

UCS - AAV:

The comparison between the UCS and the AAV for basalt and quartz dolerite (Fig 7. 5, 6) respectively, shows that as the UCS decreases due to weathering the AAV increases. In the case of basalt the relationship is best fitted using a logarithmic function of the type:

$$UCS = a - b\log AAV$$

a and b are constants

The reason behind this type of relationship, despite the different properties the tests are assessing, is that during weathering intergranular bond weakening and mineral alteration proceed together. In the fresh state material is of both high strength and resistance to abrasion, during the first stage of mineral alteration, mainly along grain boundaries, strength is significantly reduced while abrasion resistance is only slightly reduced. As mineral alteration progressively increase, mineral resistance to abrasion subsequently decreases in a greater rate than the ultimate strength decrease.

Quartz dolerite on the other hand exhibit two parallel lines when UCS is plotted against AAV. The lower line represent samples with higher plagioclase-feldspar and quartz content.

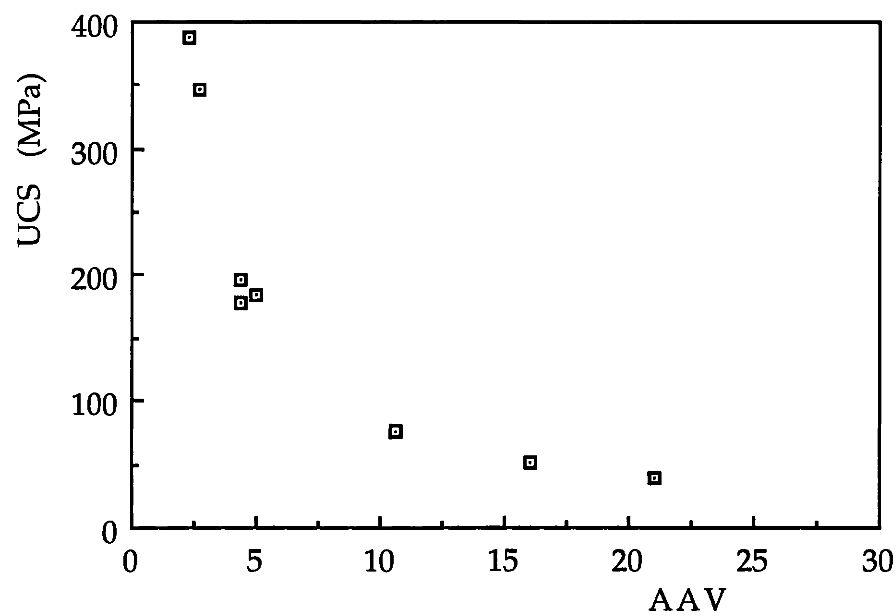


Fig 7. 5 The relationship between UCS and AAV for basalt

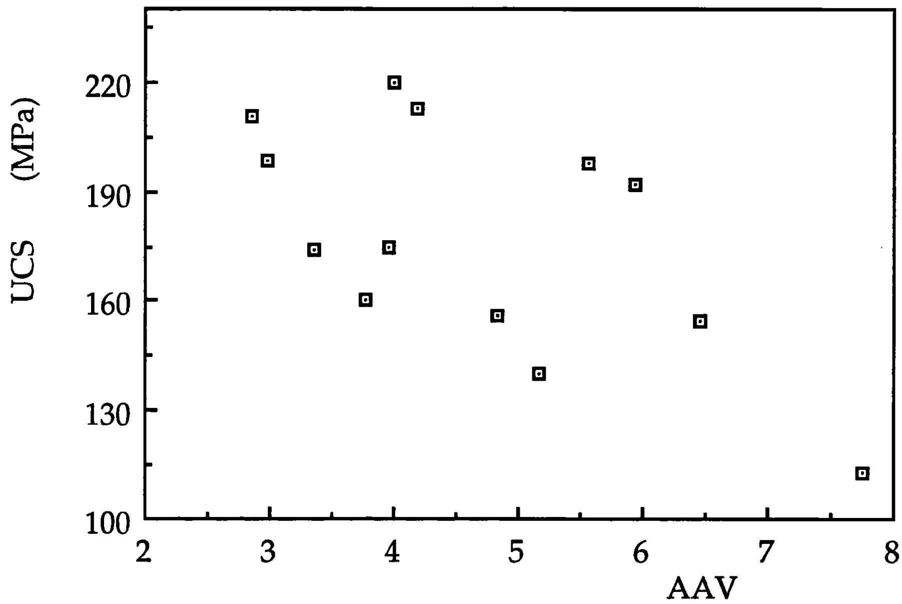


Fig 7. 6 The relationship between UCS and AAV for quartz dolerite

UCS - Mg Soundness:

Although the magnesium sulphate soundness is a chemical test designed to simulate the effect of freezing water within the pore space of the aggregate material, when correlated with the UCS it shows a trend where the UCS decreases as the loss in the Sulphate Soundness Value, i.e. the weathering is increasing (Fig 7. 7).

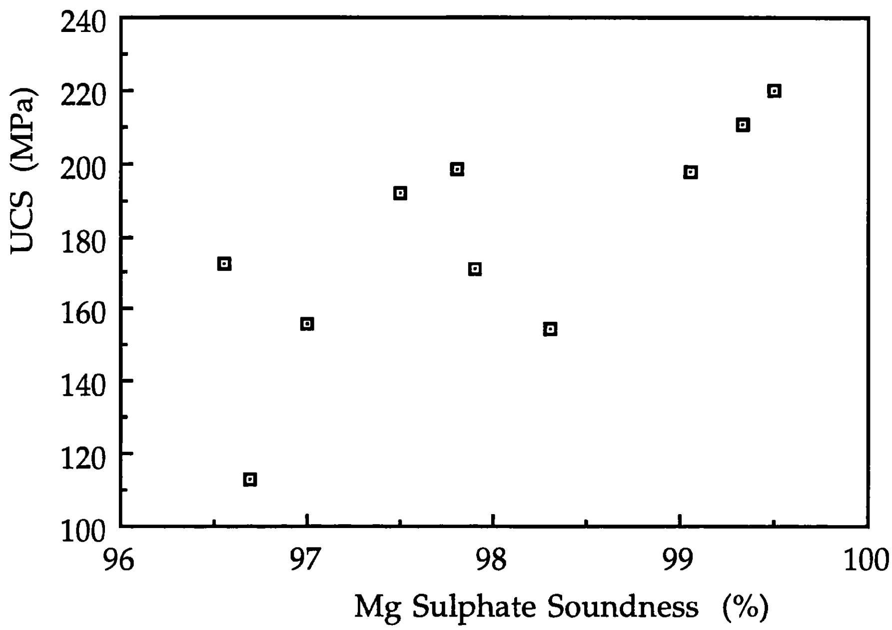


Fig 7. 7 The relationship between UCS
Mg Sulphate Soundness for quartz dolerite

PLS - AIV:

The correlation between the common index property of strength, Point Load Strength, and the aggregate impact value gives a curvelinear relationship (Fig 7. 8, 9)). This relationship is best described using a function of the type:

$UCS = a - b \log AIV$

a and b are constant

The point load strength and AIV display a negative relationship. In the case of quartz dolerite, the relationship is nearly linear before the material reach the weakened stage, i.e. PLS > 4 MPa and AIV < 13 - 14. As the material becomes weakened the curve describing the relationship deflects asymptotically to the AIV axis and the overall relationship between PLS and

AIV becomes exponential. The curve deflects asymptotically to the AIV axis at the stage when pulverisation of the weak rock become prominent. In the case of basalt the curve is a near linear one.

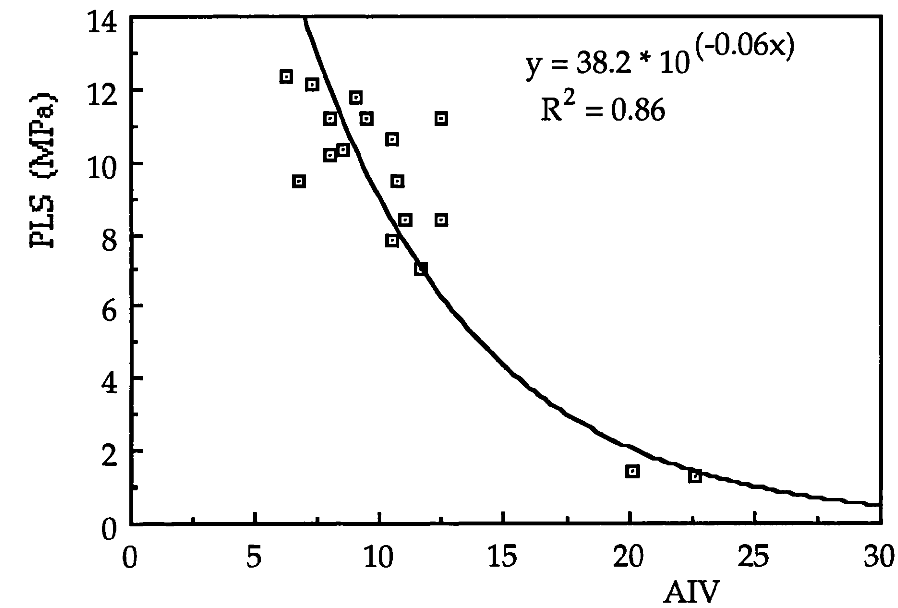


Fig 7. 8 The relationship between PLS AIV for quartz dolerite

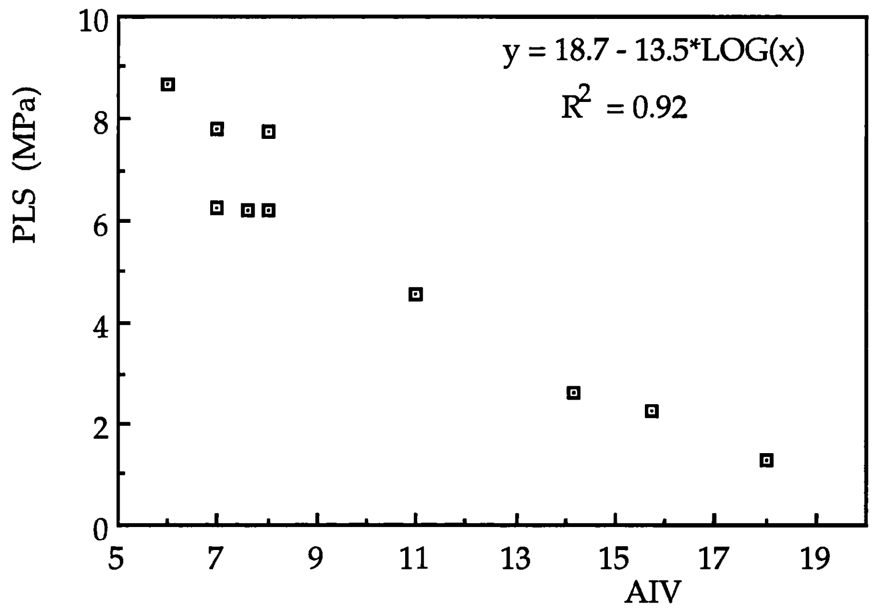


Fig 7. 9 The relationship between PLS and AIV for basalt

PLS - LAAV:

As with the AIV, the Point Load Strength exhibits a curvilinear relationship with the LAAV. It is best expressed by a logarithmic function of the type:

$PLS = a - b\log LAAV$

a and b constants

The relationship is initially linear deflecting to non linearity as the material becomes weakened (Fig 7. 10). In the case of granite the relationship is linear with moderately steep negative slope (Fig 7. 11).

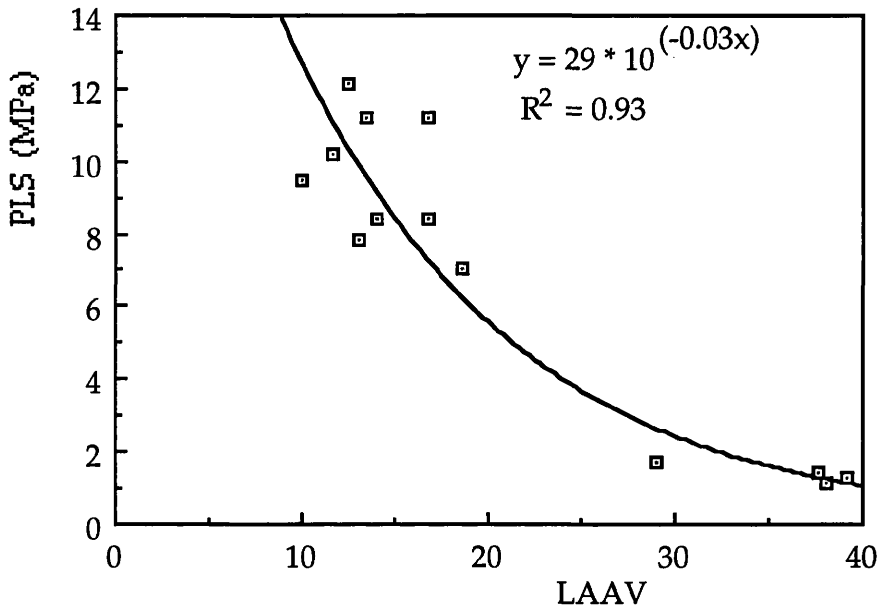


Fig 7. 10 The relationship between PLS and LAAV for quartz dolerite

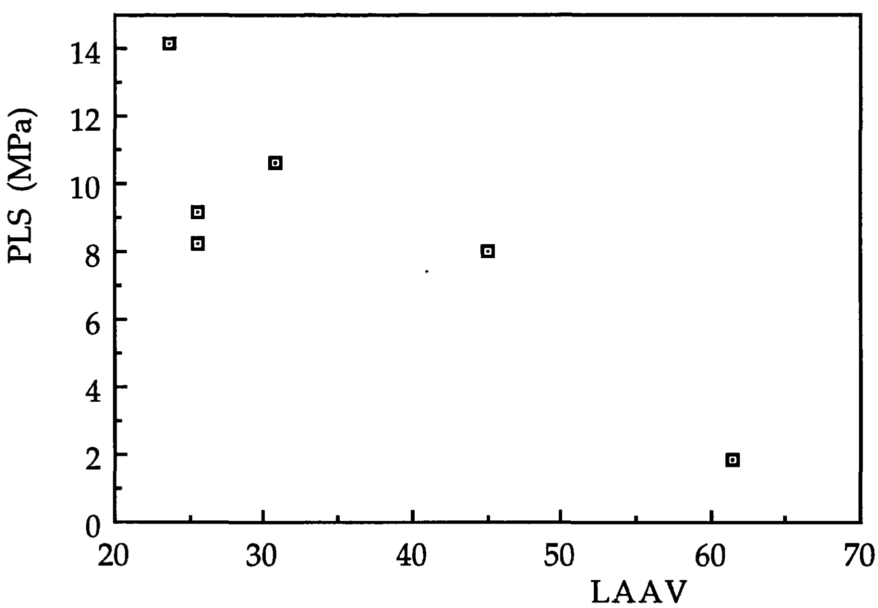


Fig 7. 11 The relationship between PLS LAAV for granite

PLS - AAV:

The resistance to abrasion and the tensile strength of the material, although they are two different properties, correlate quite well. It is shown (Fig 7.12) that with the initial increase in AAV the PLS decreases sharply after which it decreases gradually as the AAV increases.

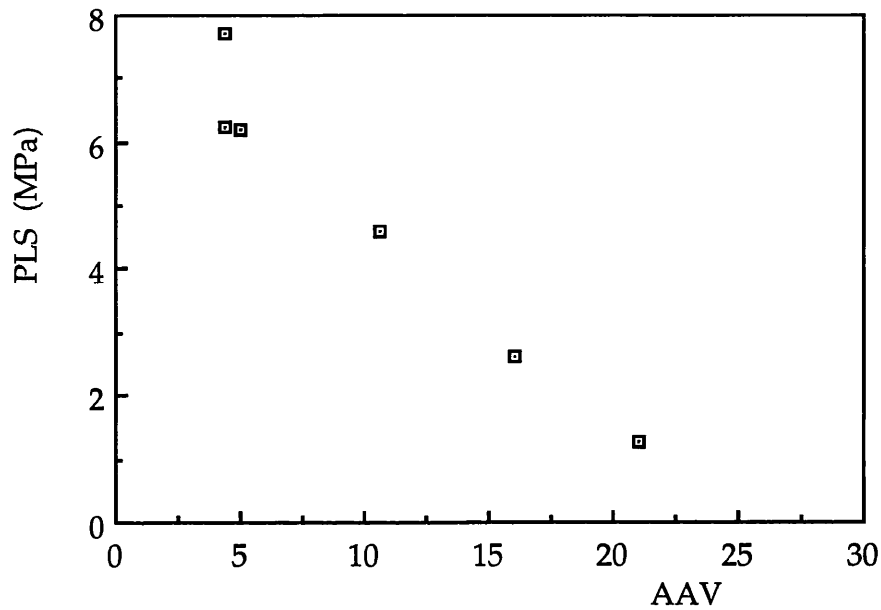


Fig 7. 12 The relationship between PLS and AAV for basalt

PLS - Mg Sulphate Soundness:

As with the UCS, the Sulphate Soundness values show poor correlation with the Point Load Strength. The graphical representation of this relationship (Fig 7. 13) reflects that at the weakened stage a heavy loss of material upon salt crystalisation occurs. It correspond in fact to the stage when the PLS falls bellow 4.

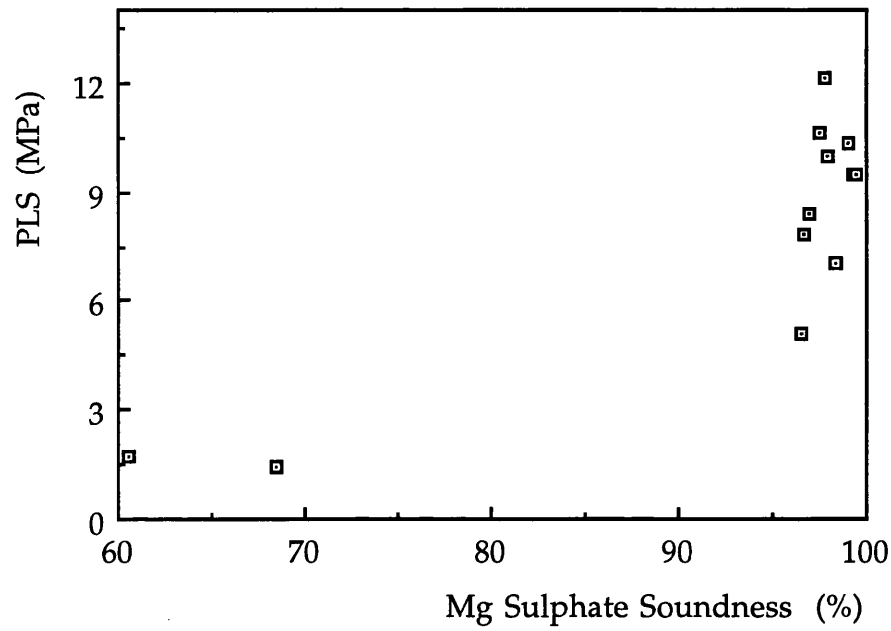


Fig 7. 13 The relationship between PLS and the Mg Sulphate Soundness for quartz dolerite

<u>Rebound Number - AIV:</u>

The correlation between Schmidt Rebound Number and the AIV (Fig 7. 14) is best described using a function of the type

$$R = a + b \log AIV$$

a & b are constants

Although, R is an assessment of the material elasticity and to some extent the surface hardness, it correlates well with the resistance to pulverisation. The fact is that both values are governed by the same fundamental parameters which affect the strength, i.e., intergranular bonding, cracks, and secondary minerals. A similar but positive relationship is exhibited with AIVR (Fig 7. 15).

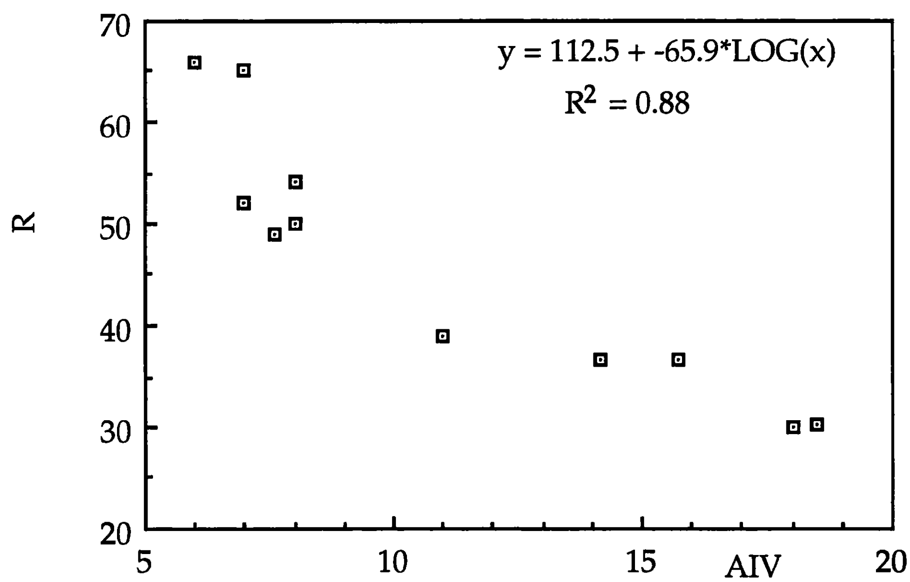


Fig 7. 14 The relationship between the Rebound Number and AIV for basalt

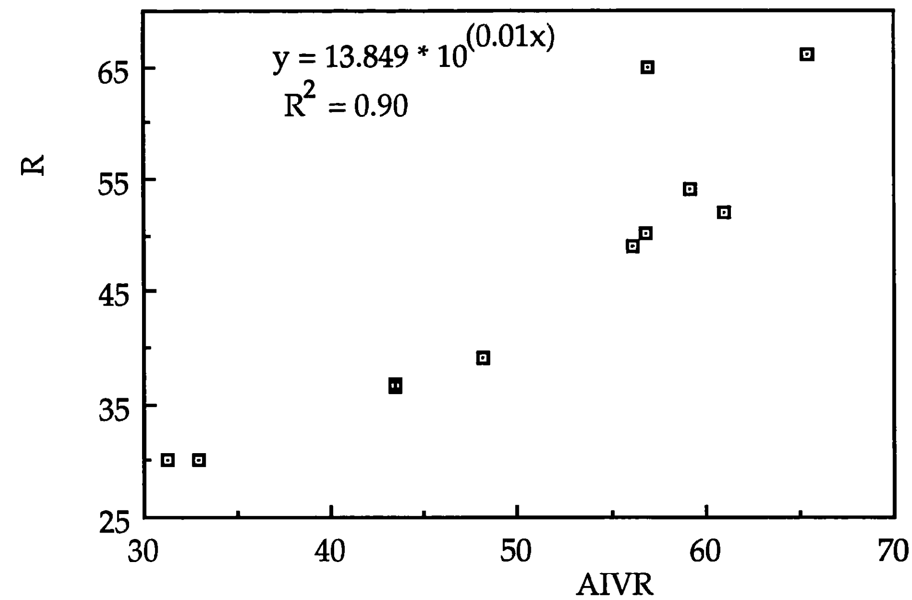


Fig 7.15 The relationship between the Rebound Number and AIVR for granite

Rebound Number - LAAV:

The correlation between R and LAAV does not show good relationship from which one index can be predicted from the other, but it does show a trend where a fall in R corresponds to an increase in LAAV. For an originally strong rock like granite, quartz dolerite, and basalt which in the fresh state display a Rebound Number > 60, a significant decrease in R to values below 40 is the result of a great loss of strength and elasticity. This loss of strength and elasticity is also reflected by a dramatic increase in LAAV (Fig 7.16, 17) indicating poor resistance to abrasion and impact.

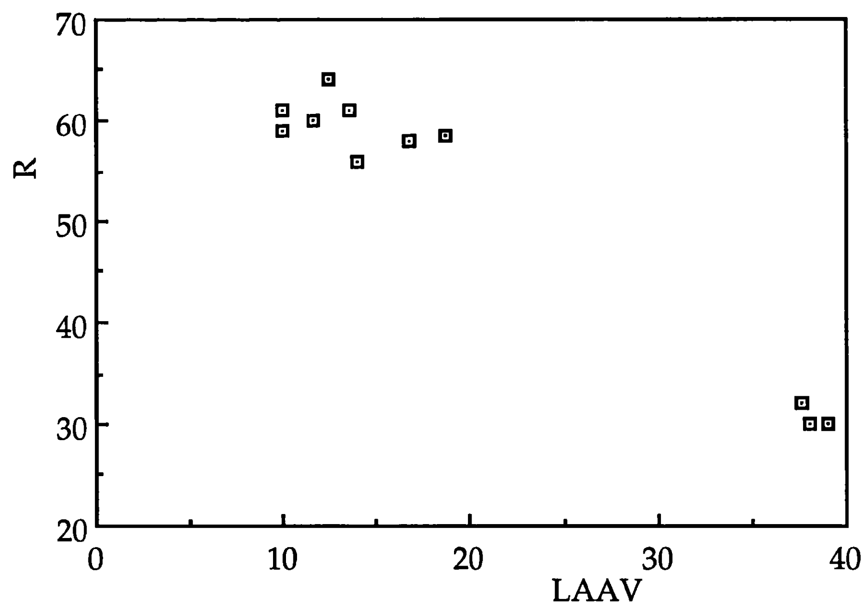


Fig 7.16 The relationship between the Rebound Number and LAAV for quartz dolerite

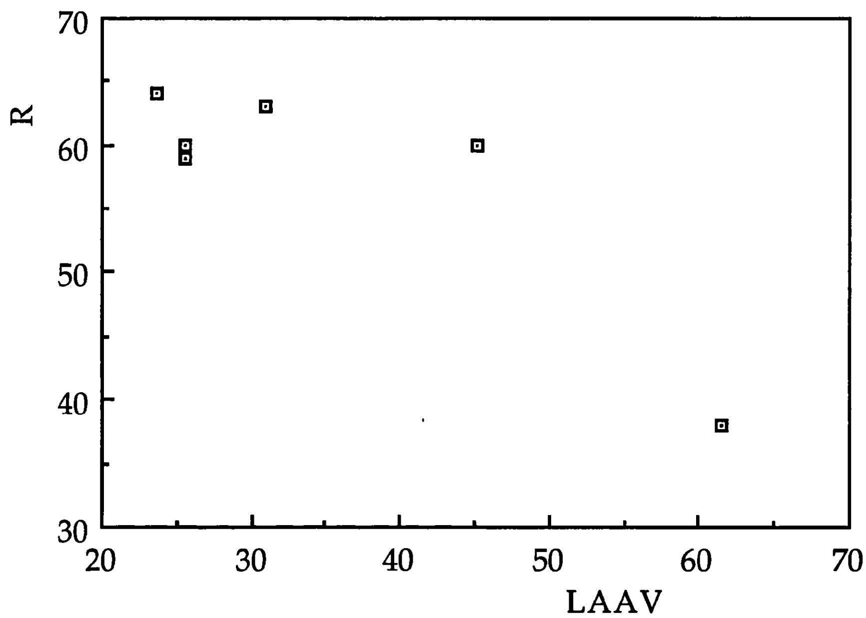


Fig 7. 17 The relationship between the Rebound Number abd LAAV for granite

Rebound Number - AAV:

The correlation between Schmidt Rebound Number and the Aggregate Abrasion Value display different types of relationships for quartz dolerite and basalt. Nevertheless both rock types display an increase in the AAV as the Rebound Number decreases. The fact is that basalt displays mainly a mineral alteration with little or no cracking, while quartz dolerite is heavily cracked in addition to mineral alteration, and it is this which causes gradual loss of strength in the basalt and sudden loss for quartz dolerite. In both cases a Rebound Number of 40 corresponds to an AAV of 8 to 10, characteristic of weakened material (Fig 7. 18, 19).

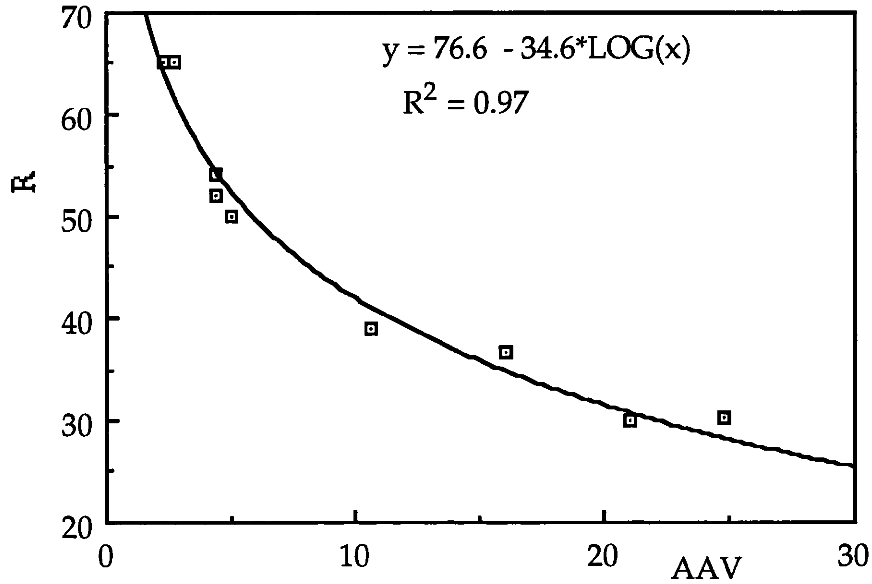


Fig 7. 18 The relationship between the Rebound Number and AAV for basalt

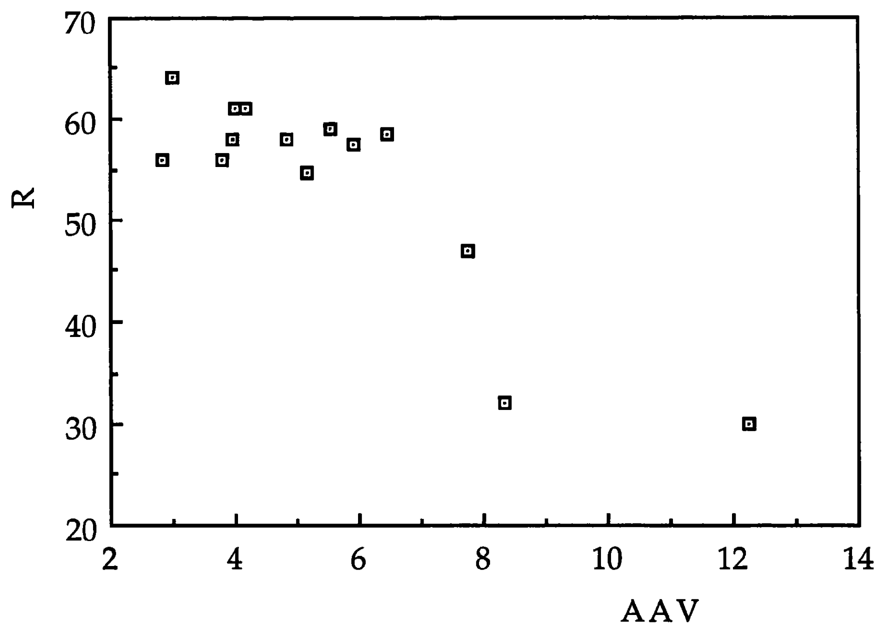


Fig 7. 19 The relationship between the Rebound Number and AAV for quartz dolerite

Rebound Number - Mg SO4:

Although the correlation between R and Magnesium Sulphate Soundness is poor, it does show a clear trend in which aggregates become more vulnerable and deteriorate more upon salt crystalisation as the strength of their source rock deteriorates. Fig (7. 20) shows that below R = 40 - 35, as the rock becomes weak, the value of sulphate soundness dramatically decreases.

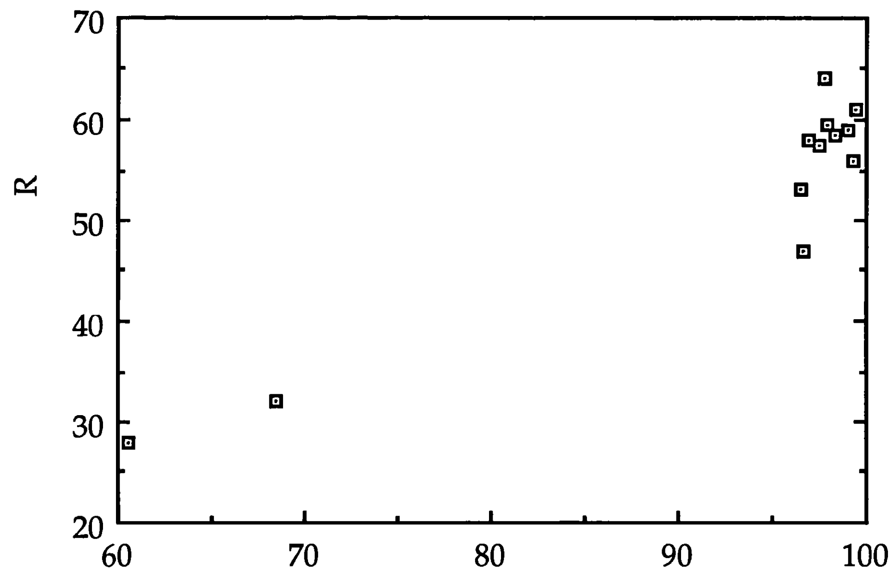


Fig 7. 20 The relationship between the Rebound Number and Mg sulphate soundness for quartz dolerite

Velocity - AIV:

The correlation between The Ultrasonic Velocity and aggregate strength (AIV) displays a logarithmic relationship of the type:

$$Y = a + b \log X$$

a and b are constants

An exactly similar relationship, with better coefficient of correlation is exhibited when dynamic Young Modulus is correlated instead of velocity (appendix IV).

The shape of the curve relating velocity and AIV results from the fact that a marked decrease in velocity occurs in the early stages of weathering with only a small increse in AIV. As weathering progresses, a dramatic increase in AIV occurs while the velocity decreases slowly causing the curve to extend asymptotically to the AIV axis (7. 21). A similar but positive relationship is exhibited with AIVR (Fig 7. 22). In the case of quartz dolerite and granite where weathering is accompanied by heavy cracking the decrease in velocity is abrupt (Fig 7. 23).

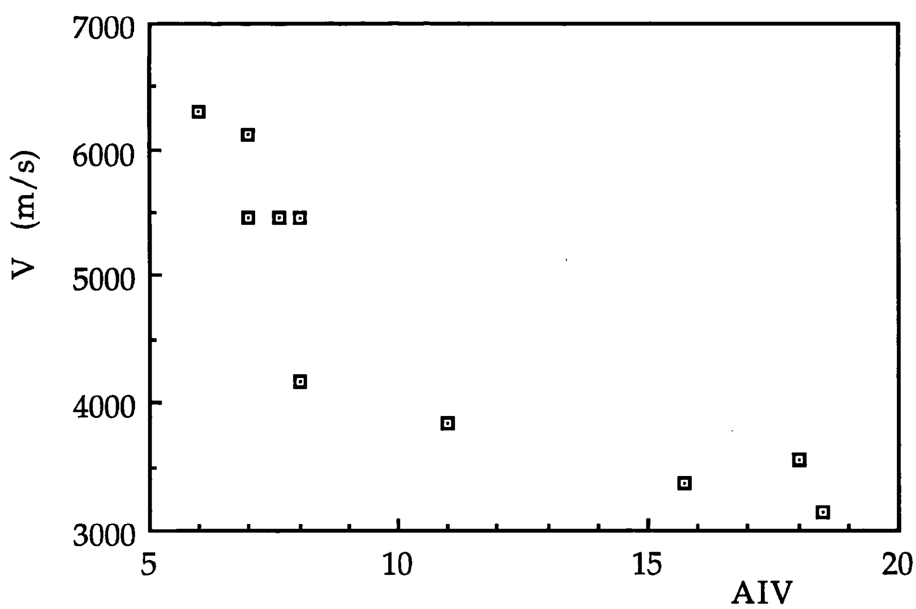


Fig 7. 21 The relationship between Ultrasonic velocity and AIV for basalt

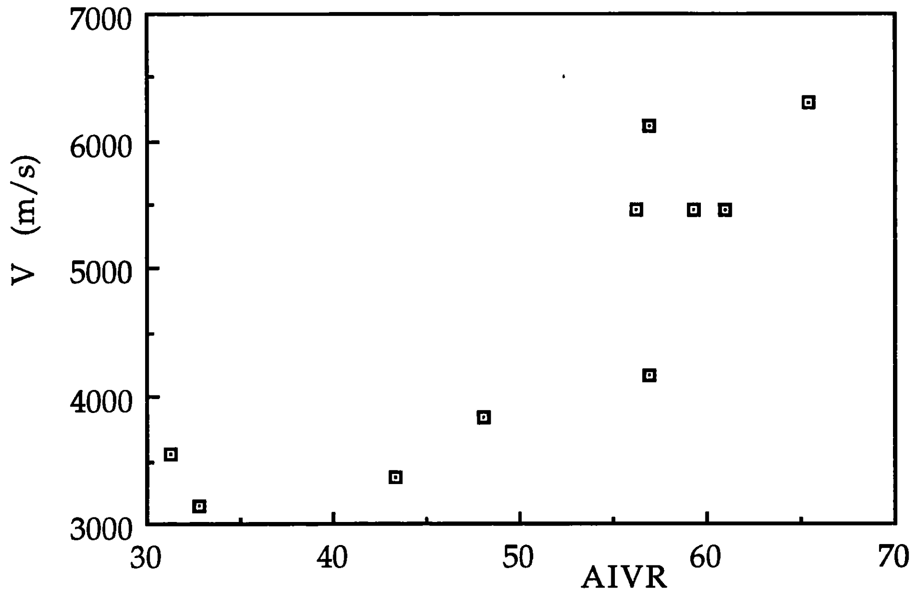


Fig 7. 22 The relationship between Ultrasonic Velocity and AIVR for basalt

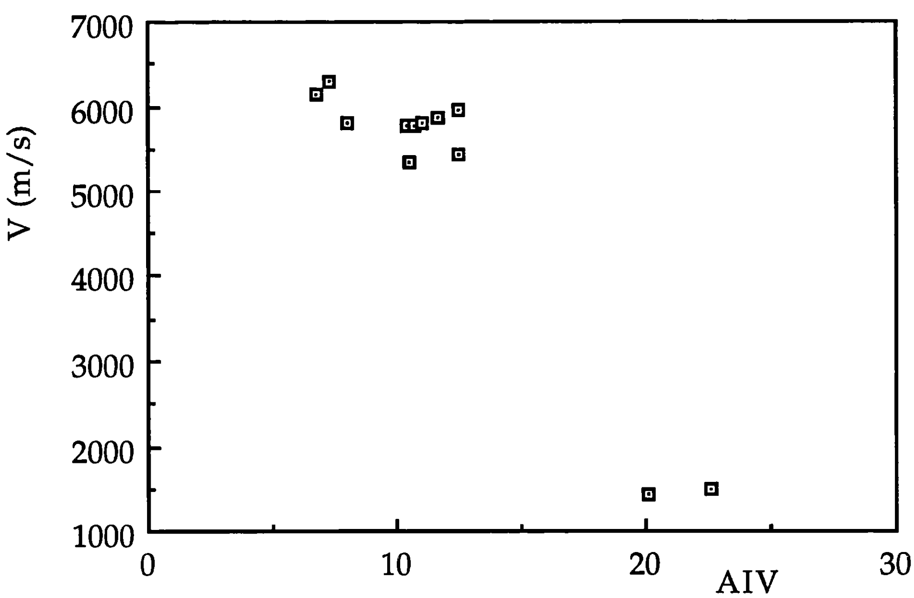


Fig 7. 23 The relationship between Ultrasonic Velocity and AIV for quartz dolerite

Velocity - LAAV:

The correlation between Ultrasonic velocity and LAAV in the case of quartz dolerite and granite show a trend in which velocity decreases as LAAV increases (Fig 7. 24).

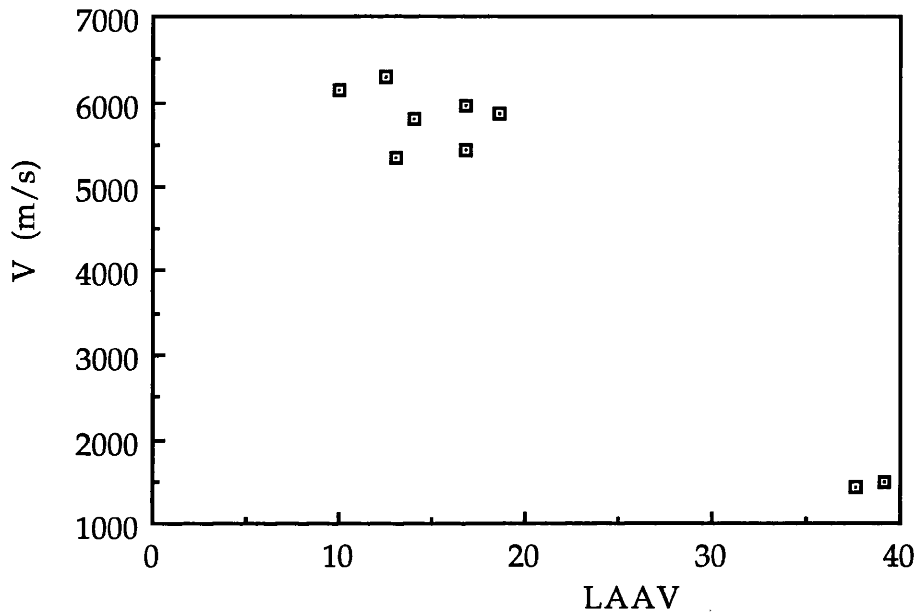


Fig 7. 24 The relationship between Ultrasonic Velocity and LAAV for quartz dolerite

Velocity - AAV:

The relationship between Ultrasonic Velocity and aggregate toughness (AAV) is logarithmic in the case of basalt (Fig 7.25). This is, as seen before, due to the gradational loss of strength and elasticity as weathering progresses.

In the case of granite and quartz dolerite (Fig 7. 26), they display a gentle and steady decrease in velocity in the initial stages, followed by a sharp decrease upon crack opening.

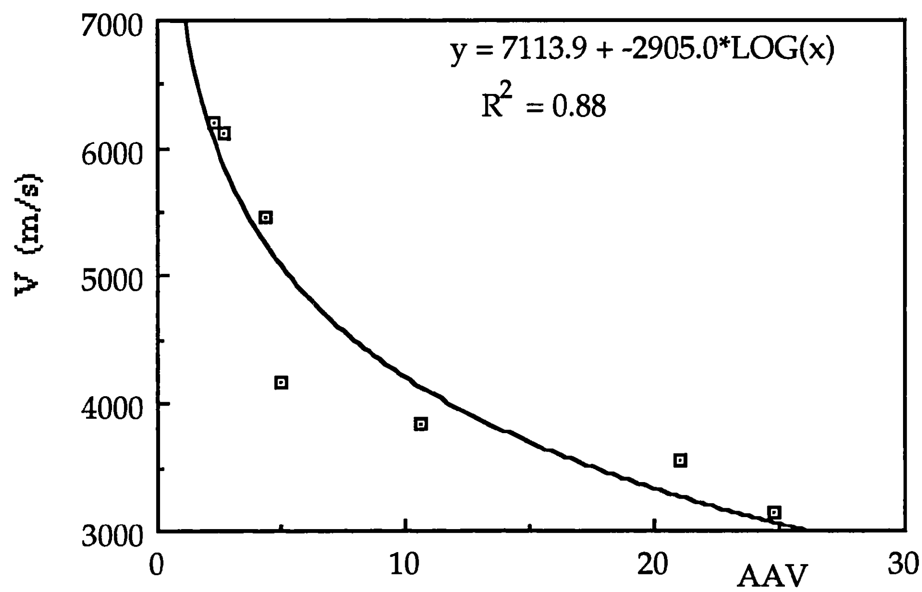


Fig 7.25 The relationship between Ultrasonic Velocity and AAV for basalt

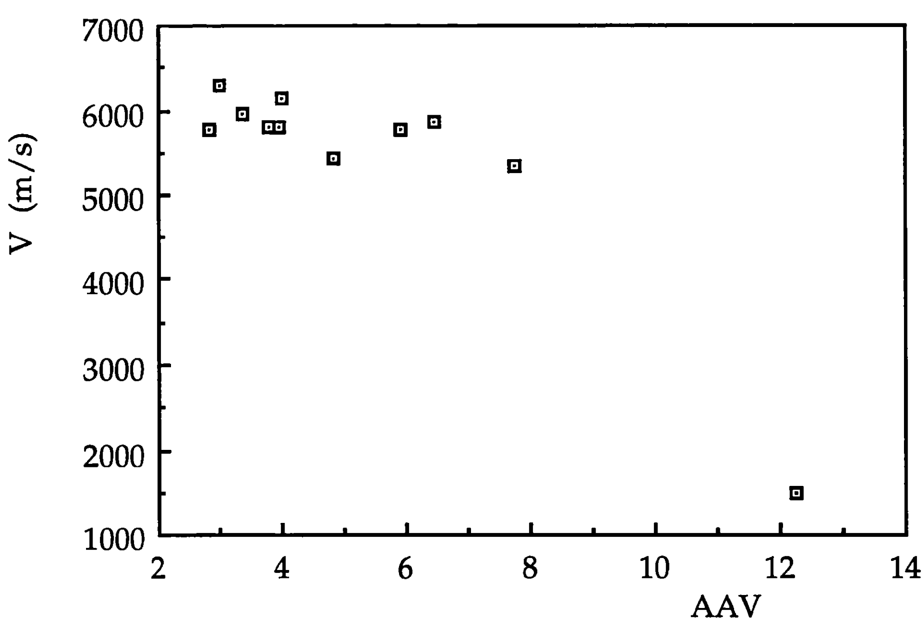


Fig 7.26 The relationship between Ultrasonic Velocity and AAV for quartz dolerite

Velocity - Magnesium Sulphate Soundness:

The graphical representation of the Ultrasonic Velocity Vs Magnesium Sulphate Soundness shows that high velocity rocks display small loss in the Magnesium Sulphate Soundness while those of lower velocity have eventually high loss in the soundness test (Fig 7.27). The principal delineating the two values is in fact the same but the sensitivity of each test to the state of the rock is somewhat different.

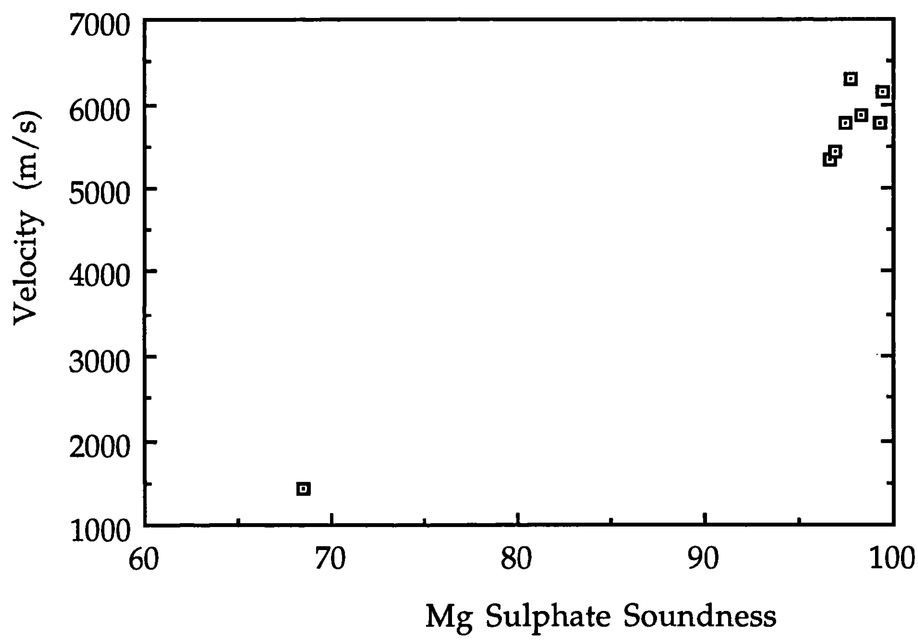


Fig 7. 27 The relationship between Ultrasonic Velocity and Mg Sulphate Soundness for quartz dolerite

Conclusions

The present study is an investigation of the geotechnical properties of intact rock and derived aggregate in terms of the geological nature and the influence of secondary processes such as weathering or alteration. A suite of igneous rocks ranging from volcanic to plutonic, acid to basic, presented a range of texture, grain-size, and degree of weathering, all of which could influence geotechnical properties.

The index properties of intact rock strength included uniaxial compressive strength and Point Load Strength (PLS), while Rebound Number (R) and Ultrasonic Velocity (Vp), despite reflecting elastic characteristics of the rock, served as supplementary or back-up indices of strength.

Weathering is perhaps the most significant geological parameter in terms of it's effect on the mechanical properties of igneous rock, both intact and aggregate. It is expressed by mineral alteration or a combination of mineral alteration and cracks, and can be characterised quantitatively by several indices both petrographical and physical. Petrographic index properties include the Micropetrographic Index (Ip) or the modal volume of secondary phenomena. Physical indices are porosity, water absorption, and specific gravity.

As weathering increases the values of strength and elastic index properties fall. A similar effect attends increase in grain size. The relationship between strength and geological parameters varies from simple and linear to a more complex power-law one. The common non-linear relationship for weathering parameters reflects the facts that secondary mineral growth commonly along grain boundaries with a dramatic effect on intergranular bond strength.

The index properties for strength and tenacity from aggregate derived from the igneous suite, namely Aggregate Impact (AIV), Crushing (ACV) and Abrasion Values (AAV) together with the Los Angeles Abrasion Value (LAAV) were affected by geological parameters in the same way as intact rock. An additional factor was clast shape, in particular the Flakiness Index (I_F).

A significant part of the thesis was an investigation of the Los Angeles Abrasion Test from the viewpoint of the methodology, processes of clast breakage and factors affecting test results. This revealed that the dominant mechanism of communition is one of impact loading, accounting for 80 % of a test value with only 20 % attributable to attrition.

To study the pattern of breakage a non-standard value, the residue value (LAAVR) was introduced. This is the proportion of material in the original size range which survives the test. This value is a better indicator of aggregate quality than LAAV and is more sensitive to the influence of the geological variables.

The path of communition when represented on a triangular diagram, in terms of LAAV - M - LAAVR resembles that established for AIV.

Breakage begins with fracturing of aggregate clasts but very few fines. As the test proceeds this breakage affects a significant proportion of the original clasts, i.e. LAAVR falls sharply, some granulation of the already broken material occurs in the M fraction while a relatively small amount of fines are generated. As the duration of the test is extended beyond the standard limit, a change in the communition path occurs reflecting a sharp increase in fines at the expense of the M fraction, i.e. strong secondary granulation. During this stage the coarser material in the original size range ceases to refine due to buffering by an increasing stream of fines being generated.

Although the choice of 500 revolution for the standard duration of the test was arbitrary it has been shown to be very appropriate.

When the several index properties for aggregate strength are compared the relationship is rational and frequently simple, indicating consistency despite the different loading systems.

Index properties for intact rock also display consistent and rational relationships with aggregate indices indicating that they could be used as indicators of aggregate quality where no source of aggregate exist, i. e. during exploration in undeveloped areas.

This was a pilot study and more work is required to widen the investigation to include sedimentary and metamorphic rocks and the additional geological factors they would introduce. The study has suggested that provided geological parameters are identified and evaluated, the values obtained from index tests are meaningful and in the case of aggregate, permit effective comparison between competing record and rock types. with this some progress has been made towards the intractable problem of predicting "*in service*" performance.

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Appendix to Chapter 4

Los Angeles Abrasion Value Data

<u>Macroporphyritic basalt</u>

I_F	I_{Fr}	Rev No	LAAV	LAAVR	M
0	-	100	3.84	78.82	17.34
0	-	200	7.66	67.43	24.91
0	-	400	15.45	41.25	43.3
0	-	500	19.6	40.25	39.15
0	-	100	39.44	21.53	39.03
20	14.85	100	4.3	76.22	19.48
14.85	10.7	200	8.46	61.48	30.06
10.7	7.7	400	17.5	40.24	42.26
7.7	6.5	500	21.2	33.6	45.2
6.5	1.4	1000	41.4	18.3	40.56
60	45	100	3.8	72.36	23.84
45	35.8	200	8.1	57.56	34.32
35.8	26.05	400	15.9	39.5	44.6
26.05	-	500	20	34	46
-	10.55	1000	39.7	14.8	45.5
100	-	100	6.9	59.8	34.01
100	-	200	10.7	43.33	45.97
I_F	I_{Fr}	Rev No	LAAV	LAAVR	M
100	-	400	19.1	-	-
100	-	500	22.8	16.96	60.24

100	-	1000	44.3	5.4	50.3

Microporphyritic basalt (Orrock)

I_F	I_{Fr}	Rev No	LAAV	LAAVR	M
0		100	4.5	78.8	16.7
0		200	6.1	71.35	22.5
0		400	9.67	59.9	30.43
0		500	11.33	54.93	33.74
0		1000	21.41	38.62	39.97
20	-	100	3	81	26
-	16.45	200	5	70	25
16.45	13	500	13	48	39
13	6	1000	21.67	32	46
33	-	100	2.5	77.3	20.2
	-	200	4.75	65.76	29.49
	-	500	11.82	42.40	52.22
	-	1000	24.95	25.62	49.43
	-	1500	34.97	18.17	46.86
60	42	100	2.9	73.4	23.6
42	39	200	5.4	62	33
39	30	500	14.6	38	47.4
30	16.5	1000	25.5	24	50.5
100		100	3.8	68.5	27.7
100		200	6.8	52.18	41.02
100		400	12.41	32.87	54.72
100		500	15.13	26.15	58.72
100		1000	28.8	13.03	58.18

Microporphyritic basalt (Langside)

I_F	Rev No	LAAV	LAAVR	M
20	100	2.09	83.37	14.54

-	200	3.9	74.22	21.88
-	500	9.55	55.64	34.81
I_F	Rev No	LAAV	LAAVR	M
-	1000	19.68	37.2	43.12
-	1500	44	16.16	39.84

Quartz dolerite

I_F	Rev No	LAAV	LAAVR	M
0	100	2.5	87.5	10
0	200	5	80	15
0	500	11.67	59	29
0	1000	24	38	38
20	100	3	86	11
	200	5.5	75	19.3
	500	13	53	33.76
	1000	26.6	31.76	41.64
60	100	3.4	73.6	23
	200	6.5	59.77	43.73
	500	15.23	35.57	49.2
	1000	31	19.5	49.5
100	100			

Dacite

I_F	Rev No	LAAV	LAAVR	M
0	100	2.35	86.27	11.38
0	200	4.6	77.7	17.7
0	500	9.92	53.9	36.18
0	1000	25.6	33.9	40.5
0	1500	39.25	23.33	37.42
0	2000	52.15	16.34	31.51
60	100	3.2	75.64	21.16
	200	6.3	61.3	32.4
	500	15.86	36.43	47.71

	1000	31.66	19.5	48.84
	1500	41.4	16.83	41.77
	1700	50.85	10.36	38.79
I_F	Rev No	LAAV	LAAVR	M
	2000	58.25	7.75	34
100	100	3.91	67.67	28.42
	200	7.1	52.43	40.47
	500	17.55	24.95	57.5
	1000	33.64	10.4	56.96
	1500	47.24	5	47.76
	1700	50	2.2	41.8

Granite

I_F	Rev No	LAAV	LAAVR	M
25	100	5.25	72.69	21.9
-	200	10.51	56.21	33.28
-	500	26.53	28.35	46.24
-	1000	52.29	10.71	37
	1500	73.6	3.6	22.8

Aplite

I_F	Rev No	LAAV	LAAVR	M
-	100	2	84.86	13.14
-	200	4.35	74.25	21.4
-	500	10.95	52.6	36.65
-	1000	23.5	31.1	45.4
-	1500	36.33	20.6	43.06
-	2000	48.83	10.06	41.11

Los Angeles Abrasion Values on Saturated Material

Macroporphyritic basalt

I_F	I_{Fr}	Rev No	LAAV	LAAVR	M

0	0	100	6.3	74.3	19.4
0	0	200	10.58	57.8	31.62
I_F	I_{Fr}	Rev No	LAAV	LAAVR	M
0	0	400	21.96	38	40.04
0	0	500	26.6	32.5	41.3
0	0	1000	44.6	15.44	41.3
20	14.5	100	5.04	72.26	22.7
	11.5	200	11.64	55.3	33.06
	6	400	21.6	36.65	41.75
	7	500	26.5	31	42.5
	1.4	1000	46.8	14.88	38.32
60	43.6	100	5.8	67.8	26.4
	35.05	200	11.2	49.9	38.9
	24.25	400	20.6	23	49.7
	17.5	500	26.3	23	50.7
	6.3	1000	44.5	11.12	44.38
100	-	100	7	50.86	42.13
100		300	19.23	22.25	58.52
100		400	25	16	59
100		500	29.3	12.31	58.39
100		1000	52.8	3.9	43.3

Los Angeles Abrasion Test without steel balls

<u>Macroporphyritic basalt</u>

I_F	Rev No	LAAV	LAAVR	M
20	100	3.2	88.12	8.68
20	200	5.4	83	11.6
20	400	8.5	76.61	14.89
20	500	9.4	74.9	15.7
20	1000	13.4	69	17.6

Microporphyritic basalt

I_F	Rev No	LAAV	LAAVR	M
0	100	1.14	93.38	5.48
0	200	1.8	92.3	5.9
0	500	3.2	88.99	8.31
0	1000	4.87	85.54	9.59

Quartz dolerite

I_F	Rev No	LAAV	LAAVR	M
100	100	1.5	84.83	13.67
100	200	2.4	81.59	16.01
100	500	6.69	74.95	18.36
100	1000	7	69.65	23.35

Appendix to Chapter 5

Intact rock strength Indices

1- Uniaxial compressive strength

<u>Macroporphyritic basalt</u>

Sample	UCS dry in RT (MPa)	UCS saturated (MPa)
1E	21.5 - 39	25. 66
1N	178 - 191.7	154
1S	48.77- 52.33	40
2S	58 - 68.7	37.71
3N(lw)	192 - 198	147.49
3N(w)	96.75	50.15
3S	70 - 82.54	43.04
4N	90.4 - 100	35.14
6N	207.72 (mean)	171.38
8N	177.5	140.58
O1	346	351.46
O2		299.53
O4	346.15	-
Ln3	388	-
Ln4	399	323

Quartz dolerite

Sz2	112.88	-
St2	172.48	-
St3	211.16	-
St4	165.57 - 171.5	-
St6	212.5	-
St8	197.94	-
St9	166.96 - 175.84	-
St11	261.5	-
St12	160.25	-
St13	175 - 211	-
St16	192.3	-
St18	174.65 - 181.76	-
St19	151.96 - 192.02	-
St20	167.35 - 192.22	-

Sample	UCS dry in RT (MPa)	UCS saturated (MPa)
St21	126	-
St22	159.85 - 176.82	-
St23	143.7 - 163.23	-
St24	160 - 163.21	-
St25	122.76 - 174.26	-
St26up	220.25	-
St26l	154.52	-

Granite

Sample	UCS dry in RT (MPa)	UCS saturated (MPa)
Gd1	191.03 - 182.94	-
Gd2	74.99	-
Gd3	178,6	158.35
Gd4	107.35	75.82
Gd5	175.05	143.5
Gd6	69.09	76.6
Gd7	195.97	-
Gd8	78.74	49.36
Gd9	229.12 - 237.61	-
Gd10	257.34	-

Dacite

Sample	UCS dry in RT (MPa)	UCS saturated (MPa)
ba1	223	-
ba2	205 - 295.19	160.05 - 174.06
ba3	149.86 - 276.3	282
ba6	335.5	-
ba5	273.53	-
ba7	158.35 - 237.4	120.58
ba8	218	175
ba9	214.7	136

2- Point Load Strength Index and Schmidt Rebound Number:

Sample	PLS (MPa)	R
1E	1.29 ± 0.62	28 - 32
1N	6.19 ± 1.63	48 - 53
1S	2.64 ± 0.91	34 - 39
2S	2.26	37 - 38
3N(lw)	6.25	50 - 54
3N(w)	-	46 - 48
3S	4.58	38 - 40
4N	3.32	36 - 39
6N	6.18	48 - 50
8N	7.73	53 - 56
9N	8.14	50 - 52
4Nb	-	50 - 52
8Eb	-	39 - 42
9Eb	-	48 - 51
O1	10.26	62 - 66
O2	8 - 9.5	
O4	8.10 - 11.3	63 - 66
Ln3		64 - 67

Quartz dolerite

Sample	PLS (MPa)	R
Sz2	7.82	47
St2	5.09	53
St3	10.19	59 - 62
St4	8.38	56 - 58
St5	9.8	61 - 63
St6	11.18	58 - 60
St8	10.34	59
St9	10	59.5
St11		59
St12	10	58
St13	9.5	56
St16	10.62	56 - 59
St18	11.2	58.5

Sample	PLS (MPa)	R
St19	12.11	62 - 64
St20	11.74	60
St21	6.14	48
St22	9.5	60
St23	12.32	54.5
St24	8.38	56
St26up	9.5	61
St26l	7.05	58.5
St30	1.17	30
St31	1.4	32
St32	1.3	31 - 33
Stw	1.7	28

Granite

Sample	PLS (MPa)	R
Gd1	12.29 ±	59 - 62
Gd2	6.48 ± 2.01	48 - 52
Gd3	-	62 - 64
Gd4	3.93 ± 1.67	42 - 45
Gd5	7.97 ± 2.38	58 - 60

Gd6	1.86 ± 0.36	37 - 39
Gd7	9.16 ± 2	58 - 59.5
Gd8	3.94 ± 0.8	39 - 41
Gd9	10.6 ± 3.8	62- 64
Gd10	13.15 ± 0.63	60 - 64

Dacite

Sample	PLS (MPa)	R
ba1	7.4 - 8	54 - 58
ba2	7.5 - 7.82	54 - 58
ba3	8.3 - 9.7	54 - 57
ba4	8.1 - 9.6	54 - 57
ba5	8.5 - 9.5	56 - 58
ba6	9.5 - 10.5	55 - 56.5
ba7	7.54	54 - 56
ba8	8.1 - 10	50 - 54

Sample	PLS (MPa)	R
ba9	6.7 - 9.7	54 - 58

Ultrasonic Velocity

Sample	V air dry (m/s)	V sat (m/s)	Edy (GPa)
1N	4174	4246	49.89
1S	3555.40	3812	34.41
2S	3379	3531	31.39
3N(lw)	5462	5378	83.78
3S	3844	4045	51.41
4N	3726	3830	39.57
6N	5462	-	84.71
8N	5459	5370	86.40
9N	5195	5306	81.14
O1	6165	6105	11.74
O4	6253	6253	116.12
Ln3	6208	6056	112.53
Ln4	6445	6306	123.78

Quartz dolerite

Sample	V air dry (m/s)	V sat (m/s)	Edy (GPa)
Sz1	5587	-	-
Sz2	5352	-	81.92
St4	5422.5	-	78.21
St11	6111	-	105.68
St13	5769.3	-	92.53
St16	5767	-	95.78
St18	5794.9	-	96.04
St19	6292	-	115.20
St21	6049.6	-	126.19
St22	5973	-	-
St24	5787.5	-	93.45
St25	5957.5	-	96.89
St26up	6133	-	107.57
St26l	5853.6	-	97.99

Sample	V air dry (m/s)	V sat (m/s)	Edy (GPa)
St31	1416	-	5.39
St32	1500	-	5.96
Str1	5028	4960	73.06
Str2	5249	6247.5	80.72
Str3	5366.5	5287	83.8
Strw	4971	4871	66.47

Granite

Sample	V air dry (m/s)	V sat (m/s)	Edy (GPa)
Gd1	5299.4	5145.34 (oven dry)	74.7
Gd2	4137.6	-	42.3
Gd4	4133.6	-	44.42
Gd5	4974.22	4595.3 "	64.33
Gd6	2893.89	2452.3 "	21.43
Gd7	4410	4419.35 "	51.14
Gd8	2738.88	2722 "	19.20
Gd9	4398.81	4357.8 "	51.08

Dacite

ba1	4666	4553	54.86

ba2	4655.17	4481	54.39
ba3	5297	5204	71.26
ba4	4798.91	4623	57.57
ba5	5350	5165	70.98
ba8	4683.41	4568	54.17
ba9	4790.32	4588	56.67

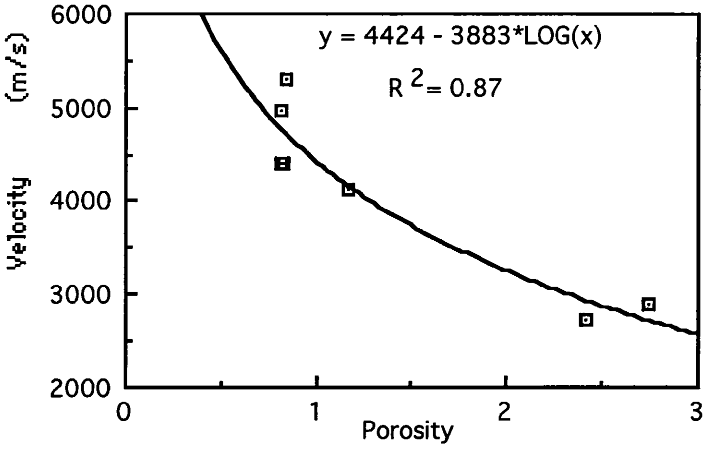

Fig . The influence of porosity on Ultrasonic velocity for granite

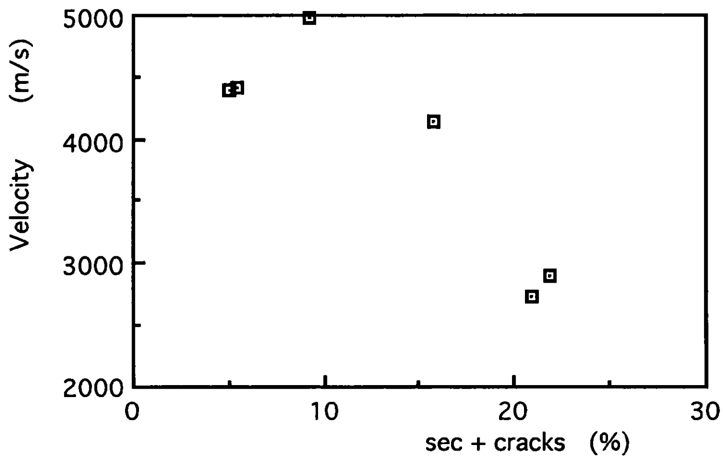

Fig . The influence of secondary minerals and cracks on the ultrasonic Velocity of granite

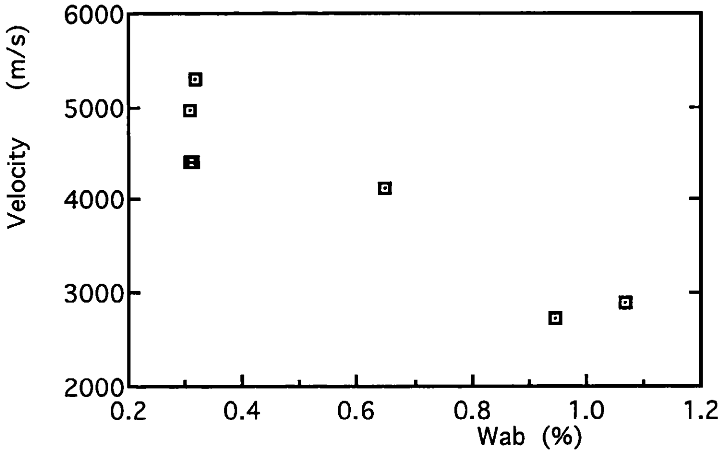

Fig . Graph showing the relationship between velocity and the water absorption of granite

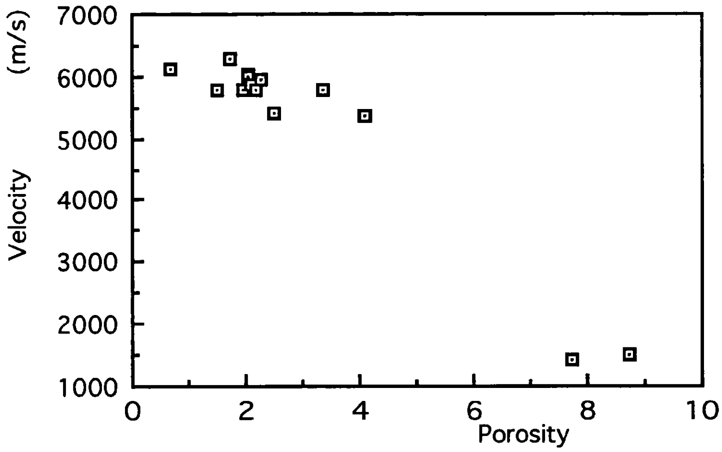

Fig . The relationship between velocity and porosity for quartz dolerite

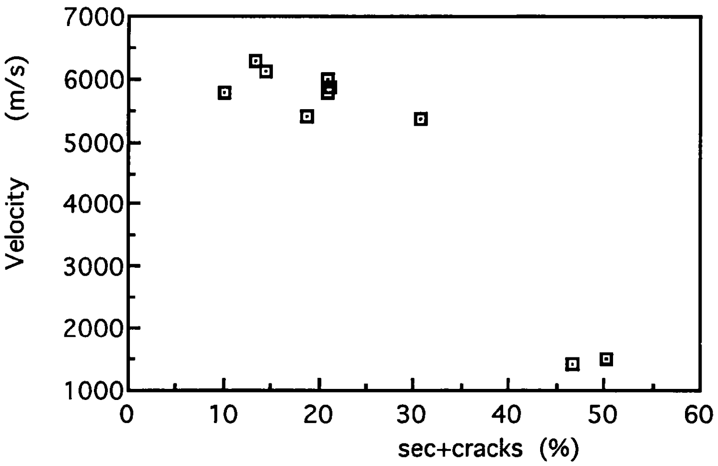

Fig ( ) The influence of secondary minerals and cracks on the Ultrasonic velocity of quartz dolerite

Appendix to Chapter 6

Aggregate Index Tests

Aggregate Impact Values

Macroporphyritic basalt basalt

Influence of the nature of the floor an AIV

Wooden floor			Concrete floor		
I_F	AIV	AIVR	I_F	AIV	AIVR
0	15 - 16	36 - 37	0	21 - 22	30 - 30.8
20	16 - 16.5	27 - 34	20	21.5 - 22	26 - 27
60	17 - 18	24 - 30	60	20 - 23	17 - 17.5
100	18. 20.5	15 - 17.5	100	24 - 24.5	10.5 - 11

Evolution of AIV with loading

Blows No	IF	AIV	M	AIVR
1	0	1.2 - 1.12	9.8 - 10	88.5 - 89
3	0	4.2 - 4.7	22.8 - 25	70 - 73
6	0	8 - 8.2	36.6 - 36.8	55 - 55.5
8	0	8.8 - 9.8	38.2 - 42.4	48 - 52.3
10	0	12 - 13.2	40.85 - 443.4	43.4 - 46.4
15	0	16 - 16.3	40.6 - 41.8	36 - 43
1	45	1.5 - 2.1	16 - 24.5	74 - 82
3	45	5.4 - 5.8	44.5 - 50	44 - 50
5	45	8.4 - 10	44 - 50	41 - 47
10	45	16.8 - 18	54 - 60	22.8 - 29

15	45	20 - 24	54 - 63	21 - 17
20	45	24.5 - 27	53 - 58	17.5 - 19.7
25	45	28	55.83	17
Blows No	IF	AIV	M	AIVR
50	45	37 - 40.5	48 - 49.5	10.74 - 12.53
1	100	1.77 - 2.5	26 - 39	58 - 66
3	100	5.2 - 5.5	47.5 - 52	43 - 47
5	100	8.3 - 9	55.6 - 59	32 - 36.7
10	100	14.3 - 15.1	62.5 - 66	18 - 23
15	100	20 - 20.8	67 - 69.5	9.8 - 13
25	100	24.5 - 25.2	63 - 65	10.7 - 12
50	100	29.6 - 34.6	57.5 - 59	6.2 - 11.25

Influence of I_F on AIV and AIVR

Beith			Orrock			Langside		
I_F	AIV	AIVR	I_F	AIV	AIVR	I_F	AIV	AIVR
0			0	6	58.5	0	5	71
20			20	7	51	20	5.5	60.5
60			60	9	39	60	-	-
100			100	12	21	100	9	27

Quartz dolerite			granite (Nairne)				dacite	
I_F	AIV	AIVR	I_F	AIV	AIVR	I_F	AIV	AIVR
0	7.6	63	0	13.8	46	0		
20	8	58.2	20	14	46	20	9	49
60	11	38	60			60		
100	12.5	25.5	100	20.55	19.26	100		

Influence of I_F on ACV and ACVR

basalt (Beith)

I_F	ACV	ACVR
0	24	31.5
20	-	-
50	24	24
100	25	12

basalt (Orrock)

I_F	ACV	ACVR
0	12.44	38.7
20	13.31	35.2
60	15.31	27
100	17.56	17

dacite

I_F	ACV	ACVR
0	14.44	39.44
20	15	37
60	16.9	26.55
100	18.25	19

Quartz dolerite

I_F	ACV	ACVR
0	14.5	38
20	15	38
60	17	26.6
100		

granite (nairn)

I_F	ACV	ACVR
0	22.13	31
15	22.2	31.5

Influence of water saturation on aggregate strength

	Air dry		Saturated		Oven dry	
	AIV	AIVR	AIV	AIVR	AIV	AIVR
basalt	15 - 16		20 -21		11 - 12	
Orrock	6	58	6	59	6	61
Langside	5 - 6	66 - 71	5 - 6 (MAIV)-		-	-
qz dolerite	7. 5	63	7.5	58 - 60	5.5 - 6.3	61 - 67
dacite	8 - 9	47 - 53	8 - 9	48 - 50	-	-
granite	13 - 14	45 - 47	12 - 14	44 - 47	13 - 14	42 - 46

Modified Aggregate Impact Tests

Quarry product aggregates

Rock type	Blows No	MAIV
qz dolerite	5	7.66 - 8.1
dacite	5	8.5 - 9.19
granite	5	15.6 - 18
basalt (B)	5	22 - 23.4
basalt (O)	5	6 - 6.3
basalt (L)	5	5.7 - 6.6

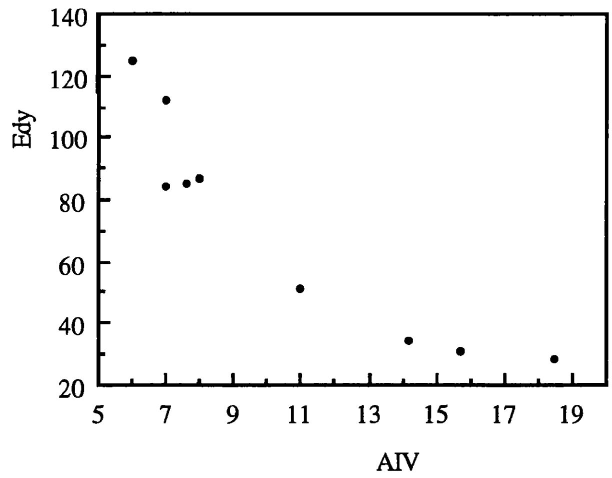

Fig ( ) The relationship between dynamic modulus of elasticity and aggregate impact value for basalt

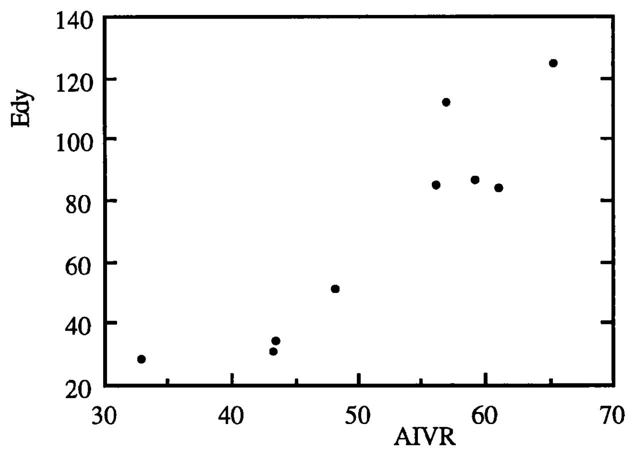

Fig The relationship between Dynamic Modulus of Elasticity and Aggregate Impact Residue Value

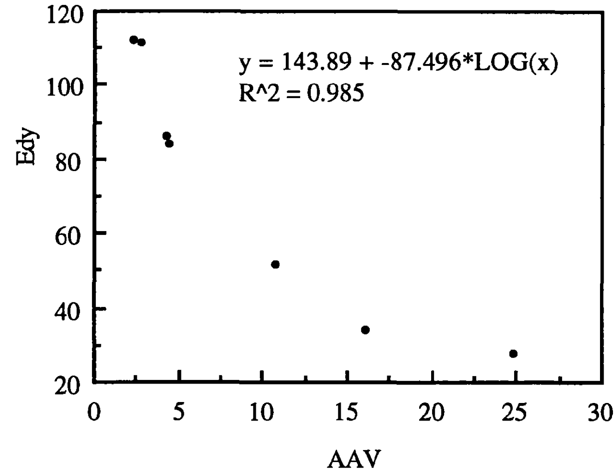

Fig ( ) The relationship between dynamic young modulus and Aggregate Abrasion Value for basalt

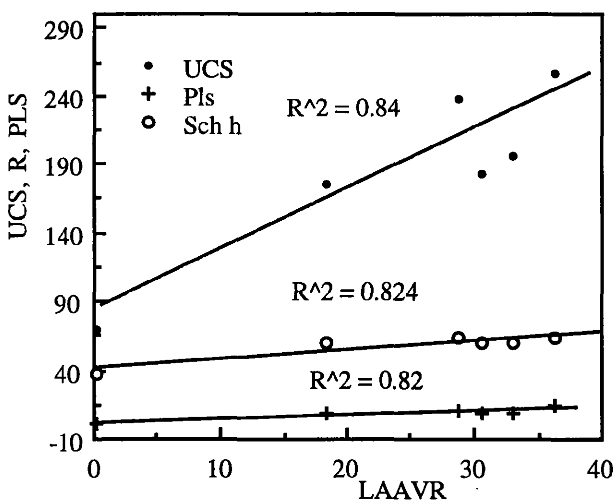

Fig The relationship between UCS, R, PLS and LAAVR for granite