Weathering and brittleness in shale fill dams and embankments

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WEATHERING AND BRITTLENESS IN SHALE FILL DAMS AND EMBANKMENTS.

by

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Abstract

Research and industrial concern has been expressed about the behaviour of mudrocks when used in engineering structures. In particular the loss of strength in shale fills caused by chemical weathering processes and by progressive failure arising from overstressing and brittleness is an important consideration in design.

A literature review into use of the term 'brittleness' to describe this loss of shear strength has revealed infrequent references, but those that were found refer to a wide scope of failures including liqufaction events and long term slope stability problems. The use of large size shear box equipment has been reviewed as an appropriate method of testing the brittleness of shale fills in the laboratory.

A range of materials, from fresh shale to shales retrieved from dams of different ages in the north east of England, has been collected for shear strength testing and for chemical and mineralogical examination to determine the condition of the fill.

Research has concentrated on such rocks as it was possible to obtain. Although the samples have not exhibited wide lithological and geotechnical variations the work has highlighted several significant features including the need for resolving appropriate methods of testing. The results of the work were generally in accordance with earlier research on Carboniferous rocks by suggesting similar material trends in, on the one hand, Namurian shale dams and associated spoil heaps and, on the other hand, colliery spoil heaps of Westphalian shale as had been previously observed.

Ackowledgements

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Chapter 1. INTRODUCTION

1.1 Failures in Shale Fill Structures

1.1.1 Carsington Dam.

The collapse of Carsington Dam in June 1984 generated a great deal of interest in the mechanisms of shear failure in earth fill dams and embankments. In the final report to the Department of the Environment in June 1986 it was concluded that the eventual collapse was through progressive failure caused by unusual geometry of the cross-section and by the brittleness of foundation and fill materials (Coxon, 1986). The location of Carsington Dam is shown on Figure 1.1, with the positions of other sites mentioned in the forthcoming chapters.

Figure 1.2 shows the general geology in the area of Carsington Dam. The bedrock comprises Upper Carboniferous mudrocks which were found to be glacially brecciated, crumpled and weathered to a depth of several metres. This upper layer of weathered mudstone plus the head (*that is* solifluction deposits) which together became known as the 'Yellow Clay' contained many shear surfaces, some up to several metres in length. Prior to construction, alluvial deposits along with top and sub-soil were removed so that the dam was founded either directly on the weathered mudstone as in the valley floor in the area of chainage 850m * , or elsewhere on the Yellow Clay, for example, at chainage 725m. Local materials were used in the construction of the dam which began in July 1982. The rolled central

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* The distances along the dam from the north end are measured in metres and referred to as chainage.

core with its boot-shaped upstream extension was constructed from weathered clay as shown in the cross sections in Figure 1.3, and the shoulders from blocky and brecciated mudstones compacted at their optimum moisture content with the finer grained material being used for the inner Zone I, and coarser material for the outer parts of the shoulders *that is:* Zone II.

On 4 June 1984, when the dam was one metre short of its intended final height, a crack became visible along part of the crest near chainage 725m. Horizontal survey peg measurements indicated a horizontal rate of displacement of 25 to 30 mm per day as a slip started on the upstream bank. Until that time piezometers and vertical settlement gauges had shown nothing untoward, although some concern had been expressed about a possible lack of strength in the Yellow Clay. The first movement followed a weekend of heavy rain during which no fill had been placed. On subsequent days observation of survey pegs indicated higher rates and a wider extent of horizontal movement as the slip spread to the valley area. The movement was finally arrested by 7th June following construction of an emergency berm at the base of the upstream slope.

Early observations in the area where first movement had occurred showed the position of the slip surface running through the core and boot, then through the Yellow Clay and thrust over the small rock fill toe. In the flood plain area at chainage 850m, where the Yellow Clay was absent, the slip passed through the Zone II fill. The dam itself was left with a near vertical backscarp approximately 10m deep at the crest and large blocks of the upstream face back rotated as thrust – graben features formed in the tension zone above the failure plane (Coxon, 1986). Figure 1.3 shows the cross section at two points (chainage 725 and 850m), before and after the failure.

Following a two year enquiry, an out of court settlement of £3.25M was made although had the case gone to court it was expected that the owners, Severn Trent Water Authority, would have claimed in the region of £50M from the designers (New Civil Engineer, 12 May 1988). In the meantime, the completion of the dam has been delayed by eight years (New Civil Engineer, 21 May 1992), and the total cost has nearly tripled as the dam has had to be redesigned with flatter slopes and double the volume of material (New Civil Engineer, 5 January 1989).

No loss of life resulted directly from the Carsington failure, which although progressive in origin was essentially rotational in style. As Bishop (1973) pointed out, rotational slips may damage property and services but, unlike flowslides, rarely cause loss of life. Problems did arise on the failed dam due to the presence of sulphuric acid generated during pyrite oxidation. This reacted with limestone drainage blankets to produce carbon dioxide which caused the death of men working in trial pits. The acid has also polluted run-off from the dam to such an extent that ponds are required to neutralize run-off before it can be allowed back into the local drainage system.

1.1.2 Aberfan

Eighteen years earlier, events at Aberfan in South Wales were to have a profound effect on both waste tip design and on Engineering Geology as a whole. Colliery spoil tips adjacent to the village had been formed by loose tipping of discard from cranes or Maclane tippers, and had been subject to minor slides on previous occasions, November 1944 and October/November 1963. On the morning of 21 October 1966, workers arrived to find a significant settlement in the crest of the tip, which was followed a couple of hours later by observed movement at the toe. Coarse discard from the the tip flowed down a 12.5° slope at an estimated $16 - 32kmh^{-1}$ (Taylor, 1985) engulfing the village school and several houses and killing 144 people. The flowslide travelled a distance of about $600m$ before coming to rest and excavations later revealed a depth of $10m$ at the toe. A layer of impervious boulder clay and head deposits effectively sealed the underlying fissured sandstone and thus gave rise to substantial artesian pressures. A small rotational slip at the base of the tip, thought to be an adjustment in response to heavy rain, caused the boulder clay to rupture and allowed water to flow from the sandstone. This water flowed into the loose tip material and was able to carry the debris in the manner described earlier. A ruptured water main exacerbated the situation, and the abundance of water gave the flow the appearance of a mudflow in places. Alleviation of the high porewater pressures caused a reduction of an estimated 18×10^3 gallons of water in the sandstone. Investigations revealed a well defined shear plane with an angle of friction approaching its residual value. Adjacent material, both above and below the shear plane was relatively undegraded (Bishop, 1973).

1.2 Summary and Ideas for Research

Initially, the events at Aberfan and Carsington appear to have little similarity other than that they are both forms of downslope mass movement. Aberfan caused loss of life although Carsington did not, and its very nature meant that it was unlikely to. These two cases illustrate the principal differences between flowslides and rotational slips – speed and extent of material movement. However, the mechanism behind the two types of slide may have some similarities. These are discussed later, in Chapter 5, with respect to brittleness.

Shear failure in water-retaining structures appears to be a relatively rare occurence, especially whilst the dam is in service, but when it does happen, can cause great inconvenience and financial cost. There are many well-documented examples of shear failure in tips of various degrees of magnitude. Since such structures are widespread, and commonly near or upstream of populated areas, safety is of paramount importance, with this becoming more important as the sizes of structures and therefore their potential to do damage, increases throughout time. Mistakes in the past have influenced present design criteria – hydraulically placed puddle clay cores and cut-offs which are particularly prone to internal erosion are no longer used.

Questions do arise however, about the type of material employed. 'Problem' shales have been discussed in the literature, particularly with respect to foundation engineering and land stability. It is essential to identify such materials at an early stage of design of earth fill dams.

1.3 Outline of Project

1.3.1 Reasons for Project Work

It is clear from the literature that a knowledge of the behaviour of shale fills is essential if they are to be used in construction. In particular, the relationship between the degree of weathering, the condition of the fill, shear strength of the material and the original geology may be critical, especially when viewed in connection with progressive failure.

Previous research at Durham had highlighted several curious aspects of mudrock behaviour which are outlined below:

1. Large shear box testing, albeit limited, suggested that the brittleness (drop in strength) of certain marine shales was both greater and more rapidly developed than that in non-marine mudrocks. Under identical test conditions, Fytis (1986) for example, found that Namurian shales from the Carsington site exhibited a higher Brittleness Index * than a fresh Kimmeridge Clay, Fresh Coal Measures Shale and weathered Kimmeridge Clay, respectively. This implies that weathering may have a significant effect on the probability of failure, and that geological age is not important.

2. The terms Brittleness and Brittleness Index appear to have been used somewhat loosely and it was felt that a review of their use in the literature would be an aid to producing a clear, workable definition. It would then be interesting to determine whether it would be useful

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* Brittleness Index is the difference between peak and residual shear strengths expressed as a percentage of the peak strength (Bishop, 1967)

to measure this parameter in fill materials. An example of a situation where quantitative estimates of brittleness are essential is in finite element analysis where the failure of each individual soil element must be modelled.

3. Further Large Shear Box Tests on a remoulded sample of fill from Carsington Dam, originally dug from near the dam's surface and remoulded at natural moisture content yielded peak shear strength parameters of $c_p = 0$, $\phi_p = 22°$ and residual parameters of $c_p = 0$, $\phi_p = 16.5 - 17°$. However, Skempton and Coats (1985) quote peak values of $\phi_p = 28°$ in freshly compacted shale and $\phi_p = 25°$ for shale in the dam. This drop of 3° was thought to be an indication that additional processes were acting on the outer zones of the embankment which had the overall effect of reducing shear strength. The Durham sample reached a residual strength after a smaller than expected displacement. This was attributed to weathering and degradation of the sample during placement and exposure. Considerable amounts of sulphate were observed on the shear plane after only 48 hours exposure in the laboratory at 20°C, which demonstrates the rapid nature of these weathering processes (Taylor, in Coxon 1986). These results are summarized in Figure 1.4.

It was suggested (Taylor, 1986 *personal communication*) that there are two possible routes of reducing a material's shear strength to a critical state (not necessarily critical state in a soil mechanics sense). Firstly by physical and chemical weathering, and secondly by overstressing, for example, within an embankment zone. These processes may act singly or in unison. There are indications that marine and non-marine mudrocks may show different patterns of behaviour in both respects.

1.3.2 Materials and Testing

Originally, it was intended to explore these physical and chemical processes in marine and non-marine mudrocks using a combination of fresh borrow pit material and fill from embankments of different ages. The search for appropriate sites was somewhat problematic. This is outlined in Chapter 3 where the sites are described in more detail.

The work began with two 'standard' materials in order to study the process of shear strength reduction during testing. Samples were collected from an embankment under construction, a scale model of a dam built as part of a project in Engineering Geology some years previously, from five earth fill dams with ages between 25 and 118 years and two uncompacted spoil heaps. These were subjected to the following tests:

1. Physical characteristics (particle size analysis, Atterberg limits, specific gravity, durability)
2. Mineralogical and chemical composition.
3. Shear strength characteristics, primarily using shear boxes, but with subsidiary triaxial and ring shear testing.

It was hoped to build up a picture which would demonstrate the condition of the fill in older earth fill dams and to show whether degradation had caused any significant loss in strength.

1.3.3 Structure of Thesis

Chapter 3 presents a general summary of the relevant aspects of shale petrology, and includes a discussion on the controversy surrounding the definitions of the terms shale and mudrock. Geological and physical testing are described in Chapter 4, with the implications of weathering. The definition, use and significance of the Brittleness Index is examined in Chapter 5 and the shear strength results presented in Chapter 6 along with a discussion of the problems of shear box testing. The results are integrated and discussed as a whole in Chapter 7.

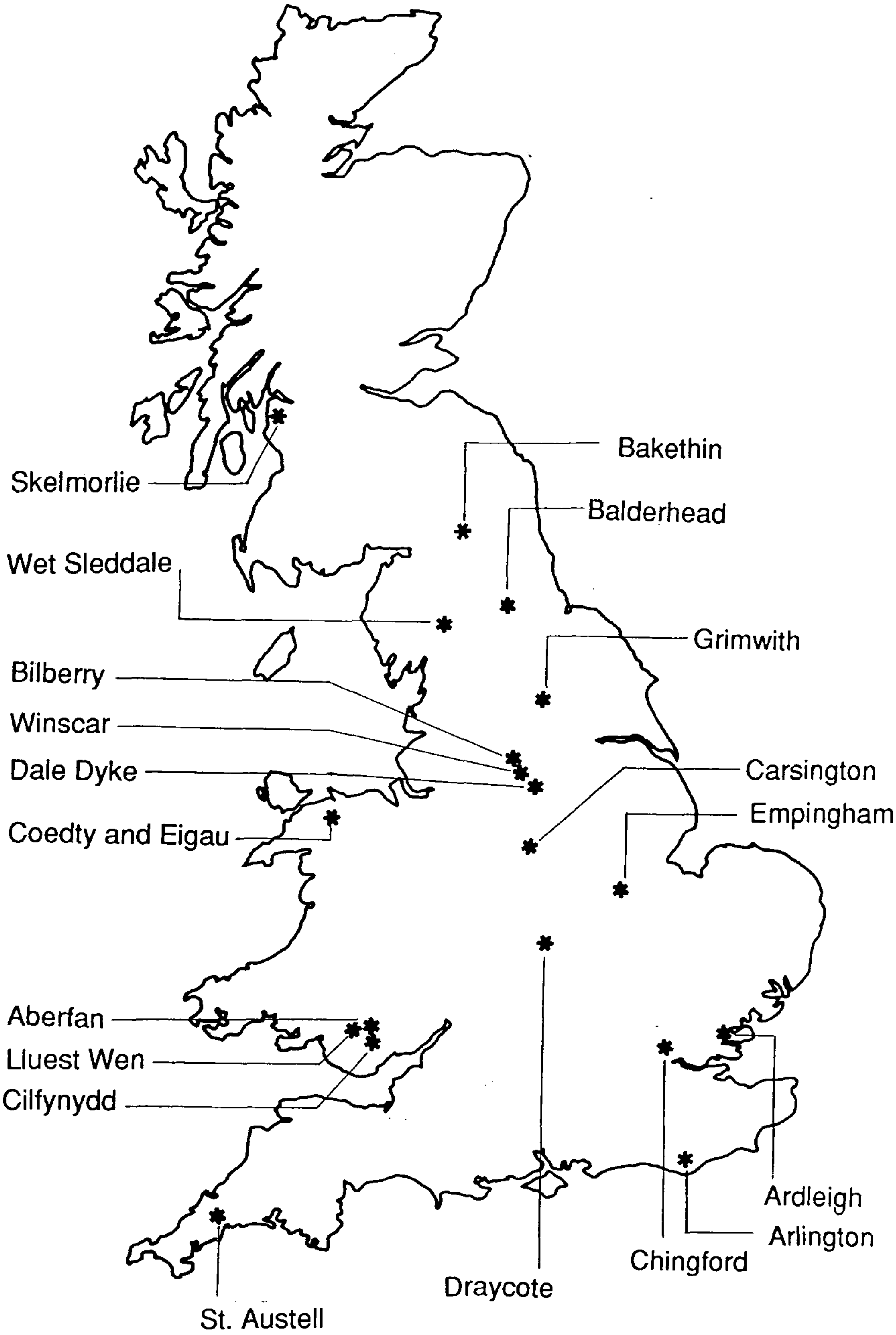


Figure 1.1 Map of Great Britain showing locations of dams mentioned in Chapters 1 and 2.

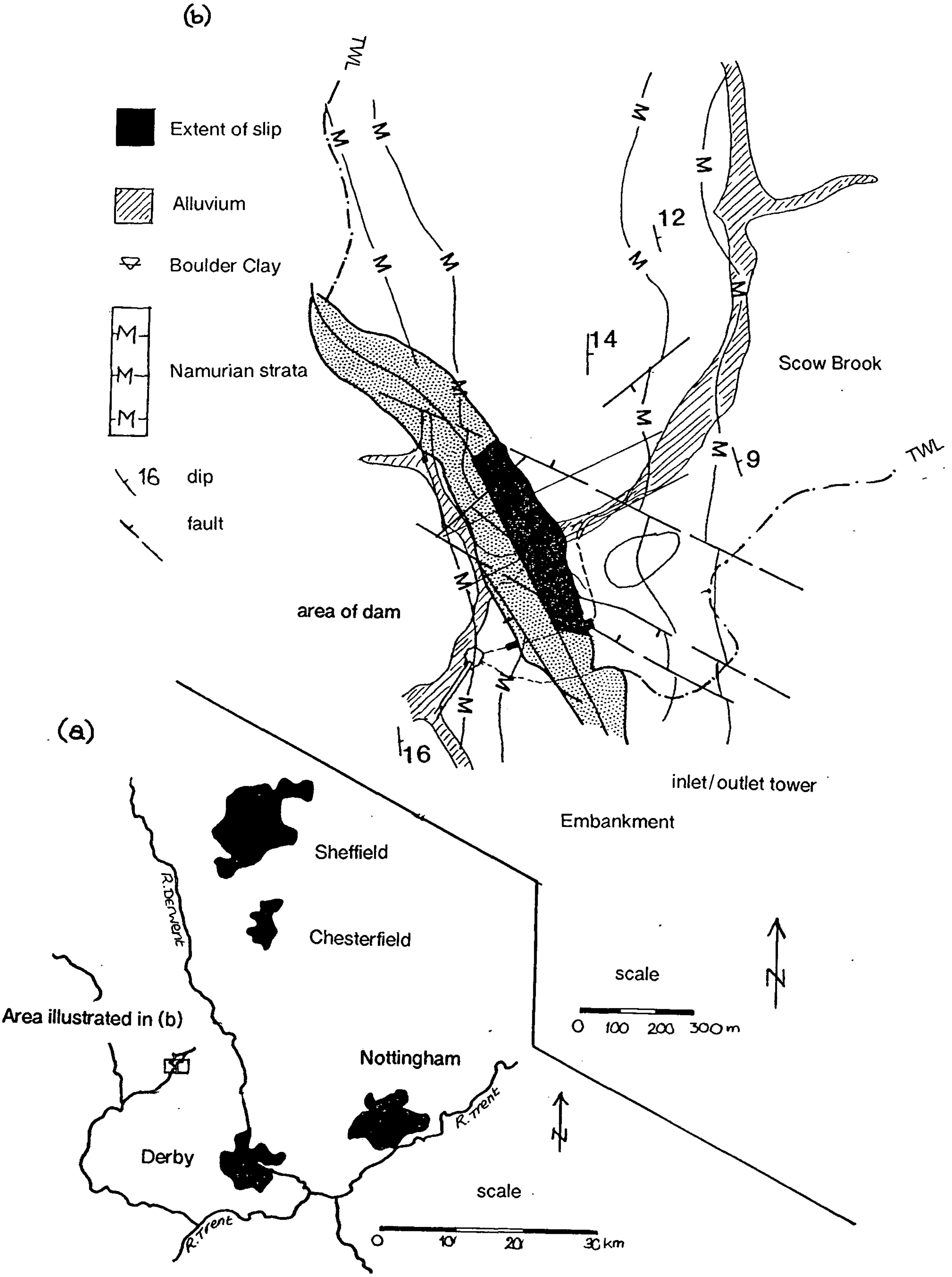
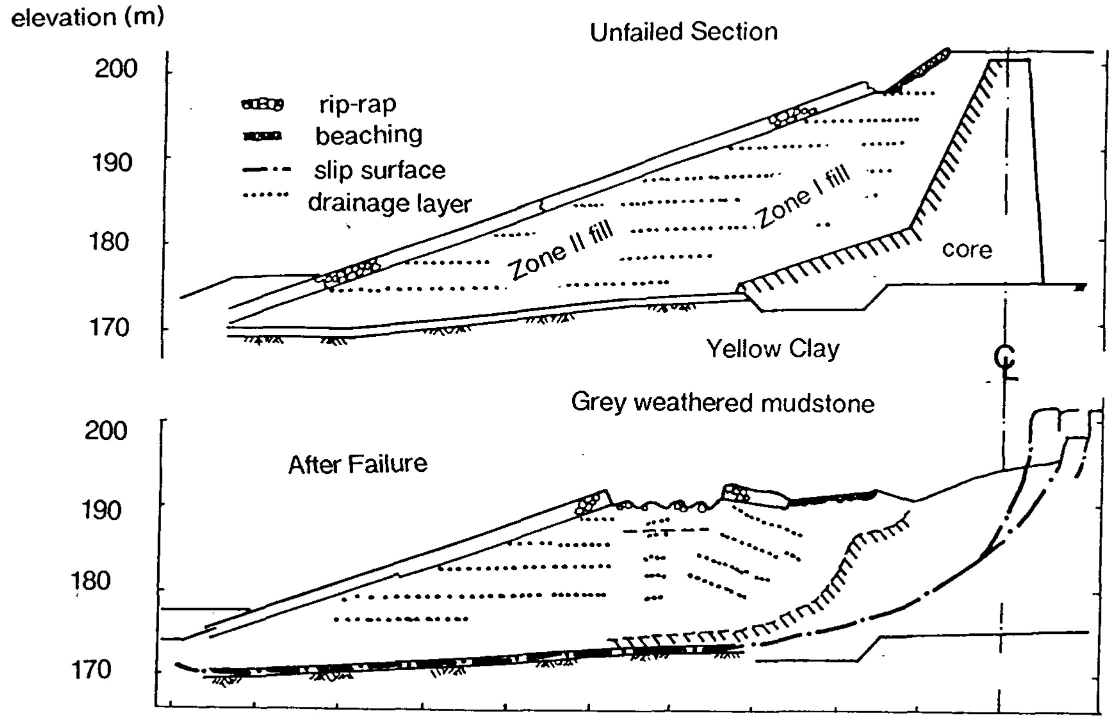
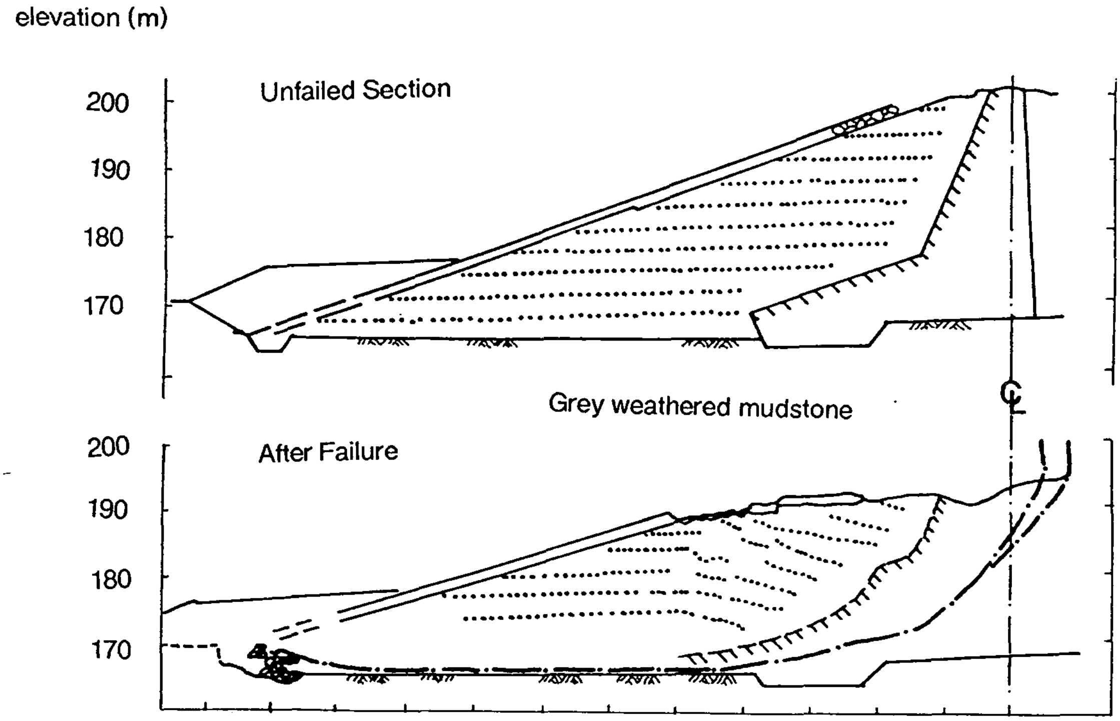


Figure 1.2 Carsington Dam: (a) Location plan, and, (b) plan of reservoir and dam area showing local geology and extent of slip.



SECTION AT CHAINAGE 725m



SECTION AT CHAINAGE 850m

Figure 1.3 Cross sections of Carsington Dam at chainage 725m and 850m before and after failure (based on Coxon, 1986).

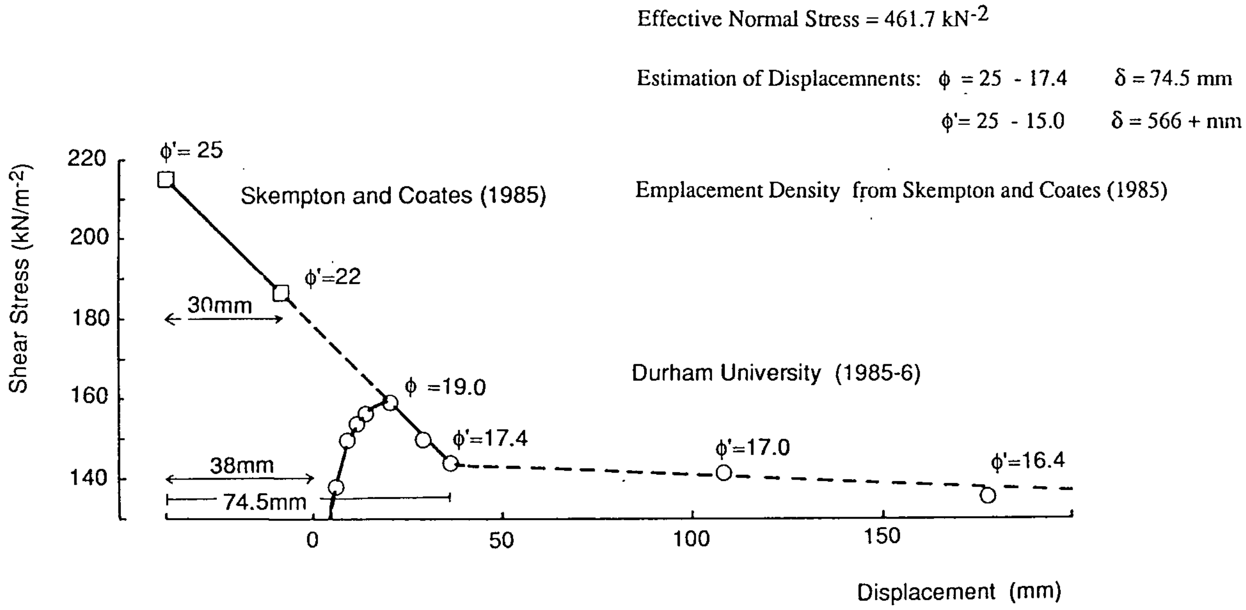


Figure 1.4 Shear stress - displacement curve for post-failure shear box tests on material from Carsington Dam, using data from Skempton and Coates (1985) and University of Durham (1985 and 1986, unpublished). For explanation see Section 1.3.1.c.

Chapter 2. MAN MADE EARTH AND ROCK FILL STRUCTURES

2.1 General

Man made fill structures are widespread and can be divided into two categories – those which function purely as storage tips for waste materials, and those which are designed as engineering structures to fulfil a specific purpose. The latter may or may not be water retaining in character. The two are briefly described below.

2.1.1 Spoil Heaps, Waste Tips and Mine Tailings

Bishop (1973) gives figures for the waste produced per annum in the U.K. These are shown in Table 2.1 below.

Source	Amount (million tonnes)
Coal Mining Waste (10% is tailings)	60
China Clay	21.5
Slate Quarrying	1.4
P.F.A. (60% is utilised)	10
Industrial Waste (eg blast furnace slag)	11
Domestic Waste	14
	120

Table 2.1 U.K. Waste production per annum

The design of tips has been radically altered following the Aberfan disaster. Legislation was introduced in the Mines and Quarries (Tips) Act 1969 which required adequate design and inspection of such structures. British Coal, for example, has abandoned its custom of loose tipping and the current practice is to compact coarse discard into stable horizontal layers by the use of heavy earth moving equipment (N.C.B. Technical Handbook, 1970). This has the added advantage of reducing the risk of spontaneous combustion which can be a problem in loose spoil with a high air voids ratio.

At the time of writing, however, reports in the national press described several tips which had recently moved or were potentially unstable. Wheal Remfrey near St.Dennis (Sunday Times, 25 February 1990), Penrose near St.Austell and Fraddon Down (New Civil Engineer, 1990), all belonging to English China Clay in Cornwall slipped following periods of intense storms, while Cilfynydd in South Wales (The Times 21 February 1990 and the Sunday Times 25 February 1990) was being pumped in an attempt to relieve high porewater pressures. Despite this work, local residents were worried that there could be a repetition of the Aberfan disaster.

Bishop (1973) summarized several cases of tip instability and concluded that the modes of failure were determined by the behaviour of *in situ* foundation strata and the properties of the tipped material. In many cases small rotational slides were responsible for triggering disasterous flowslides, the common factor in each case being the loosely tipped material.

2.1.2 Embankments and Earth fill dams

Engineered earth fill structures perform a variety of functions, for example, road and railway embankments, tailings dams and water- retaining dams. Other structures include dykes and levées. Construction material may derive from a variety of sources although these are usually local and the digging of borrow pits in an area of excavation so that cut-and-fill methods are used is common. Waste material such as minestone or slag may be brought in where excess fill is required, but transport costs generally preclude derivation from distant sources.

Earth and Rock fill Dams

The earliest dams built by man were from earth or rock, and many of those still standing are centuries old. Sowers and Sally (1962) describe examples from the Middle East, India and Sri Lanka which are over two thousand years old. Natural deposits can also retain water – both landslides and glacial morraines are known to form natural dams. These may not always be stable, as illustrated in the following example given by Sowers and Sally (*op cit*). A rockfall south of Nanga Parbat temporarily dammed the River Indus forming a 275m deep lake in 1840. This burst only 6 months later causing widespread death and destruction in the valley below.

The advantages and disadvantages of building earth fill dams are straightforward. The raw materials can be found locally and are easily handled by both ancient and modern construction methods. The resulting structures are suitable for weaker foundations, ideal for low flat valleys and the costs are generally lower than for corresponding concrete dams. At Winscar in South Yorkshire (shown on Figure 1.1) for example, the cost of building a rock-fill dam was found to be 70% of the cost of a gravity dam and 88% of the cost of a buttress dam to perform the same function (Collins and Humphries, 1974). Factors which are disadvantageous to this method of constuction may also be sources of potential danger. Local material is not always suitable as a fill and quality control during construction is more difficult than with a man made material. The spillway cannot be constructed from fill and therefore a separate concrete or masonry structure must be built. Once completed, earth fill dams tend to require greater maintenance than do concrete types.

Fill may be placed either hydraulically, where earth in suspension is pumped to site and allowed to settle out, or mechanically, in layers, with compaction. The former method, although popular in the early part of this century is not now used following several major failures, notably that at Fort Peck dam in the USA. The placement moisture content which is a critical factor in producing optimum density of a fill cannot be controlled in hydraulic fills. Rock can be blasted *in situ* then placed and compacted as required. In distinguishing rock fills from earth fills, Sowers and Sally (1962) define rock fills as those containing boulders 'larger than a man can lift'.

Tailings dams are generally constructed from earth fill, but unlike other water retaining dams are not included in the World Register of Dams produced by the International Commission on Large Dams (ICOLD). In 1982, ICOLD published a separate Register of Mine and Industrial Tailings Dams but this is by no means complete – the British entry numbers only seven, although British Coal own at least 400 lagoons bounded by shale fill dams in which fine slurry and tailings can settle (Penman, 1985). Other owners include English China Clay who build sand and granite fill dams to allow mica residue tailings to settle. The Central Electricity Generating Board (CEGB) owns several pulverised fly ash (pfa) lagoons which are designed and built according to the 1930 Reservoirs Act (see Section 2.2) so are included in the ICOLD Register of Dams. These are again constructed from local materials often with conditioned pfa and furnace bottom ash incorporated into the fill.

2.2 British Earth Fill Dams

The majority of dams in the British Isles are earth fill and most commonly service the public and industrial water supply. Figure 2.1 shows the distribution of dams, by type, in Britain. Unsurprisingly, the concrete dams tend to be encountered in highland areas where hard basement rock and steeper narrow valleys occur. Despite the greater number of earth fill dams, Walters (1962) estimates 200 – 300 in the Pennines area alone, a higher proportion of Britain's water is contained by concrete dams. The British entry in the World Register lists 404 earth fill and 131 concrete dams built in the period 1797 – 1986, but the total collective capacities of these reservoirs are around 2,000 million and 4,000 million cubic metres, respectively. Average dimensions for the two types are shown in Table 2.2 which highlights some of the typical differences.

Earth fill dams (N=404)			
	x̄	σ_x	Dimensions
Height	26.02	30.68	m
Crest Length	555.59	794.50	m
Volume of Dam	548.51	1246.18	$\times 10^3 m^3$
Reservoir Capacity	5154.11	15143.37	$\times 10^3 m^3$
Reservoir Area	626.47	1787.32	$\times 10^3 m^2$

Concrete dams (N=131)			
	x̄	σ_x	Dimensions
Height	32.61	14.90	m
Crest Length	282.12	215.36	m
Volume of Dam	166.35	501.75	$\times 10^3 m^3$
Reservoir Capacity	32327.23	72249.31	$\times 10^3 m^3$
Reservoir Area	2498.49	4532.66	$\times 10^3 m^2$

Table 2.2: Mean (x̄) and Standard Deviations (σ_x) values of dimensions of British Earth Fill and Concrete Dams. Data from World Register of Dams (ICOLD , 1986).

On the whole, concrete dams tend to be high and have narrow crest lengths due to valley shape. They require smaller volumes of material in construction and impound larger reservoirs in terms of both volume and surface area. It must be emphasized that the deviation on all the mean dimensions is very high, so although the statistics are useful in showing overall trends it is important not to attach too much significance to them. Figure 2.2 demonstrates the change of some of these statistics with time, and also shows the number of dams built in consecutive decades beginning with 1797. The early British dams were earth fill with puddle clay cores and were built to impound supplies for canals and water mills. After the 1830s, when methods of water treatment and distribution began to be developed and improved, more emphasis was placed on public water supply. The increase in numbers in the late nineteenth century reflects the industrialisation of the north of England, particularly Yorkshire and Lancashire. The numbers of concrete and earth dams being constructed dropped during both world wars as would be expected, and show an overall decrease during the last few decades. The construction of hydro-electric schemes in the Scottish Highlands between 1930 and 1960 resulted in increased numbers of concrete dams constructed during that period. The data show an overall increase in reservoir capacity and dam size with time. This is due to a rise in demand and improvement in technology both in terms of dam engineering and water treatment. Of the 67 dams built in the U.K. between 1964 and 1984, 65 are more than $15m$ high and 16 more than $50m$ high although this is still modest by world standards (Griffiths, 1983).

Legislation governing British dams and changes in design practice have been influenced by several failures – some very serious. These are summarized by Kennard (1983) and Charles and Boden (1985). Following the worst British dam failures on record, those at Bilberry in 1852 and Dale Dyke in 1864, which claimed 81 and 238 lives, respectively, a set of Parliamentary recommendations was produced in 1865 which required the inspection both of existing reservoirs and of the plans for new ones. No legal action was taken until 1930 after further failures at Coedty and Eigau in North Wales where 16 people died and at Skelmorlie in Scotland where 3 were killed. The three incidents, which occurred in 1925, caused public outcry and put pressure on the Government to act.

The 1930 Reservoirs (Safety Provisions) Act requires that all reservoirs over 5 million gallons (or $22700m^3$) in capacity are inspected on a regular ten yearly basis by qualified Engineers who are appointed to a Panel. Likewise, design and construction must be under the supervision of a Panel Engineer who, although the dam owner is responsible for maintenance, carries the ultimate responsibility for safety. Before this legislation, owners were able to appoint any engineer they considered to be suitable, and inspections, if undertaken at all, tended to be spasmodic. Although this Act was an improvement, and indeed no lives have been lost in the intervening period, between 1963 and 1966 ICOLD recommended that the Institute of Civil Engineers (ICE) submit new proposals to the British government to modify the 1930 regulations. No immediate action was taken. The Mines and Quarries (Tips) Act of 1969 covered tailings dams *adjacent* to mines and quarries, but not those at distant sites so was not as far reaching as the ICE would have liked. In 1969 however, a potentially serious case of internal erosion was accidentally discovered at Lluest Wen Dam in South Wales, when a horseman riding over the crest fell into a swallow hole caused by internal erosion. Six years later the 1975 Reservoirs Act was passed. This includes some of the ICE's 1966 recommendations and specifies that, for the first time, county councils become the enforcement authorities. Regulated lakes are also included, and the Act also provides for the abandonment and demolition of old reservoirs.

At present the ICOLD World Register contains around half the estimated 2,000 reservoirs in Britain (Kennard, 1984), the average age of these being around 150 years.

2.3 Earth Fill Dam Failures

The causes of failure in earth fill dams are varied and may occur during constuction or while the dam is in service. In some cases the dam remains usable (serviceability limit state), whereas in others, failure may render it completely inoperable (ultimate limit state). Statistics for these conditions, based on work by Charles and Boden (1985), are summarized in Tables 1.3, 1.4 and 1.5.

Cause of Failure	U.K.		U.S.A.
	No	%	%
External Erosion	17	24	30
Internal Erosion	39	55	38
Shearing	10	14	15
Other	5	7	17
	71	100	100

Table 2.3: Causes of in-service failure in earth fill dams, U.K. and U.S.A. (data for USA from Middlebrooks (1953))

Time of Failure	No	%
Construction	17	19
In-Service	71	81
	88	100

Table 2.4: Timing of failure in U.K. earth fill dams.

Extent of Failure	No	%
Ultimate Limit State	18	25
Serviceability Limit State	53	75
	71	100

Table 2.5: Extent of in-service failure in U.K. earth-fill dams.

Table 2.3 shows the comparitive frequency of shear failure as a cause of in-service failure in earth fill dams. Earth fill dams tend to become more stable with time as consolidation processes (especially in wet puddle clay cores) take effect, so the likelihood of failure with age is substantially reduced (Londe, 1982). Charles and Boden (1985) concluded that the end of construction, or the first rapid filling, is the most critical time for shear instability.

The most frequent cause of failure is internal erosion, that is, seepage or leakage of water through the dam and/or its foundations. This causes finer material to be washed through the structure and leads to settlement of overlying unsupported material. This mechanism was responsible for the Bilberry and Dale Dyke incidents. In the first case the first sign of trouble was the observation of muddy water running through the culvert. This eventually burst and subsequent excessive settlement left the dam with no freeboard, hence it was overtopped during the next high rainfall. A similar process occurred at Dale Dyke where hydraulic fracturing of the core was caused by arching due to differential settlement between the weak puddle clay and relatively incompressible crushed Millstone Grit shoulders. In both dams the highly permeable shoulders were unable to retain the fines being washed through and thereby prevent, or slow, the process.

In some cases overspilling alone, with no previous settlement or other lowering of the crest, may be responsible for the failure. At Coedty Dam, South Wales, a flood caused by the breaching of a smaller dam upstream overtopped the dam. Water flowing over the downstream slope eroded the shoulder material and hence the unsupported central concrete core collapsed. There is a danger of overtopping if floods occur during construction and inadequate provision has been made for diversionary flow away from the impounded area.

Sliding or shear failure may be initiated in either the fill material or foundations and may be influenced by the presence and shape of the core. The relatively rapid construction rate used at Chingford Dam, where modern earth moving equipment was used for the first time, was thought to be responsible for the build up of high porewater pressures in a thin layer of yellow clay in the foundation, which led to its failure in 1937. Traditionally, shear stability has been analysed by limit equilibrium methods but, for reasons explained in later chapters, this analysis does not apply to situations where progressive failure is operating. Finite Element Analysis has been employed by workers at Imperial College, London (Dounias, 1987) and the University of Swansea (*eg:* Naylor in Coxon, 1986) to interpret the failure of Carsington Dam.

There have been relatively few complete failures in the UK in which dams have been completely breached, but serious incidents where emergency drawdown and extensive remedial works are required may be just as costly. Constructional events, even though not threatening reservoir safety or putting the public at risk, lead to high costs and great inconvenience.

2.4 Types of Fill – Marine versus Non-marine

It was noted in Section 2.2.2 that one of the advantages of building earth fill dams is the opportunity to use a cheap and convenient source of material. A wide variety of rock types has been utilized in the past as shown in Table 2.7, which summarizes materials used in shoulders of dams constructed in the period 1964 – 1984.

Material used for shoulders		number of dams	number of incidents
Earth fill	Glacial Till	14	1
	Fluvio-glacial Sands and Gravels	3	
	Tertiary Sediments	3	
	Weald Clay	2	
	Oxford Clay	1	
	Lias Clay	2	2
	Carboniferous Shales and Grits	7	1
	Carboniferous Marine Shales	2	2
Rock fill	Sandstone and Grit	5	1
	L.Palaeozoic Mudrocks	4	
	Limestone	1	
	Unspecified	3	
	Granite and Sand Waste (E.C.C.)	1	
Concrete		19	1
		67	9

Table 2.7: Materials used in shoulders of recently constructed dams. Data from Dams in the U.K. 1964 -1984 (BNCOLD, 1984).

During this period there have been few problems during construction of new dams, the failure at Carsington being the most disastrous. Minor slips at Draycote and its subsidiary dam Toft, near Coventry, occurred in 1967 due to the presence of fissures in the Liassic foundation clays. The design was modified with the addition of berms. Altmore in County Tyrone developed significant leakage on first filling and grouting of the Boulder Clay fill became necessary. Bakethin, in Northumberland, also showed signs of leakage and cavities developed at the junction between the upstream clay blanket and the permeable rockfill shoulder. At Balderhead, south of Teesdale in County Durham, increased seepage and the appearance of swallow holes were noted during the first filling, and grouting was undertaken to fill cavities and prevent further internal erosion. Unforeseen ground conditions have been reported at several dam sites and these have necessitated modifications to the original design. Empingham, in the former county of Rutland, is built in an area where the Liassic stata have suffered considerable cambering and valley bulging. Wet Sleddale, a gravity dam near Penrith, is founded on a complex fault zone, which necessitated further excavation and redesign of the cut-off so substantially increasing the volume of concrete used. Remedial work to stabilize both old coal workings and a landslip was necessary before construction of Grimwith Dam near Harrogate. A shear surface in the foundations at Ardleigh, just north of Eastbourne, resulted in the need for a redesign of the dam with flatter slopes.

A major factor in earth fill dam building is the suitability of the available fill material. Obviously the design of the dam will allow for certain material inadequacies by the use of flatter slopes, addition of berms and so on, but it is not possible to use certain materials. Collins and Humphreys (1974) describe the construction of Winscar dam, near Huddersfield. The local geology comprises Millstone Grit with beds of shale, mudstone and siltstone of varying thickness. The *in situ* undisturbed strength of the shale, although variable, was found to be adequate for supporting the dam, but on exposure to air and water it underwent rapid deterioration. For this reason, Millstone Grit alone, and no shale, was used for the embankment shoulders and during excavation fine granular fill was placed over the shale foundations to prevent further breakdown.

In non-marine mudrocks and shales, physical disintegration is the primary control on breakdown – this indirectly controls the rate of chemical weathering which is dependent on the surface area available. Marine shales, especially those which are brittle and fissile, are more susceptible to chemical weathering because they customarily contain pyrite which readily oxidises. Added to this, the relatively high quantities of calcite will give rise to greater rates of weathering (see Chapter 4). In non-marine Coal Measures shales amounts of weathering were found to be at a low level (Spears *et al* 1970) whereas geologically older marine mudrocks of Cambrian and Ordovician age tend to be more susceptible to chemical breakdown

(Taylor, 1986).

This raises the question of whether it is wise to use marine shales in earth fill structures when their geochemical stability is in doubt. Taylor (1985) suggests that such materials are appropriate for use in construction provided that adequate quality control is employed, and that the mechanical and geochemical behaviour is fully understood.

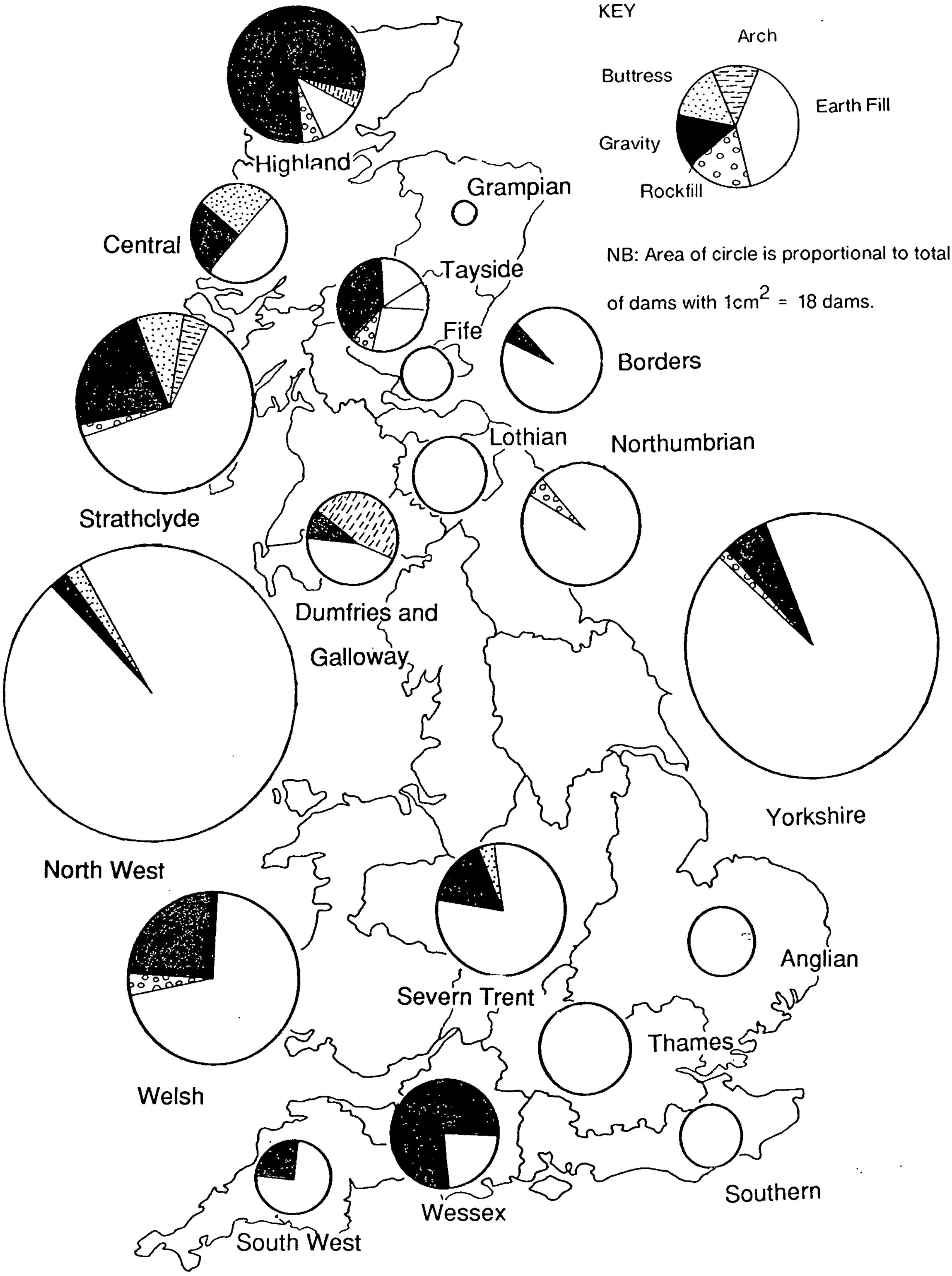


Figure 2.1 Map of great Britain showing distribution of dams, by type, for Water Company (England and Wales) or region (Scotland).

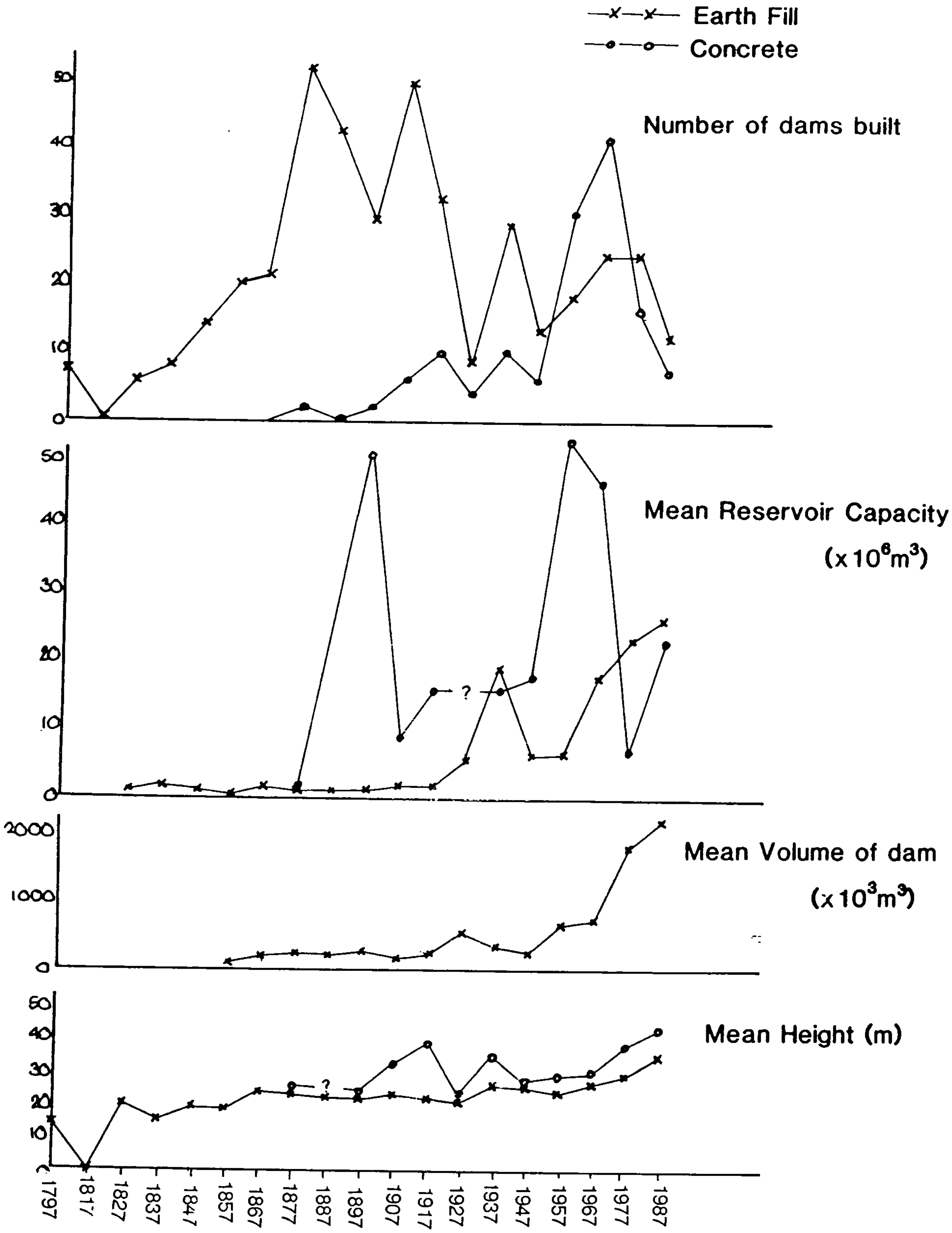


Figure 2.2 Change in dimensions of earth fill and concrete dams with time.

Chapter 3. THE PROPERTIES OF FRESH SHALE

The following chapter reviews the definition of and classification of the term 'shale' which has caused and continues to cause a certain amount of confusion. Typical compositions of British shales are reported, including in particular those aspects which may be problematic. The effects of weathering processes are discussed later in Chapter 4. The history of formation of shales, particularly their sedimentary and diagenetic history, determines their properties. In the case of geotechnical properties, consolidation history is important. Finally, the geology of the shales used in the experimental work discussed in subsequent chapters is examined.

3.1 Classification and Terminology

According to Stow (1981) around 50 to 75% of the geological column comprises fine grained or argillaceous sediments, that is, those with a dominant grain size of less than $63\mu m$. Until the advent of modern analytical techniques such as X-Ray Diffraction (XRD), X-Ray Fluorescence (XRF) and Scanning Electron Microscopy (SEM), little was known about such rocks. Their fine grain size allowed only limited study both in hand specimen and with the optical microscope and their cause was not helped by their usually poor exposure at the surface. The latter is largely a result of their susceptibility to weathering. Limestones and sandstones on the other hand are typically much better exposed and a greater amount of information concerning texture and composition can be observed with the naked eye. Various different criteria – texture, mineralogy, chemistry, colour, degree of metamorphism, and depositional environment have been used to classify sedimentary rocks, including fine grained varieties, at one time or another, and with varying degrees of success.

Pettijohn (1975) quotes an old definition of the term clay: 'a natural plastic earth, composed of hydrous aluminium silicates'. The key factor appears to be 'fine grained', although, as Shaw (1981) and Stow and Piper (1984) point out, this can include substances of different hardness, origin and composition. Problems arise because the term 'clay' is used to refer to both grain size, and to mineralogical composition. To assist clarification, Weaver (1989) suggests using the term 'physil' (an abbreviation of phyllosilicate) for mineralogy, and to keep the term 'clay' solely for grain size. This however would only avoid some of the confusion as there is no agreement between geologists and engineers on the size of clay – petrologists using $4\mu m$ as the upper limit (Wentworth, 1922) and engineers $2\mu m$ (Atterberg, 1905). The latter convention will be adhered to throughout this thesis. It is important that any classification is employable in the field, and as such, most workers employ the grain size definitions shown in Table 3.1, which are based on Stow (1981). A recent paper by Hawkins and Pinches (1992) notes that in many cases rocks are merely described as mudrocks when the terms claystone or siltstone would be better employed. Furthermore, they suggest that claystones should be defined as those 'mudrocks' with more than 40% clay size and siltstones as those with less than 25% clay size as this reflects differences in geological properties and engineering behaviour.

size definition	field criteria	classification
$> \frac{2}{3}$ silt	silt visible with hand lens	silt (*-stone*)
$\frac{1}{3} - \frac{2}{3}$ silt	feels gritty when chewed	mud (*-stone*)
$> \frac{2}{3}$ clay	feels smooth when chewed	clay (*-stone*)

where
clay size $< 2\mu m$
and silt size 2 to $63\mu m$

Table 3.1 Field classification of fine grained sediments and their lithified counterparts (based on Stow (1981)).

Blatt *et al* (1980) quote Picard in saying that modern marine muds comprise on average 15% sand, 45% silt and 40% clay thus placing them firmly in the 'mud' category. There are drawbacks to using such a scheme however. The wisdom of chewing sediments in order to distinguish clay from silt on many engineering sites is dubious to say the least, especially when one considers the range of possible contaminants encountered in industrial areas. A major problem is the efficiency of disaggregation possible, especially of lithified deposits, before particle size analysis can be carried out. It is equally possible to break up original individual grains or to fail to separate aggregates of grains. In many cases the results will reflect this part of the test rather than the true grain size of the sediment.

Spears (1980) considered the problem of what the grain size of a fine grained rock actually means and found that it reflects the mineralogy, as shown in Figure 3.1. As the silt/clay size ratio increases then so does the quartz/clay minerals ratio. As quartz content affects physical and engineering properties, Spears recommended that quartz content, determined directly, be used as a means of classification.

Other problems which have been associated with a grain size based classification are the time consuming nature of the experimental work. The minerals present will have undergone diagenesis which may well have altered the grain size distribution, hence any analysis will not reflect the original distribution and is therefore of limited use in determining the initial environment. For engineering purposes, however, which deal with what exists at present, these analyses are adequate.

Having distinguished between silt, mud and clay in terms of grain size, several problems still remain. As indicated in Table 3.1 the suffix *-stone* may be added to indicate the lithified counterpart, and similarly, many workers will add the suffix *-shale* when the rock is fissile. The term shale is often used rather loosely, in isolation, and to some has become meaningless and as such, many civil engineering consultants refuse to acknowledge its use. Potter *et al* (1980) quote Tourtelot (1960) who defined shale as follows: 'a generally accepted class name for all fine-grained argillaceous sediment, including mud, clay and mudstone'. Apart from its broad basis, this definition has the earlier noted problem of being ambiguous in that chalks and other fine grained carbonates are included with siliciclastics. Stow and Piper (1984) reviewed the use of a two-fold division *ie:* mudrocks which are comprised of detrital siliclastics, and biogenic mudrocks which are composed of biogenic material - either calcareous or siliceous in origin. The word shale usually implies fissility, the nature of which is further discussed in Section 3.1.1. Whatever recommendations are made, the word will still continue to be used in connection with several stratigraphic units, for example the Lias Shales, Wenlock Shales and many others.

At the root of another minor controversy is the use of the term 'mudrock'. Both Grainger (1984) and Weaver (1989) give reviews of this. As Stow (1981*b*) indicates, the words mud and rock describe opposing physical states and hence he claims that the two cannot be used together. However there do not appear to be any other useful descriptors in this context, so the name is frequently used. Tucker (1991), for example, uses the term in a much wider context as a group name for all fine grained clastic sediments. It is therefore necessary to define the point at which a mud becomes a rock. The criteria for distinguishing muds from mudrocks (or shales) are based on geological changes which accompany the rock during diagenesis and are discussed in Section 3.3. The words argillite and slate † are used by geologists to describe shales/mudrocks which have suffered very low or low grade metamorphism and are thus harder than their unaltered counterparts.

To engineers, such features are of little importance unless they have a direct bearing on the strength characteristics of the resulting soil or rock. Geological classifications do not distinguish between over-consolidated clays and indurated mudrocks, yet it is well known that the sensitivity of geotechnical properties is directly related to the degree and nature of induration. As early as 1936, Mead realised the difference between compacted and cemented shales. Morgenstern and Eigenbrod (1974) devised a scheme of classification based on the unconfined compressive strength (UCS) of a soil or rock, and the effect of immersion in water on this property. Basically, they proposed that a soil ('clay') has an initial UCS

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† pelite and lutite have also been used to describe shales in the past but are no longer in vogue; pelite in particular has metamorphic connotations

of less than $3.6MPa$ (c_u of less than 250psi, or $1{,}800kN/m^2$) and that it would loose more than 60% of this strength on soaking. Further subdivisions are based on the time of softening during which more than 50% of the initial UCS is lost. On the other hand, a rock ('mudstone') has an initial UCS greater than $3.6MPa$ and will never loose more than 40% of this strength on soaking. The Engineering Group of the Geological Society, in a Working Party (1977), proposed somewhat different criteria, in which a weak or very soft rock has a UCS of $1.25 - 5.0MPa$, a weak rock or hard soil $0.6 - 1.25MPa$ and a soil less than $0.6MPa$. BS 5930 (1981) employs similar criteria with very weak rocks having UCS of less than $1.25MPa$ and weak rocks between $1.25 - 5.00MPa$. This classification is in many ways more useful in that it is directly applicable, enabling instant identification in the field without the necessity of time consuming tests. The two schemes are compared in Figure 3.2 and the field criteria employed by the latter method are listed. Some formations normally considered to be rocks turn out to be defined as soils when this classification is applied (*see* Taylor and Cripps 1987). The simplest and quickest method of testing rocks in the field or laboratory is the Point Load Test, although it is claimed that it is unsuitable for rocks with strengths less than moderately weak (Norbury, 1986, Hawkins and Pinches, 1992) as part of the measured load reflects the indentation of the plattens into the sample. There is at present, no generally accepted correlation of the Point Load Index with UCS.

3.1.1 The Property of Fissility

If the property of fissility is to be used to distinguish between shales and mudrocks its nature and origin must be considered. Fissility is the tendency of a rock to split along relatively smooth planes which may be parallel to bedding, or to a tectonically imposed fabric such as cleavage. Various classifications have been suggested based on thickness, morphology and ease of breakage although none has found its way into common useage. There are various reasons why a rock will split easily in one direction or plane. These include clay mineral orientation, either natural or by compaction, and the presence of layers of organic matter. It is likely that various causes will apply in different shales. In a study of a Carboniferous shale, Spears (1976) compared samples from a surface exposure with those from a borehole. While the surface samples were fissile, the deep ones were not, although they contained laminations of a comparable thickness to the fissility and were assumed to be a varves. The dark laminations contained a higher proportion of organic material and clay minerals than the lighter layers and although the clay mineralogy was no different, these layers showed a greater swelling potential. In this case the fissility appears to be a surface (weathered) expression of laminations observed in fresh samples. In the same study, exposed mudstones were observed to break irregularly into blocky fragments and did not appear laminated when sampled at depth.

Such features are useful environmental indicators – formation occurs in conditions of alternating moving and slack water and relatively low energy conditions are required for preservation. Mud laminae have reasonably high preservation potential due to their cohesiveness but they will be destroyed by bioturbation. Hence a mudrock deposited with an active infauna is unlikely to become fissile in the manner described.

To summarize then, for the purposes of this work, a shale is a member of the class of mudrocks which are dominantly siliclastic in composition and defined according to the percentage of silt and clay size particles present. This is also in keeping with British Standard 5930 (1981). Shales are those mudrocks which display the property of fissility.

3.2 The Composition of Mudrocks

3.2.1 General

As mentioned previously in Section 3.1, until fairly recently fine grained sediments were studied only in terms of bulk properties. Apart from the small grain size, there was little economic interest in, or knowledge about, prevailing depositional environments and likely palaeocurrents. Although interesting to the geologist, it is bulk properties which most influence the geotechnical properties, hence it is pertinent to discuss bulk chemistry and mineralogy at this point. Each section begins by looking at average compositions of shales collected by various authors. Defining an 'average' mudrock presents many problems, not least of which are the substantial regional and local variations resulting from conditions prevailing during deposition. These are subsequently modified during diagenesis, and later during weathering.

3.2.2 Chemistry

The average chemistry of three sets of shales and mudrocks sampled by three authors is shown in Table 3.2. Despite potential problems in looking at average values, the results are remarkably consistent.

	Weight(%)		
Oxide	A	B	C
SiO_2	58.1	58.5	56.2
Al_2O_3	15.4	17.3	15.1
Fe_2O_3	6.4	7.4	5.7
MgO	2.4	2.6	2.1
CaO	3.1	1.3	4.4
Na_2O	1.3	1.2	1.1
K_2O	3.2	3.7	2.6
TiO_2	0.6	0.8	0.8
P_2O_5	0.2	0.1	0.1
MnO	trace	0.1	0.1
CO_2	2.6	1.2	3.3
S	0.6	0.3	0.2
C	0.8	1.2	0.8
H_2O	5.0	3.9	7.6
$Misc.$	-	1.2	-
Total	99.7	100.8	100.1

Table 3.2 Average chemistry of 3 Sets of mudrocks and shales. (A: Clarke (1924) - 78 shales, B: Pettijohn (1975) - 69 shales, C: Ranov *et al* (1966) - 4030 mudrocks)

It is possible to estimate the mineralogical composition of a rock using normative formulae and known oxide concentrations. Nicholls (1962) describes the procedure adopted for mudrocks, and an alternative method by Taylor is described and utilised by Middleton (1985). It is possible to demonstrate the correlations between the various oxides and the minerals in which they appear within the rock (*see* for example Taylor, 1986). Silica is a major constituent of both clay and silicate minerals and may also be present as biogenically formed free or detrital silica. Alumina occurs in clay and silicate minerals as do the alkali metals, magnesium and iron. The latter is found in various iron oxides, sulphides and so on, while magnesium is a constituent of dolomite. Calcium oxide is present in calcite and dolomite. Carbon and sulphur generally derive from organic matter while carbon dioxide is released from the breakdown of carbonates. The iron and carbon contents are thought to be responsible for the colour of mudrocks (*see* Section 3.2.3a).

3.2.3 Mineralogy

One expects to find substantial amounts of clay minerals in mudrocks even though classification is size rather than composition based. Composition has important commercial implications - for example, potential hydrocarbon sources, pure clay mineral sources (bentonite, china clay). Within engineering studies, knowledge of mineralogy is essential in predicting long term behaviour. Such determinations, however, are not necessarily helpful in determining the origin of the sediments, although some mineral suites are characteristic of given environments.

Mineral	A	B	C	D	E	F	G	mean	st.dev
Clay Minerals	25	59	61	66.9	58	72.8	60.6	57.6	14.2
Quartz and Chert	22.3	20	31	36.8	28	19.2	32.4	27.1	6.3
Feldspar	30	8	4.5	4.5	6	1.2	3.4	8.3	9.1
Carbonates	5.7	7	3.6	3.6	5	4.5	3.2	4.5	1.3
Organic Matter	-	-	1	1.0	-	1.1	0.8		
Iron Oxides	5.6	3	0.5	0.5	2	-	-		
Pyrite	-	-	-	-	-	1.8	0.2		
Miscellaneous	11.4	3	-	2.0	-	-	-		

Table 3.3 Average mineralogy of seven sets of mudrocks (A: Clarke (1924), B: Yaalon (1962), C: Shaw and Weaver (1965), D: Weaver (1967), E: Pettijohn (1975), F: Smith (1978) - UK samples, G: Smith (1978) - USA samples.

As with the chemical compositions shown in Table 3.2, the mineralogies shown in Table 3.3 are surprisingly consistent bearing in mind the variation expected. Variability is more likely to show when clay minerals are analysed separately since these are present in high quantities overall and are susceptible to environment related changes during burial and diagenesis. The greatest deviation from the mean is shown by Group A (Clarke, 1924); this is largely due to inadequate analytical techniques and lack of resolution - it would appear that a proportion of the clay minerals were interpreted as feldspars. A fine grained sediment is unlikely to have so much feldspar due to textural and mineralogical maturity. Shaw and Weaver (1965) noted that the quartz/clay ratio is generally variable in mudrocks but tends to be related to that of associated sandstones. The changes in clay mineralogy of UK sediments with time are discussed in Section 3.4 . The origins of the various minerals present in mudrocks are given below.

3.2.3.a Non Clay Minerals

Quartz: The majority of quartz is detrital but a certain amount of both biogenic and authigenic silica may be present. This is described as chert by Blatt *et al* (1980). During early diagenesis in marine sediments, silica may be released during clay mineral transformations such as the illitisation of smectite-illite mixed layer clays (Millot, 1970).

Feldspar: Similarly, there is likely to be a mixture of detrital and authigenic forms. Plagioclase, being slightly more resistant tends to dominate over potassium-rich feldspars as it does in other sediment types.

Carbonates: Carbonates may be present as discrete clasts or as chemically precipitated cements. Calcite and dolomite are the more common varieties; siderite forms authigenically in brackish, partly reducing environments and diagenetically to form ironstone nodules and layers.

Iron Oxides and Hydroxides: These are usually present as grain coatings and have a strong influence on the overall colour of the lithified sediment. Haematite is the most common form in older sedimentary rocks, although hydrous forms such as goethite may dominate in weathered rocks and younger strata.

Sulphides: Pyrite is the most abundant of the sulphides, and is probably also one of the most troublesome minerals. It forms within the sediment, under reducing conditions, during early burial diagenesis and is commonly finely disseminated throughout the rock. This fineness of grain-size leads to a rapid rate of reaction under later oxidising conditions in which acid solutions and substantial amounts of heat are generated. Less distinctively crystalline varieties such as hydrotroilite occur in modern muds, although these quickly recrystallise to pyrite.

Sulphates: Gypsum and anhydrite are quantitatively important but tend to be restricted stratigraphically, being, for example, related to evaporite sequences. Gypsum, in particular, is commonly found on the bedding planes of weathered fissile shale, the sulphate deriving from pyrite oxidation.

Organic Material: More than 2% – 10% carbonaceous matter in a shale, if of the right type, can make the shale a good source of hydrocarbons; some black shales may contain as much as 20%. Carbon, with iron, is largely responsible for the colour of mudrocks (*see* Figure 3.3). Overall, the colour reflects the redox conditions, the production of organic matter and the relative rates of decomposition and sedimentation since it is these factors which control the concentration of iron and organic matter in the sediment.

3.2.3.b Clay Minerals

The clay minerals are a group of hydrous aluminium silicates, the majority of which have a sheet structure. Some notable exceptions with other morphologies include palygorskite which has a chain structure.

Detrital clays are formed through the progressive leaching of metal cations during the weathering of pre-existing silicates either directly at the source, during transport and deposition, or in a soil profile. The type of clay mineral formed is in part related to the environment, with kaolinites forming under humid, acid, well drained conditions and illites and chlorites under temperate, alkaline conditions. Impeded drainage tends to encourage formation of smectites as shown in the left hand side of Figure 3.4. Present day marine muds reflect the prevailing climate and drainage conditions on adjacent land masses. Several authors have noted lateral variation in clay percentage with increasing smectite and decreasing kaolinite and/or illite away from terrigenous source. This has been explained as the result of differential settling rates resulting from flocculation to form different sized particles (Whitehouse, 1951) or by transformation via halmyrolysis.

It is known that hydrothermal fluids alter silicates and aluminosilicates to produce clay minerals and other secondary phases. Again, if the conditions are moderately acidic then kaolinite is formed while smectites, illites and chlorites are formed in weakly acidic or alkaline environments. Clays can also form authigenically, for example glauconite, while chlorite often forms during diagenesis at the expense of smectites. Submarine alteration of volcanic debris to form tonsteins and bentonites also occurs. In some situations, such as hypersaline lakes, authigenic forms may dominate the assemblage.

Figure 3.4 summarises the changes which accompany clay minerals during burial diagenesis. As temperature increases (this is controlled by burial depth – depth on its own is not an influencing factor) the trend is towards increasing stability and degree of crystallinity. Illites and chlorites become more stable; smectites are converted to illites or chlorites via mixed layer phases. Kaolinite is transformed into other kandites – dickite or nacrite which break down completely at temperatures in excess of $200° - 250°$. The other factor determining diagenetic changes is porewater chemistry. If conditions change from acid to alkaline then kaolinite may transform into illite, smectite or chlorite depending on the ionic species present, or vice versa with leaching under acid conditions.

From these observations, several authors, for example Shaw (1981), have explained the variation in clay mineralogy with age. The data from the literature is inevitably biased by sample numbers and positions, varying methods of analysis and so on, but the trends shown in Figure 3.5 appear well founded. In Tertiary sediments smectite is more abundant than mixed layer clays while the reverse is true for Mesozoic rocks. Smectites are effectively absent in rocks older than Carboniferous, while mixed layer clays do not appear to have been found in rocks older than the Silurian. Illite, kaolinite and chlorite are ubiquitous and clearly are the stable species.

3.3 The Engineering Properties of Fresh Mudrocks and Shales

Investigations into the properties of British over-consolidated clays and mudrocks by Cripps and Taylor, (1981, 1986 and 1987) revealed a wide variation and range in properties. These are furthermore substantially modified by the process of weathering, which will be discussed in the next chapter. Apart from obvious differences in lithology, a number of factors are well known to influence the geotechnical behaviour of mudrocks. These factors include overconsolidation ratio, the extent and degree of fissuring, the variability of suction pressures and anisotropy. Problems associated with testing on small samples also leads to difficulties in applying the measured values to in-situ rocks. Sampling methods may result in stress release and thus misleading results. Geographical variation in composition/lithology and other previously mentioned factors causes wide variation in properties for any one formation.

In general, Mesozoic mudrocks tend to be stronger, less compressible and more durable than their Tertiary counterparts (Cripps and Taylor, 1986). This is due to greater depth and duration of burial which leads to enhanced induration, and the lithological differences described in Section 3.2.3.

3.4 Distribution, Formation and Environment of British Mudrocks and Shales

The majority of the sedimentary column, as already stated, comprises fine-grained rocks, with that of Britain being no exception, as shown in Figure 3.6. The following section outlines the distribution and dominant facies of these strata, throughout time in the UK.

Lower Palaeozoic fine-grained rocks are restricted to Scotland, Wales and the Lake District and are largely metamorphosed although by varying amounts. These rocks contain mostly illite and chlorite although minor mixed layer clays have been found in the Silurian rocks of Wenlock/Ludlow age in Wales (Shaw 1981). Most are either associated with turbidity currents, or are black graptolitic deep-sea shales/slates, grading into siltier muds and greywackes at basin margins.

Several distinct depositional environments existed during the Devonian; sandstones dominate, although significant amounts of fine-grained deposits were laid down. In Devon and Cornwall there are several kilometres thickness of deep marine mud, now metamorphosed to slates and low grade phyllites, with interbedded Old Red Sandstone tongues to the north. These marine muds now contain mostly illite with minor chlorite, kaolinite and expandable mixed layer clays. Lacustrine deposits in the Orcadian basin contain laminated black silty mudstones within cyclothemic units (House, 1981). Micas again predominate amongst the clay minerals present. Shaw (1981), quotes the oldest smectites in the British geological record in North East England. Further south, there are no significant muds in the intermontane deposits.

The Carboniferous depositional pattern was determined by block/basin structures which resulted in a mixture of marine and non-marine environments. The Dinantian was dominated by limestones with interbedded muddy clastics which were illite rich derived from warm, upland regions, or kaolinite rich, originating from tropical lowland swamps. Associated shales are often calcareous and may inherit limestone type properties (Ramsbottom et al, 1981). Cyclothemic sedimentation continued into the Namurian during which the proportion of limestones decreased, with around 40% of the deposits being true mudrocks. During the Westphalian, non-marine, fluviodeltaic conditions ensued, resulting in a predominance (greater than 50%) of mudrocks together with significant fireclays/seatearths and coals. Local kaolinite-rich tonstein deposits resulted from the alteration of pyroclastic debris in acid brackish swamp conditions. Taylor (1986) found a regional variation in the clay mineralogy of Carboniferous mudrocks with kaolinite being more abundant in Scotland and North-East England, and a higher proportion of expandable mixed layer clays occurring in Yorkshire.

Unlike the Palaeozoic, the structure of Britain was relatively simple during Mesozoic and Tertiary times although, despite this, stratigraphical units are locally heterogeneous. Clay assemblages are easier to interpret as diagenetic changes have not been as profound (Sellwood and Sladen, 1981).

The Permo-Trias saw a diversity of depositional environments beginning with a hot dry climate under which high salinity playa basins (inland seas) existed. In these conditions the Mercia Mudstone (formerly known as Keuper Marl) was deposited. These are aeolian silts dominantly illitic (70 – 80%) but with some chlorite, expandable mixed layer clays and authigenic clays such as sepiolite, palygorskite, corrensite and attapulgite. Lagoonal conditions marked the initial stages of a marine incursion and the Rhaetic Group consists of marine and semi-marine shales and limestones.

The Rhaetic Westbury beds contain more mixed layer clay and, later, more vermiculite and smectite reflecting increased weathering at the source area. Kaolinite appears at the expense of chlorite in the Upper Rhaetic, reflecting an overall increase in humidity (Sellwood and Sladen, 1981).

In the Lower Jurassic Liassic formation, illite is again the dominant clay mineral in shallow marine shelf deposits of shales and mudstones. Periods of oxygen depletion resulted in the deposition of bituminous shales, which, with more than 4% carbon form potential petroleum source rocks. The Mid-Jurassic saw a return to humid, sub-tropical weathering shown by higher kaolinite concentrations in clastics derived from the north, while in central and southern England carbonate shelf seas and lagoons persisted. High local concentrations of smectite, with illite-smectite mixed layer phases in adjacent beds, occur in the Fuller's Earth deposits. These are thought to be weathered volcanic debris.

Deep marine conditions were widespread during the Upper Jurassic and resulted in major clay/mudrock deposits: the Oxford Clay, Ampthill Clay and Kimmeridge Clay. Low oxygen concentrations led to reducing conditions in the depositional basins which were responsible for the bituminous nature of the Oxford Clay and locally high organic content of Kimmeridge Clay. The two are separated by the Corallian limestone deposited during a regression in southern England and Yorkshire.

Minor uplift, associated with tectonic movements in Europe at the Jurassic – Cretaceous boundary, saw the return of alluvial conditions to the Wessex – Weald Basin, and shallow marine depositional environments in the Yorkshire – Lincolnshire – Norfolk area. Approximately 80% of Lower Cretaceous deposits comprise fine grained clastics with apparently unaltered quartz–clay mineral assemblages together with early diagenetic minerals such as pyrite, chamosite, glauconite, and siderite. Although the source rocks were illite-bearing Mesozoic and Palaeozoic sediments, warm temperate conditions and acid leaching of soils at the source resulted in kaolinite, mixed-layer clay and vermiculite as the main clay mineral species, with local smectites derived from the volcanic debris of periodic eruptions. During the Cretaceous transgression, the proportion of clastic detritus was reduced to generally less than 5% within the chalk itself. Occasional beds and partings are present in the chalk marl and Jeans (1968) recognised two clastic assemblages in the Lower Chalk. These are a detrital illite, kaolinite, chlorite, vermiculite assemblage and an assemblage comprising mixed layer illite/smectite together with authigenic smectite, illite and mixed layer smectite/illite. The latter assemblage increases in percentage with distance from the shore. Negligible amounts of illite and smectite have been found in the Upper Chalk.

The Tertiary marks the return of shallow marine, brackish and alluvial, conditions the deposits of which are now exposed in the London and Hampshire Basins. Quartz and clay minerals are again thought to have suffered little, if any, alteration since deposition. Associated flora indicate tropical conditions as do lateritic deposits. Kaolinite dominated sediments and clays are found in the west, while to the east, illite and smectite-rich mixed-layer clays predominate. Chlorite is found in minor amounts throughout. Discrete smectite horizons accompanied by zeolites occuring in the London basin have been linked to volcanic activity during the early Tertiary.

The widespread occurrence of mudrocks and shales leads to their inevitable varied use in construction. Detailed knowledge of their behaviour is therefore essential for the establishment of appropriate design criteria. The prediction of long and short term effects including weathering and overstressing are most important.

Overstressing includes disturbance during construction.

3.5 Geology of the shales used in this study

The shales studied in this work were all taken from northern and north-east England; Figure 3.7 shows the locations. As explained in Chapter One, two sets of materials were collected. Firstly two shales to act as 'standards' and secondly shales from real and dateable structures.

3.5.1 Geology of the Westphalian shale at Tanners Hall, County Durham.

Tanners Hall Opencast site (National Grid Reference NE 170 380 on Ordnance Survey 1:50,000 Sheet Number 92), owned and operated by British Coal until its closure in 1989, was situated between the villages of Oakenshaw and Stanley Crook to the west of Durham City. Coal was extracted from the Hutton and Low Main seams. In March 1987 a bulk sample from the intervening shale was collected. Figure 3.8 gives the stratigraphy of the relevant part of the Westphalian of the Durham Coalfield which has been interpreted as a floodplain with deltaic sequences. The majority of the beds comprise grey mudstones and silty mudstone. Immediately after excavation the rock appeared massive and it was possible to transport large equidimensional blocks. However, after only a short period of exposure the blocks developed a marked fissility and rapidly disintegrated into flaky fragments, with many of the flat surfaces revealing plant fragments. The typical mineralogy and physical properties of the material are shown in Table 3.4.

Mineralogy		Physical Properties	
Quartz	7 – 15%	Specific Gravity	2.50
Illite	45 – 65%	Liquid Limit	29%
Kaolinite	13 – 18%	Plastic limit	19%
Chlorite	2 – 7%	Plasticity Index	10%
Montmorillonite	0		
Mixed Layer Clay	14%		
Calcite	3 – 4%		
Pyrite	2 – 3%		
Gypsum	2%		
Other			

Table 3.4 Mineralogy and physical properties of fresh Coal Measures shale from Tanners Hall.

3.5.2 Geology of the Kimmeridge Clay at Foxholes Quarry, North Yorkshire

A bulk sample of Lower Kimmeridge Clay was collected from Foxholes Quarry (National Grid Reference SE 724 831 on Ordnance Survey 1:50,000 Sheet Number 100) which is situated near the village of Marton, south west of Pickering, North Yorkshire. The quarry has been disused for many years and is largely overgrown. New exposures occur following slipping of the side walls, which are in many places sub-vertical. The stratigraphy of the Mid Jurassic in North Yorkshire is shown in Figure 3.9, with a section through the sequence at Foxholes Quarry. The Middle Jurassic consists of fluvio-deltaic facies followed by a marine transgression. Following the transgression thick sequences of marine shales were laid down. These are locally rich in organic matter and may also contain calcareous laminations due to seasonal blooms of coccolithoporoids (Anderton *et al*, 1978). Rhythmic alternations of laminated bituminous shales, bioturbated shales, shales with calcareous laminations and thin laminated limestones have been observed. The section at Foxholes Quarry (Figure 3.10) confirms these small scale variations. Overall the exposure is composed of blue grey clays and shales (the Kimmeridge Clay is often referred to as a 'clay-shale') which are iron stained and frequently fossiliferous. The sequence was also described by Middleton (1985) who noted high concentrations of calcite but generally less than 1% pyrite.

Mineralogy		Physical Properties	
Quartz	15	Specific Gravity	2.51
Illite	20	Liquid Limit	49
Kaolinite	10	Plastic limit	23
Chlorite		Plasticity Index	26
Montmorillonite			
Mixed Layer Clay	20		
Calcite	35		
Pyrite			
Gypsum			
Other			

Table 3.5 Mineralogy and physical properties of Lower Kimmeridge clay at Foxholes Quarry, near Pickering, North Yorkshire.

3.5.3 Geology of the Westphalian Shale at Gale Common PFA lagoon, South Yorkshire

Gale Common (National Grid Reference SE 535 217 on Ordnance Survey 1:50,000 Sheet Number 106) is situated near Knottingley, South Yorkshire. The pfa lagoon acts as a disposal point for the approximately 1 million tonnes of pfa produced each week by coal fired power stations at Eggborough and Ferrybridge, 4.8 and 7.2 km away, respectively. Construction began in 1964, and at the time of writing, work on the third stage of the scheme is under way. At the end of the second stage, the main lagoons covered an area of 21.3 hectares and emergency lagoons 20 hectares. A volume of approximately 16 million cubic metres of shale was used in constucting the embankments. Four sets of samples were collected in May 1987, these being:

1. Fresh shale from a lorry arriving at the site
2. Stockpiled material placed during the previous autumn (1986)
3. Five year old shale (1982) from the 19.5m elevation, on the south side of the main lagoon B approximately 100m east of cross section Q. This shale had been exposed in the side of a roadway excavated one week previously in preparation for the addition of further material. Iron staining was observed at approximately 1m depth, and was unusual according to the site engineer. Such staining is seen in coal tips and attributed to weathering during earlier exposure (Billing 1987).
4. Fifteen year old material (1972) from the embankment between emergency lagoons C and D.

Shale used in the construction of the embankments at Gale Common derives from Kellingley Colliery which generates around 800 000 tonnes per annum. The colliery is owned by British Coal and was mining from the Beeston seam in 1972, and from the Beeston and Silkstone seams in 1982, 86 and 87. Fresh samples of the roof measures from these two seams were obtained from the colliery and analysed, the results are shown in Table 3.6. Ramsbottom *et al* (1978) show the position of the two seams in the Westphalian 'A', Figure 3.8. In South Yorkshire the sequence is about 168m thick, compared with 98m in the Durham Coalfield.

Mineralogy		Physical Properties	
Quartz	23%	Specific Gravity	2.54
Illite	54%	Liquid Limit	34
Kaolinite	19%	Plastic limit	20
Chlorite	0.5%	Plasticity Index	14
Montmorillonite	-		
Mixed Layer Clay	-		
Calcite	trace		
Pyrite	-		
Gypsum	-		
Other	3.5%		

Table 3.6 Mineralogy and physical properties of fresh Coal Measures shale from Kellingley Colliery.

Several points of significance emerge from the XRD traces and these will be expanded in Chapter 4. The kaolinite and illite peaks are sharp and symmetrical indicating crystalline materials. There is little expandable mixed-layer clay and no montmorillonite present. This is shown by glycolated analyses and also by the fact that the sum of the components is approximately 100%. The two samples have closely similar compositions and therefore variations on the embankment are likely to be due to later weathering effects rather than mixing of the two extremes of underground measures. This was demonstrated by McWilliam (1975). No pyrite or gypsum and only a small amount of calcite was recorded.

	Elemental Oxide (wt %)	
Oxide	Silkstone Roof	Beeston Roof
SiO_2	57.62	54.28
Al_2O_3	21.78	22.28
Fe_2O_3	4.65	4.91
MgO	1.73	1.66
CaO	0.29	1.31
Na_2O	0.75	0.64
K_2O	3.64	3.80
TiO_2	1.07	1.19
P_2O_5	0.04	0.02
MnO	0.13	0.13
CO_2		
S		
C	8.29	9.78
H_2O		
$Misc.$		
Total		

Table 3.7 Major Element compositions of Coal Measures Shale from Kellingley Colliery.

Major element data again indicate that there is close similarity between the two shales. It is interesting to note that while the amounts of illite and quartz can be closely estimated by calculation from silica and potassium oxide levels, the kaolinite value obtained by recalculation is significantly smaller than the XRD value. The small amounts of calcium oxide present correlates with the minor amounts of calcite measured.

3.5.4. Sampling of Dams

A series of near surface samples was taken from shale fill dams in the Teesside and Weardale areas of County Durham in order to gain quantitative and qualitative estimates of the condition of the shale fill. The relevant entries for these dams in the ICOLD World Register of Dams are listed in Table 3.8. Data for Carsington Dam and for Gale Common have been added for comparison. Originally it was hoped to sample Cod Beck Dam in North Yorkshire. This dam, described by Cook (1973), has suffered from problems related to foundation leakage and slope instability in the valley downstream during its lifetime and several phases of remedial work have been necessary. A bulge in the downstream slope has existed for some time and exploratory work was undertaken in the Autumn of 1987 to check for water-tightness of the dam itself. This involved continuous coring and open hole rotary drilling, but neither method revealed Liassic shale as expected. The dam is founded partly on Liassic shale and partly on sands and gravels which derive from a fluvio-glacial channel running down the east side of the valley. Enquiries to the consulting engineers confirmed that although shale was used near the core during the early construction, the source had run out by the time the dam reached half height and consequently the most clayey material available from the neighbouring glacial deposit was employed (Bass (of Consulting Engineers Rofe, Kennard and Lapworth), 1987 *personal communication*). Consequently the samples from Cod Beck, being sandy and not shaly as hoped, were not considered appropriate for use in this study.

The three dams to the north of the Weardale Valley, Waskerley, Tunstall and Smiddy Shaw were constructed by the Victorian water engineer Thomas Hawksley whose life and achievements are described by Binnie (1981). Waskerley and Tunstall were both large schemes for the time in which they were built. Problems occurred during the construction of Waskerley dam when movement occurred in the outlet shaft due to differential pressure on the 3 in 1 upstream slope. Cracks and cavities were backfilled and sealed, and impounding proceeded with no further trouble. The shaft, however, remains in its non-vertical position. Tunstall also showed signs of leakage through the puddle clay core when only half full and as a result became one of the first earth fill dams in the world to be grouted. Smiddy Shaw is slightly unusual in being impounded on all four sides by embankments. It is fed by rainwaters and has no natural catchment area of its own. In hard winters it has been known for the surface to completely freeze over and hence pipes have been incorporated into the embankments to allow entry of air during withdrawal thereby preventing a vacuum forming. Samples were also taken from Hisehope dam but these proved to be sandy with sandstone clasts and therefore not appropriate to this study.

To the south of Teesdale, two of the five dams in Lunedale and Baldersdale were examined. The shaly nature of the fill at Grassholme was confirmed in exploratory work by Rofe, Kennard and Lapworth in 1969. This work was required due to observed seepages on the downstream slope manifest as damp patches with soft rushes (sp. *Juncus*). There were also questions relating to the stability of the boulder clay supporting the spillway. A relatively permeable layer was proved at a depth corresponding to the observed wet spots. It is now thought that the seepages are due to percolating rainwater at the permeable layer and not to water leaking through the dam.

Balderhead dam was also large for its time. Due to concern over using shale on a project of such magnitude an experimental programme to outline the material properties was initiated (Kennard et al, 1967). The seepage problems experienced during the first filling have been mentioned in Chapter Two, but the dam has performed satisfactorily since then.

In addition to these dams, two spoil heaps were sampled. These were formed from waste created by the excavation of a tunnel in 1907. Many reservoirs are interconnected and this particular tunnel runs from Grassholme reservoir allowing water to be pumped into Hury reservoir. In a similar way water may be transferred from Selset into Grassholme, or from Balderhead to Blackton to Hury via other tunnels, when the levels in the lower reservoirs are depleted. Selset dam itself comprised boulder clay and therefore was not suitable for sampling. This was unfortunate because at the time of work the level had been drawndown for repair to the riprap. The spoil heaps provided fill material, which was again dateable, but had not suffered compaction or been used to impound water. The geology of the areas surrounding the dams is shown in Figures 3.11 to 3.14, and described in futher detail below.

Name of dam	Year completed	Situation			Height	Length	Volume	Reservoir Capacity	Reservoir Area	Owner
		River	Nearest town	County	(M)	(M)	(10^3m^3)	(10^3m^3)	(10^3m^3)	
Waskerley	1872	Waskerley Beck	Consett	Durham	27	732	685	2045	?	N.W
Smiddy Shaw	1875	-	Consett	Durham	14	960	-	1386	?	N.W
Tunstall	1879	Waskerley Beck	Crook	Durham	25	366	551	2364	?	N.W
Grassholme	1915	Lune	Darlington	Durham	30	274	417	6060	567	N.W
Burnhope	1936	Burnhope	Crook	Durham	40	540	1223	6168	?	N.W
Cod Beck	1953	Cod Beck	Northallerton	N.Yorks	24	155	?	514	?	Y.W
Balderhead	1965	Balder	Darlington	Durham	48	914	2307	19,684	1169	N.W
Gale Common	C	off river	Doncaster	Yorks	51	2170	12,300	11,000	273	C.E.G.B
Carsington	1992	Scow Brook	Derby	Derbys.	35	1250	2231	35M	295ha	S.T.W.A.

Table 3.8 Statistics for dams mentioned in study (taken from ICOLD World Register of Dams, 1986). (C – under construction, N.W – Northumbrian Water, Y.W – Yorkshire Water, S.T.W.A – Severn Trent Water Authority)

3.5.5.a. Geology of the Area surrounding Grassholme and Balderhead Reservoirs.

This area lies in the Stainmore Trough which forms a physiographic and structural depression between the Alston and Askrigg blocks of the North Pennines. The sub area between Lune Forest and Stainmore is known as the Cotherstone Syncline, and is described by Reading (1957). Although covered with thick drift deposits, bedrock is exposed in valley and stream sections, and here ranges from Late Dinatian through Namurian in age. The Middle Limestone Group comprises typical Yoredale facies cyclothems which pass from marine limestones and marine shales through unfossiliferous ferruginous shales, sandy shales and shaley sandstones, sandstones, ganisters and coal going up the sequence. The cycles become less regular in the Upper Limestone Group with irregular marine bands, some have more than one sandstone and in some cases a marine sandstone or shale may replace the limestone. It is important to note the lateral variations in thickness of individual beds. This is demonstrated by Reading (1957) who shows the variation in thickness of the Four Fathom and Iron Post Cyclothems across the area.

Grassholme reservoir is founded on beds from the Middle and Upper Limestone Groups between the Three Yard cyclothem and the Crow Limestone cyclothem. 5 Yard and Scar Limestones were encountered in the trench excavations (Burgess and Holliday, 1972). Mudstones and shales from these cyclothems are dominantly marine although a few are thought to be non-marine. Balderhead reservoir covers an area including a distinct marine band below the Upper Felltop Limestone. Both dams lie on near horizontal strata as shown on the map.

3.5.5 b Geology of the area around Burnhope reservoir.

Burnhope Reservoir lies on rocks of Middle Limestone age, *ie:* below the Great Limestone. Although the strata are again horizontal the area lies to the north of the Lunedale fault system and therefore, being on the Alston block, the sequence is significantly thinner and contains fewer limestones.

3.5.5 c Geology of the area around Waskerley, Smiddy Shaw and Tunstall Reservoirs

These reservoirs are founded on late Namurian strata which, as shown on the map in Figure 3.13, is gently inclined to the east. The rocks are part of the Millstone Grit group and here the cyclothems comprise a marine band, shale, sandstone or grit followed by coal. Limestones are much less frequent or persistent, and where they do occur are much thinner - see for example the Upper Felltop Limestone on Figure 3.13 compared to that in Figure 3.11. Marine bands too are rare, but are of great importance in mapping and correlation. During the Namurian sandstones and/or gritstones became both more frequent and thicker with time. The Namurian/Westphalian boundary is marked by the *Gastrioceras subcrenatum* Marine Band (Ramsbottom *et al* 1978) which was formerly known as the Quarterburn Marine Band. The stratigraphy of the area is described in detail by Dunham (1948). Figure 3.15 shows Waskerley and Tunstall sited on shale between sandstone bodies while Smiddy Shaw is on sandstone just below the *subcrenatum* marine band. In view of this it is quite surprising that the samples contain a fairly high proportion of shale. Hisehope Reservoir is founded on and surrounded by sandstone and as expected, shallow pits on the surface yielded no shale fragments. Typical properties of fresh Namurian shales are summarized in Table 3.10 below.

Mineralogy (%)		Physical Properties	
Quartz	30 – 60	Specific Gravity	2.48
Illite	15 – 40	Liquid Limit	61
Kaolinite	5 – 20	Plastic limit	24
Chlorite	2 – 5	Plasticity Index	37
Montmorillonite			
Mixed Layer Clay	0 – 20		
Calcite			
Pyrite	0 – 5		
Gypsum	0 – 2		
Other			

Table 3.9 Mineralogy and Physical Properties of Fresh Namurian Shale. Data from Fytis (1986) - Carsington Dam, and Middleton (1985) - Plankey Mill (Grid reference 795 622), near River Allen in Allendale. The shale was an unweathered sample from the Upper Limestone Group and due its proximity to a coal seam was assumed to be non-marine.

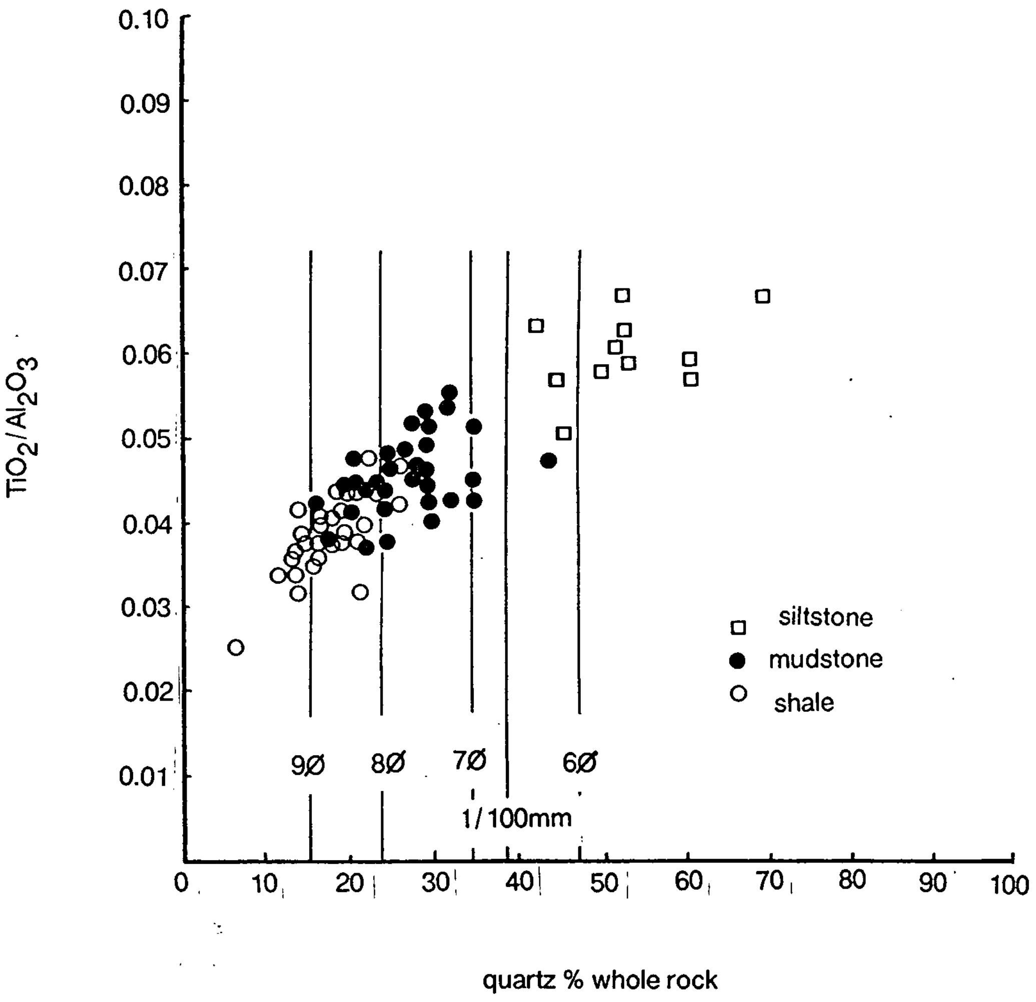
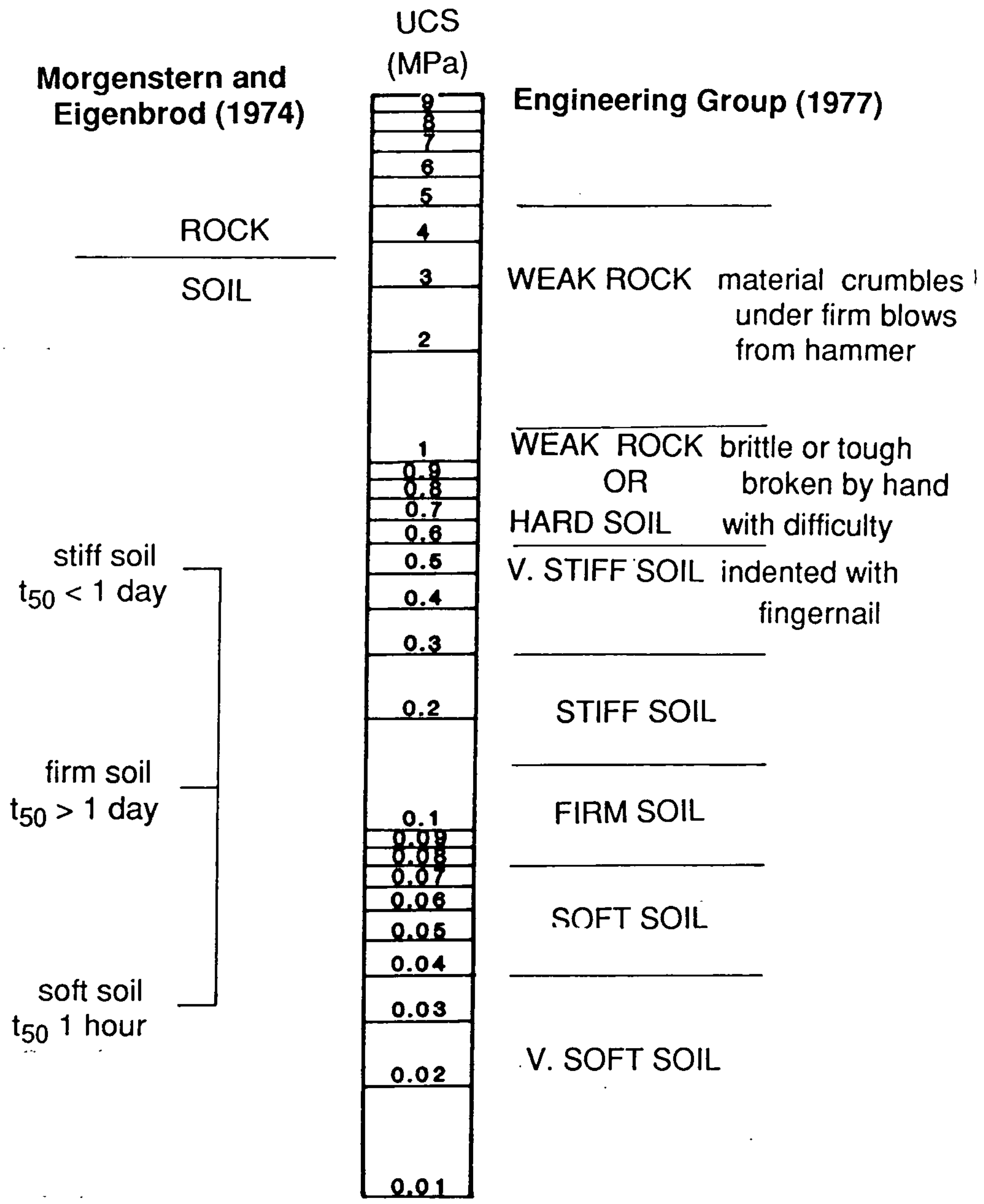


Figure 3.1 Relationship between grain size and clay mineral/quartz content (after Spears, 1980)



t_{50} = soaking time required for reduction of 50% in UCS.

Figure 3.2 Strength (UCS) classification of mudrocks and muds according to Morgenstern and Eigenbrod (1974) and the Engineering Group (1977).

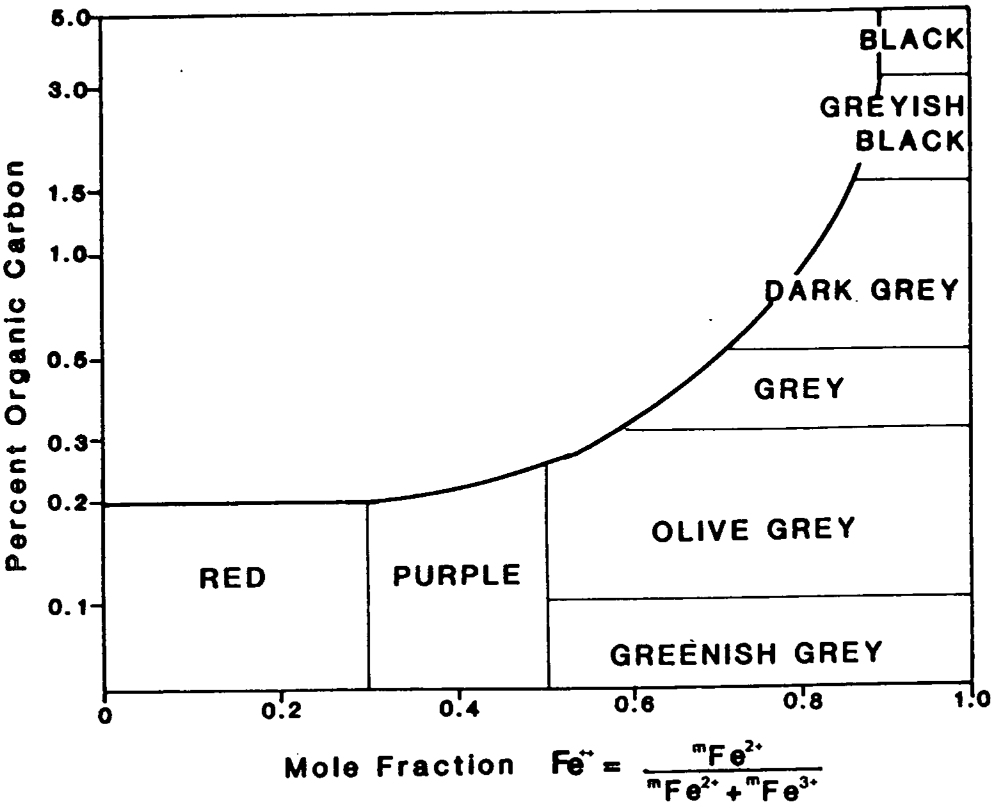
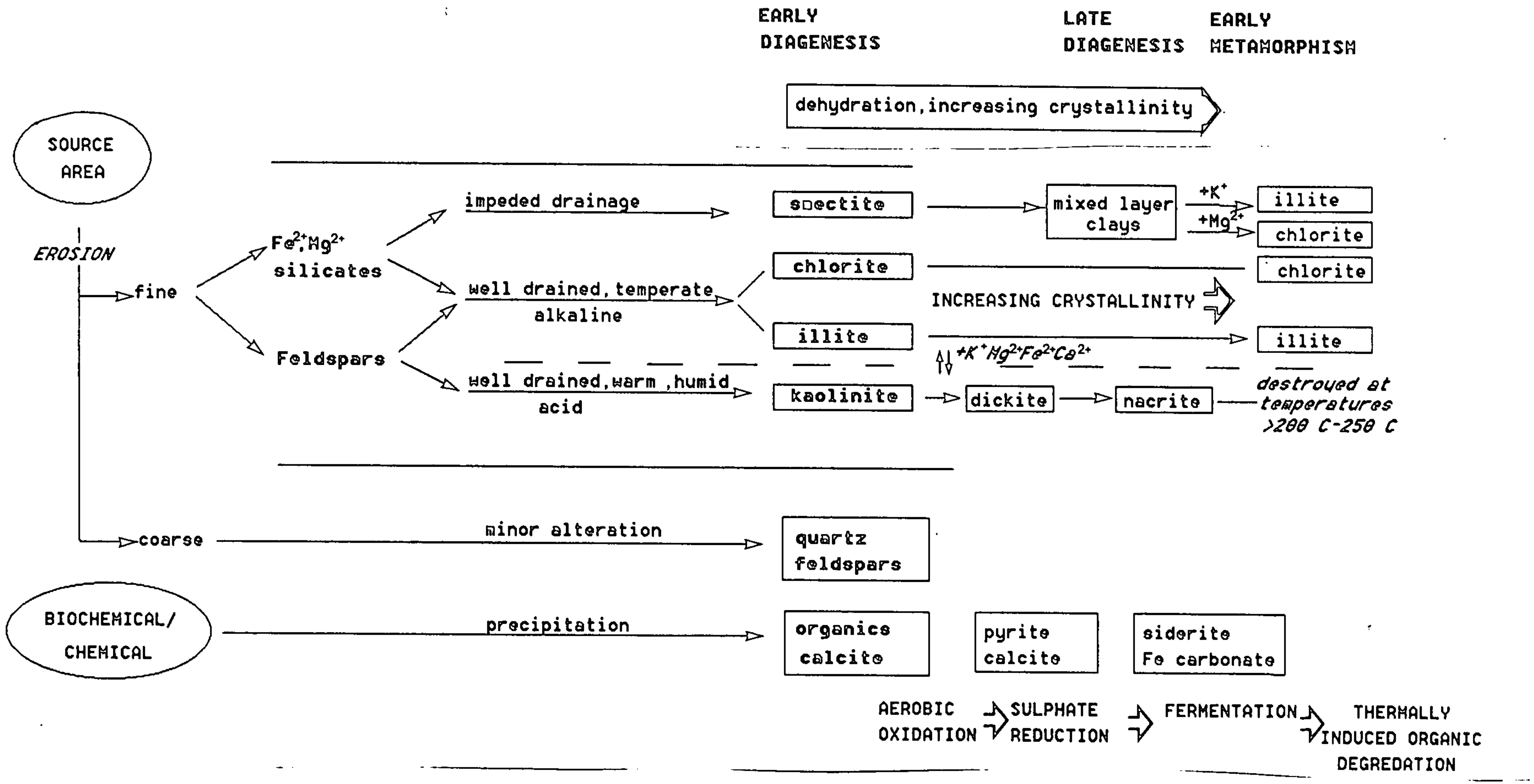


Figure 3.3 Relationship between organic carbon content, iron II/iron III content and the colour of mudrocks (after Potter *et al*, 1980).

Figure 3.4 The formation of clay minerals and their modification during diagenesis.



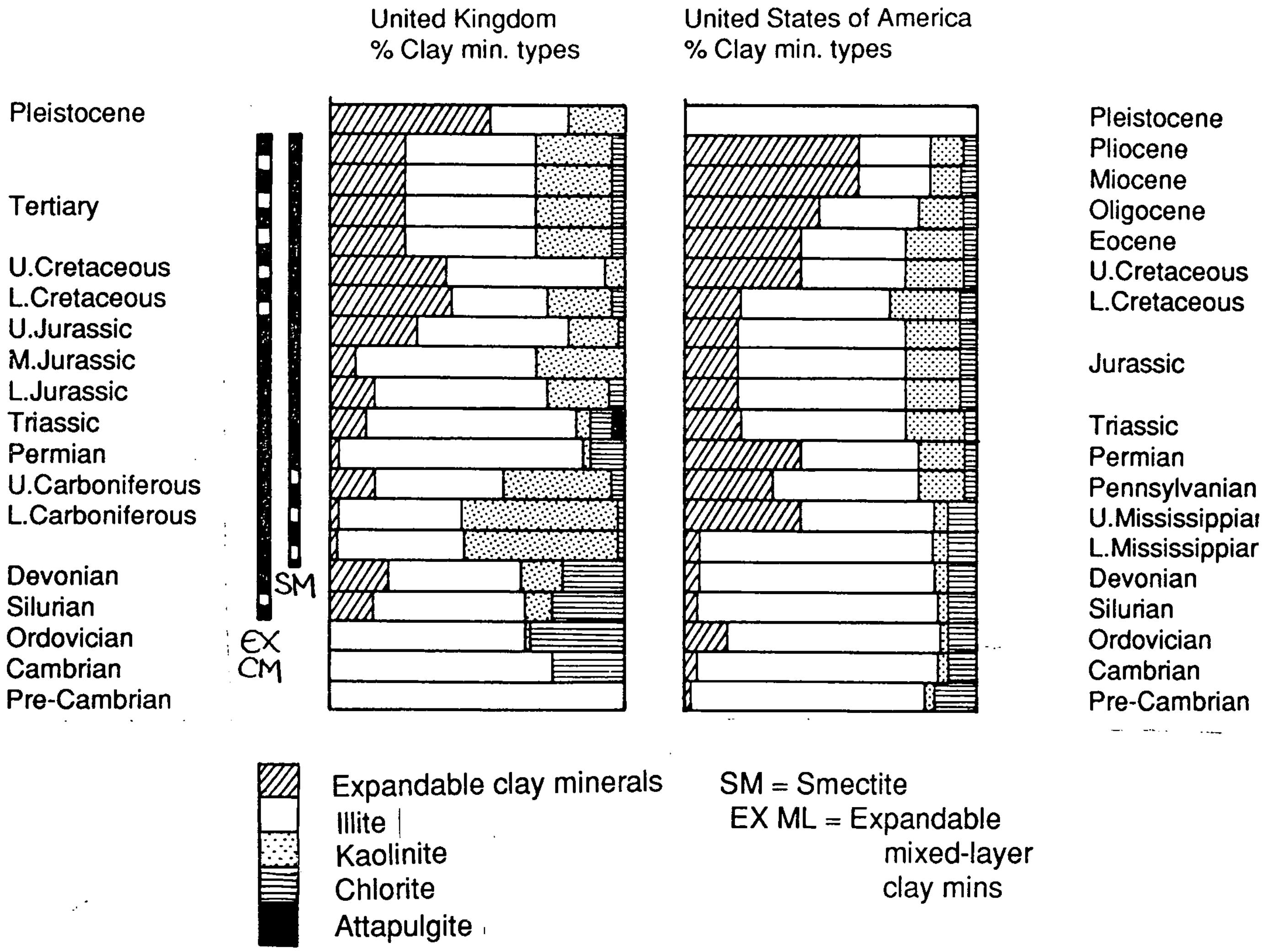


Figure 3.5 Variation in clay mineralogy with geological age (after Shaw, 1981, and Weaver, 1967).

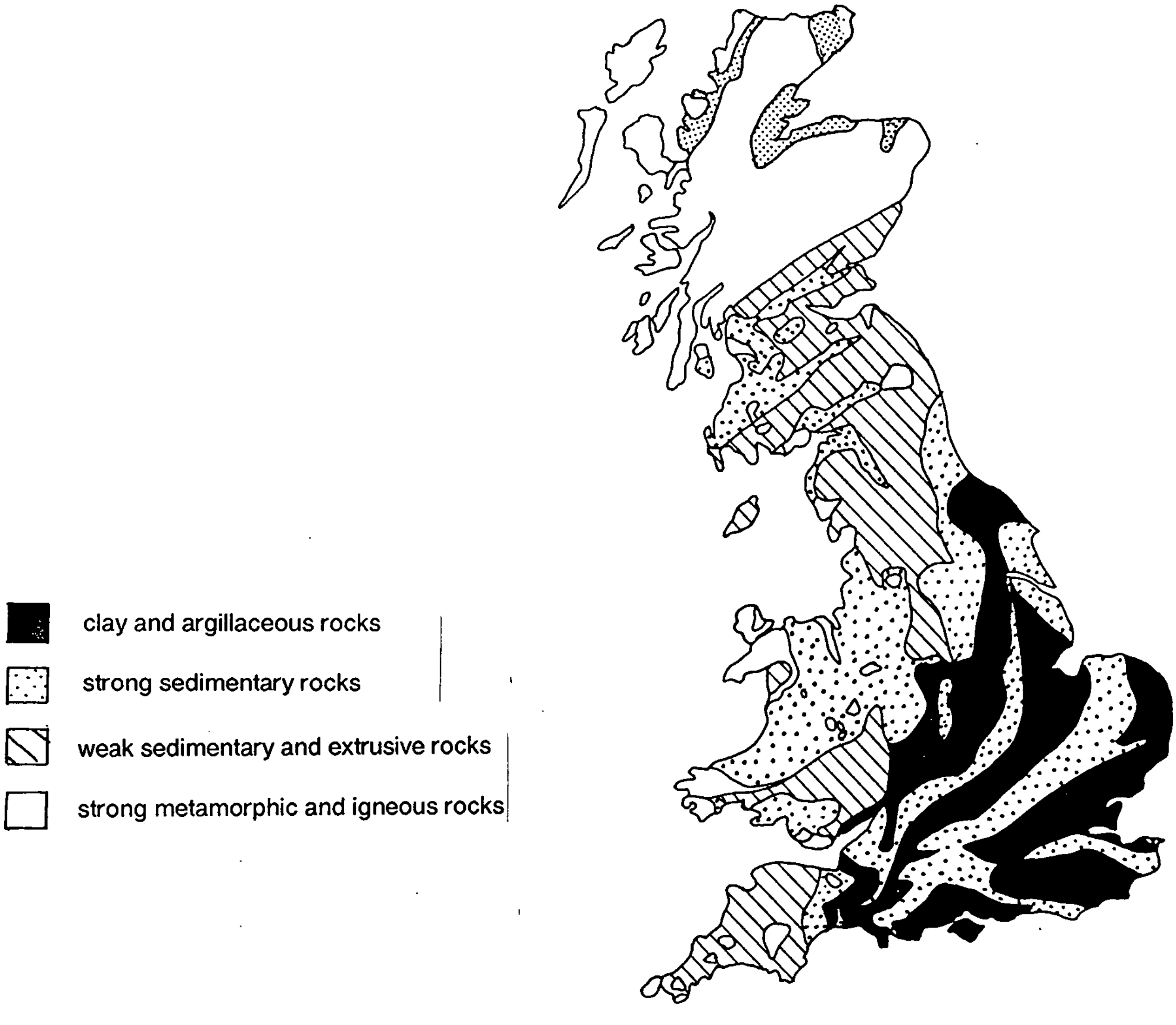


Figure 3.6 Distribution of mudrocks in Great Britain.

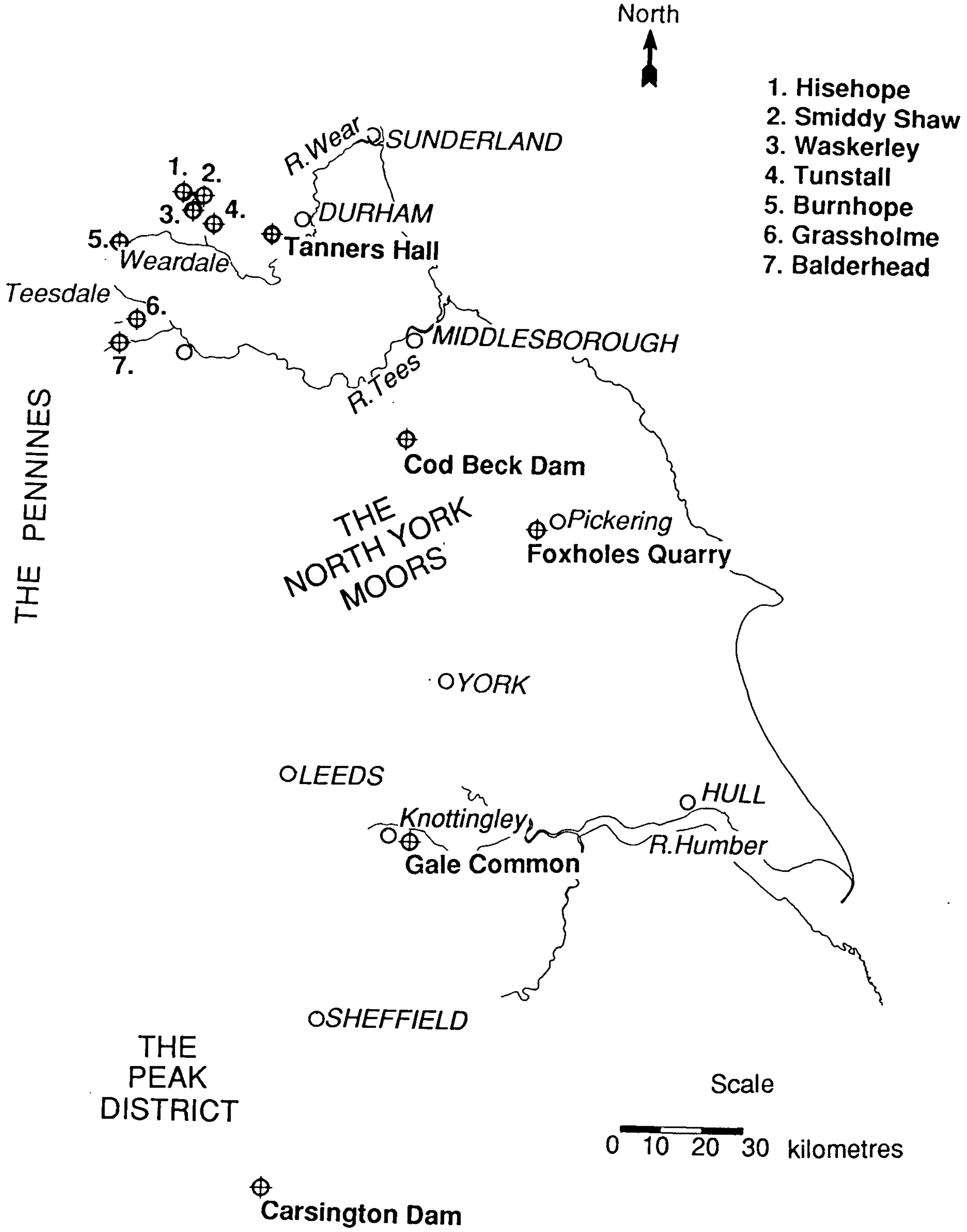


Figure 3.7 Location of sites from which shales collected.

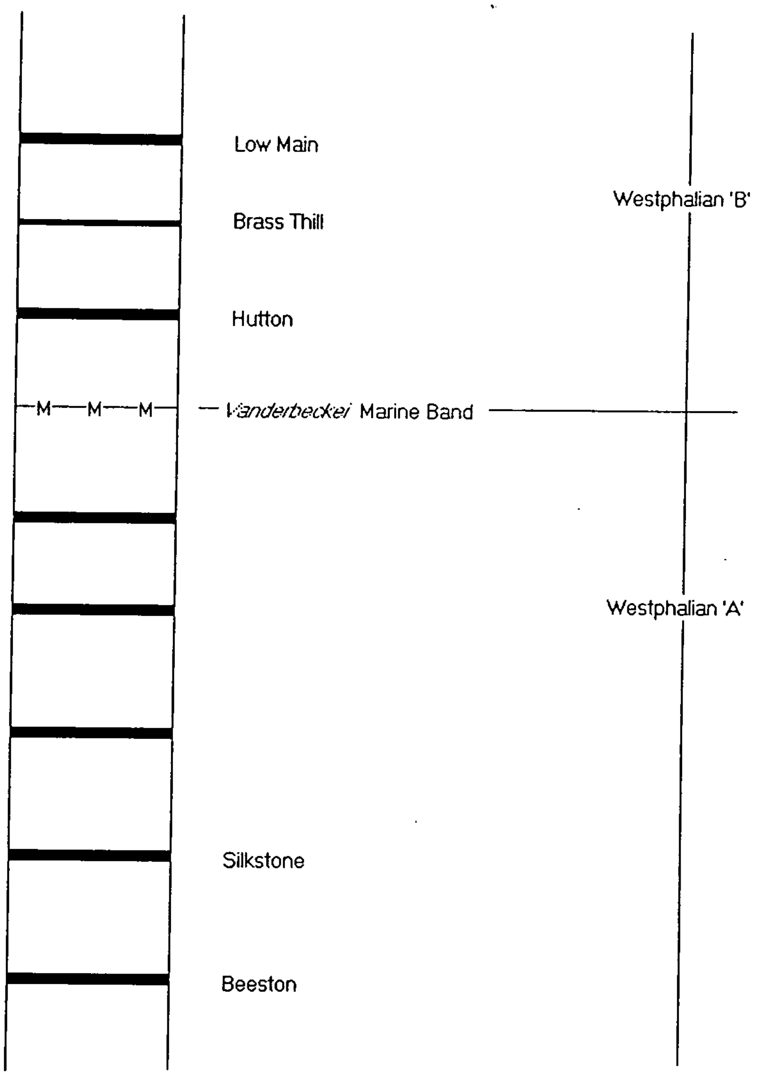


Figure 3.8 Stratigraphy of Westphalian rocks in the Durham coalfield (after Ramsbottom *et al*, 1978).

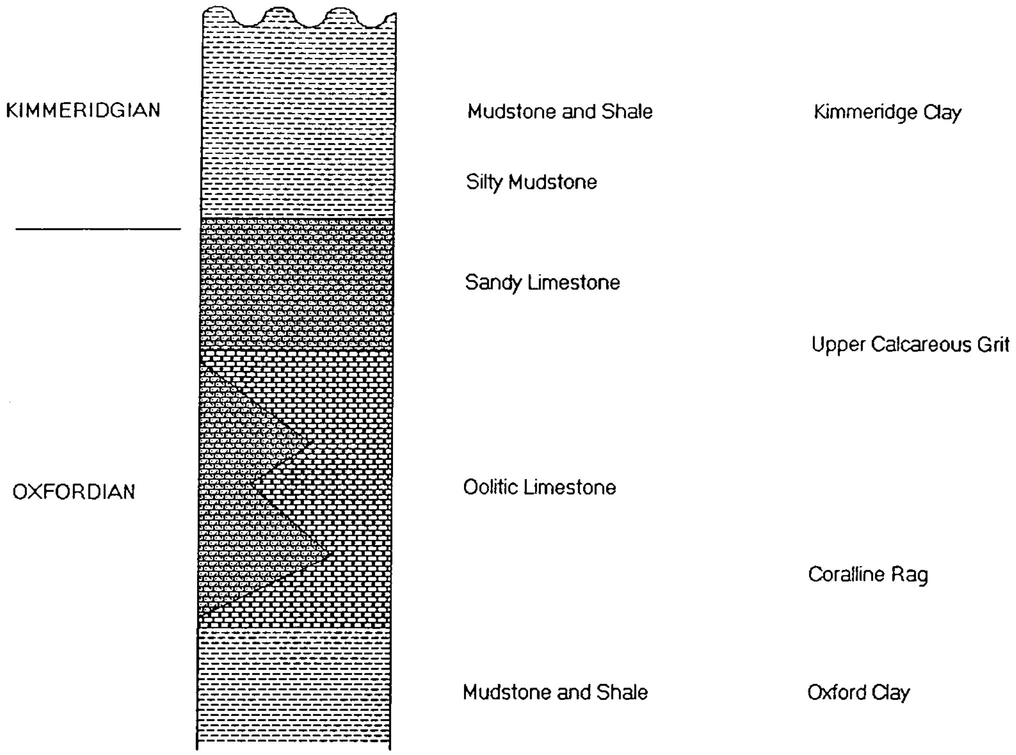


Figure 3.9 Lower Jurassic stratigraphy.

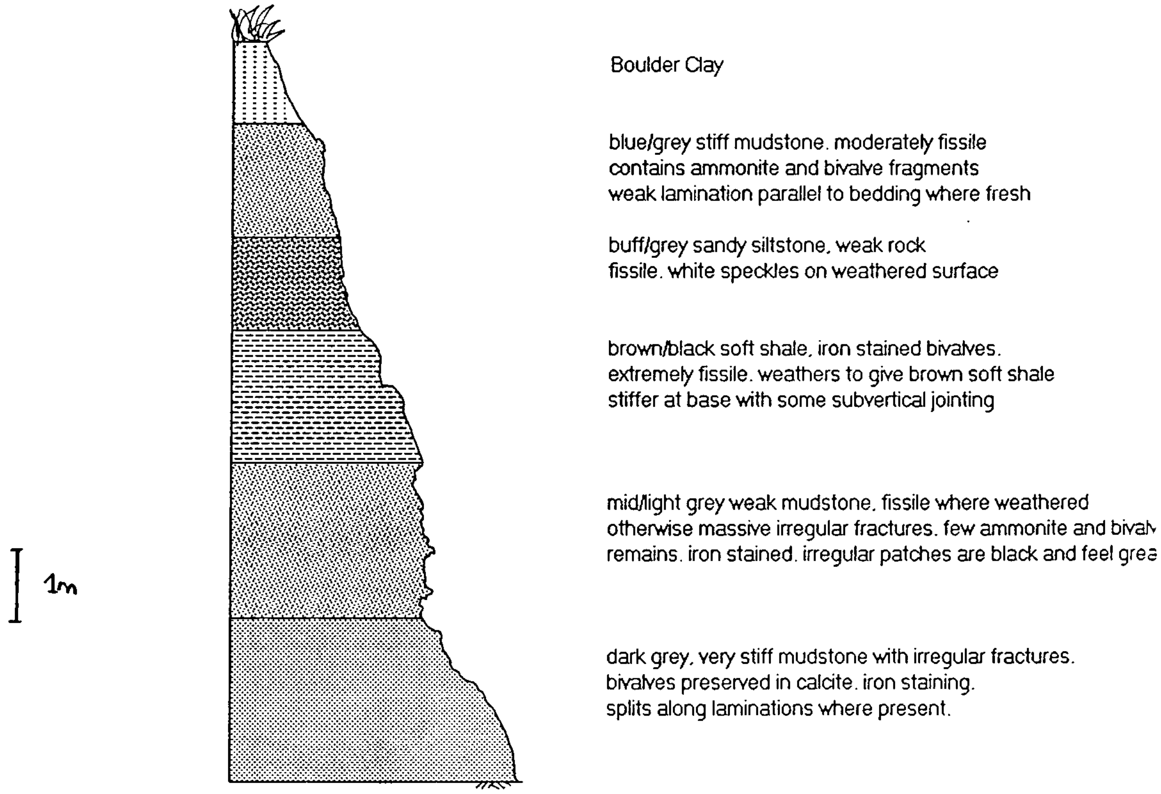


Figure 3.10 Quarry wall section in Kimmeridge Clay at Foxholes Quarry.

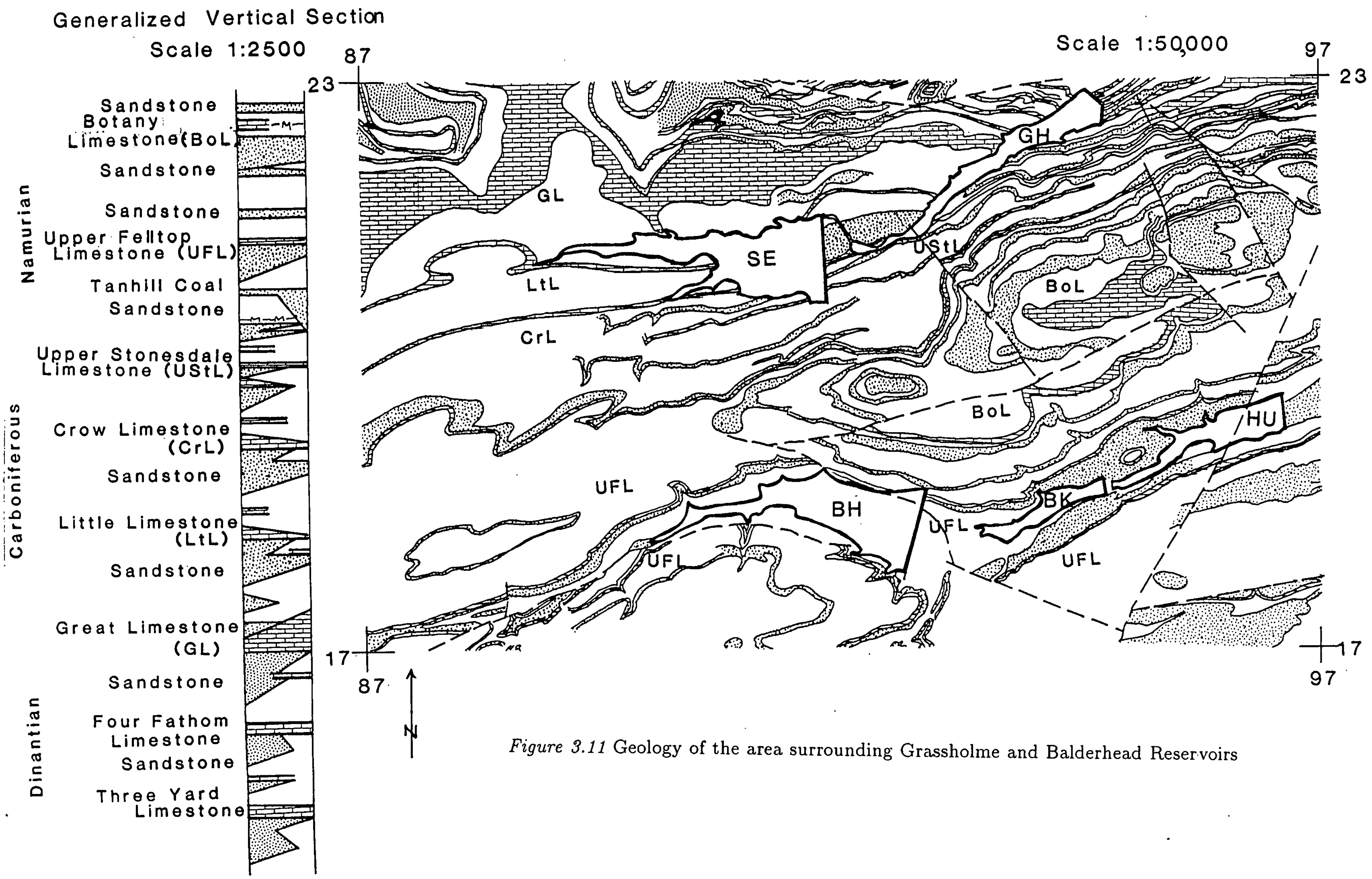


Figure 3.11 Geology of the area surrounding Grassholme and Balderhead Reservoirs

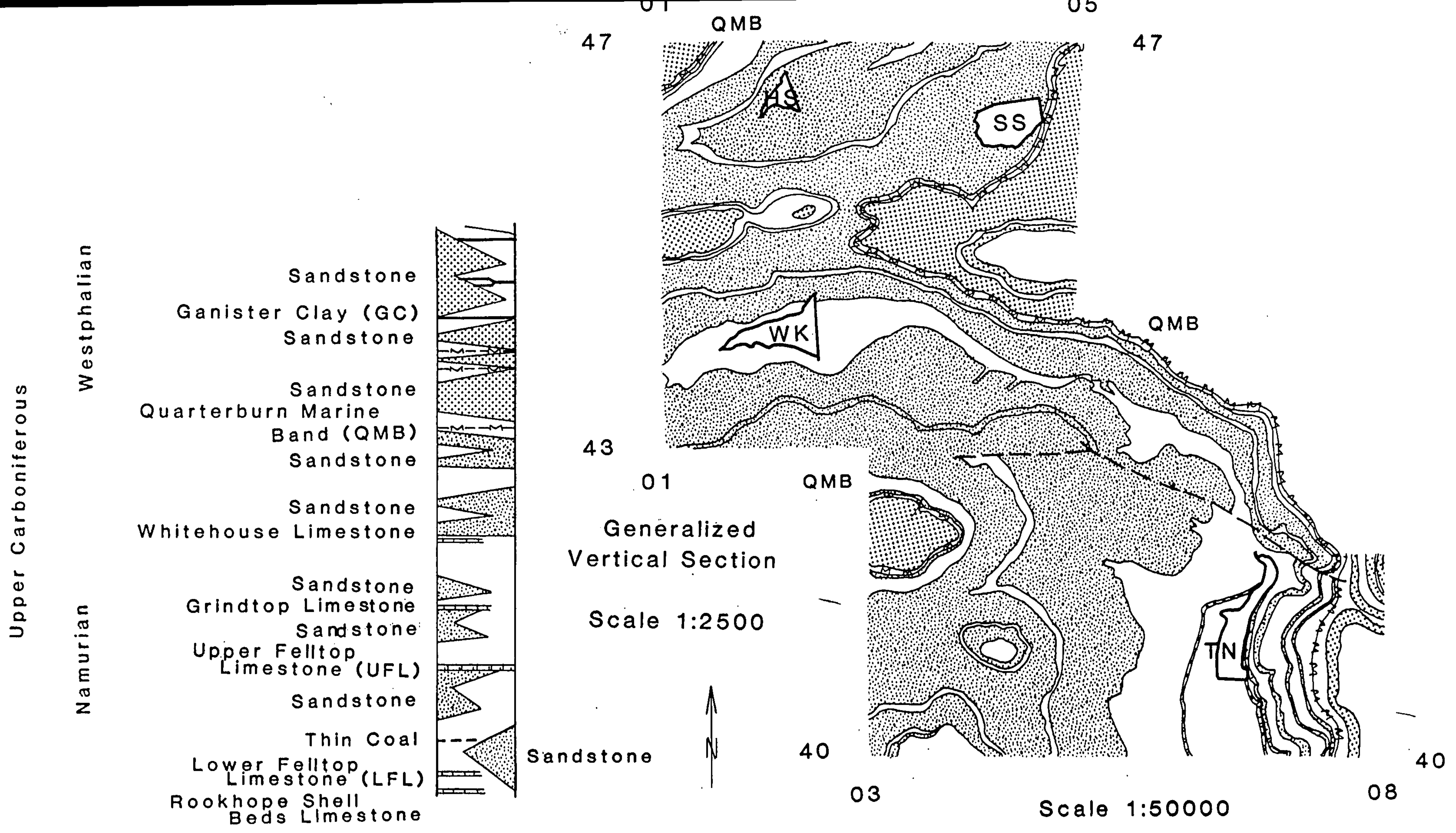


Figure 3.13 Geology of the area surrounding Waskerley, Smiddy Shaw and Tunstall Reservoirs.

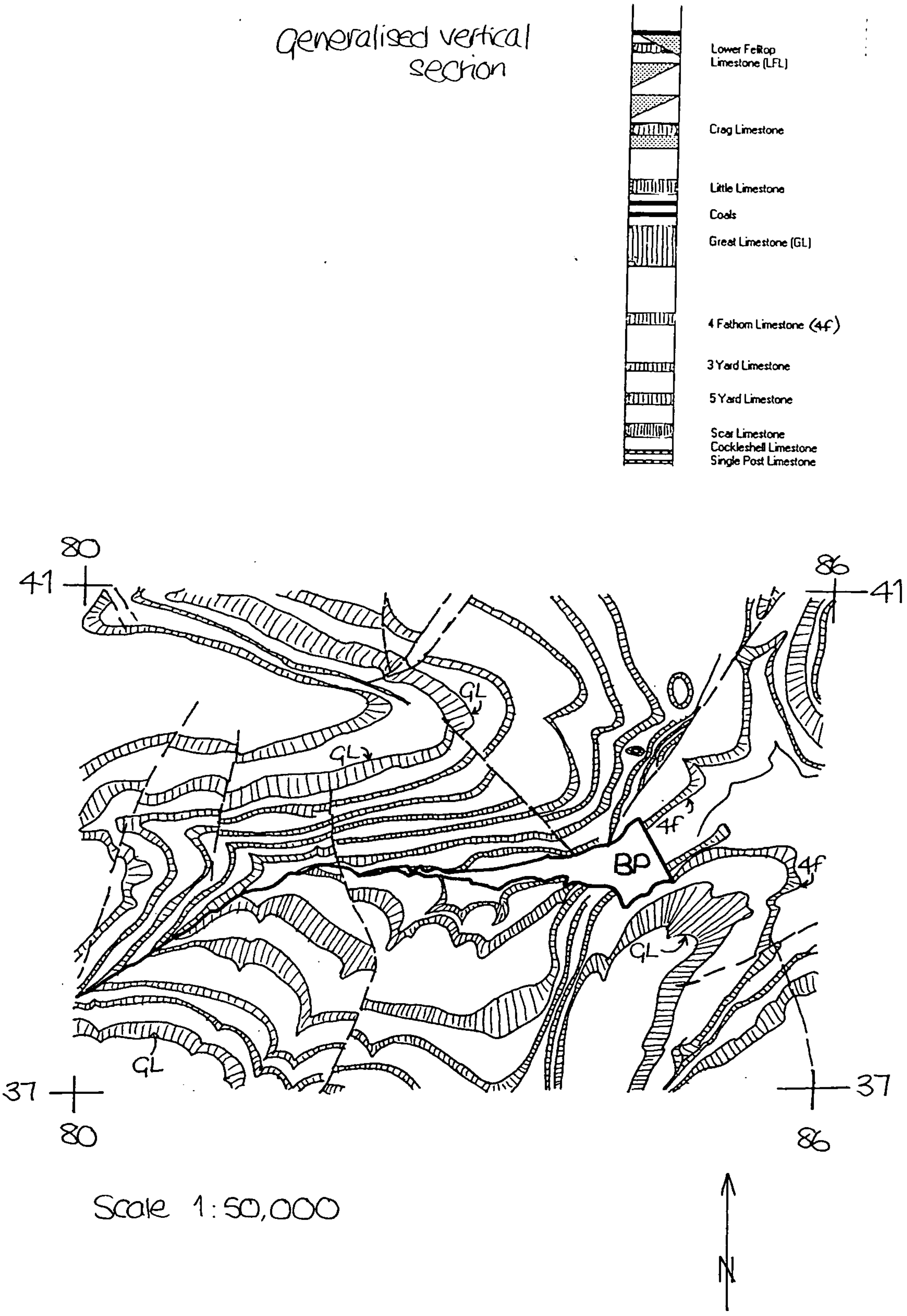


Figure 3.12 Geology of the area surrounding Burnhope reservoir.

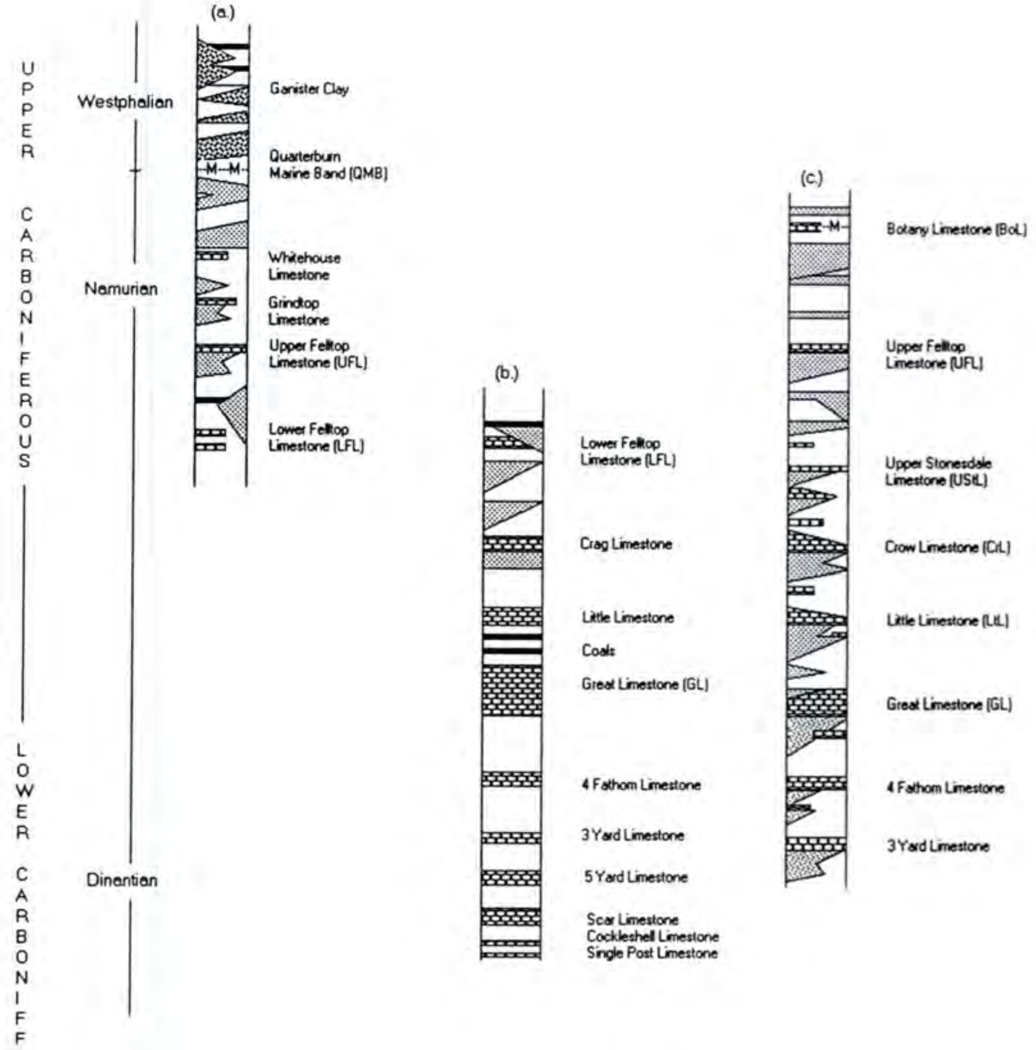


Figure 3.14 Stratigraphy of the rocks in areas shown in Figures 3.11 to 3.13.

4. WEATHERING OF SHALES

4.1 Introduction

Weathering is often regarded as the reverse process of diagenesis and results in an overall deterioration in quality and loss of integrity of a soil or rock. During diagenesis, increase in compaction leads to increase in permeability and porosity. Chandler (1969) notes that one of the most important effects of weathering is the increase in void ratio.

Both diagenesis and weathering result from bringing a material into equilibrium with the prevailing pressure and temperature regime, and the degree of weathering is therefore directly related to the extent by which a material is out of equilibrium with its surroundings. By definition, weathering is caused by the action of atmospheric agents, occuring at or near the surface and involving little or no transport. Most rocks form under pressure – temperature conditions which are very different from those existing at the surface today and hence tend to be more susceptible to weathering than soils which have been formed by prolonged weathering and are therefore generally stable.

Diagenesis is initiated by burial and loading. Weathering on the other hand results from unloading, either by natural (uplift and erosion) or man-made (excavation) processes. Unloading leads to the release of strain energy and thus propagation of high horizontal stresses close to the surface. These are believed to be an important factor in exacerbating weathering (Bjerrum, 1967). K_o values * obtained by both measurement and calculation (Taylor and Cripps, 1987) are highest close to the

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* K_o is the ratio between horizontal and vertical stresses

ground surface. In laboratory experiments, Brooker (1967), demonstrated that stored energy was increased by post geological loading and resulted in increased values of K_o. Such energy is stored by the bonding and deformation of individual particles. Furthermore, diagenetic bonding, that is authigenic cementation, adhesion at particle contacts and grain to grain 'cold welds' (Bjerrum's so-called 'strong diagenetic bonds') contribute to storage of energy in the system. During unloading, the soil expands and there is an increase in the void ratio and water content. Strain energy stored by bonding and compression is recovered relatively quickly but strong diagenetic bonds take longer to break down; thus there is a secondary time dependent effect. Expansion is obviously restricted in the horizontal direction; consequently stresses are set up which may lead to fracturing and breakage subparallel to the ground surface.

4.2 Weathering Processes and factors influencing weathering

Weathering occurs by a combination of two interacting processes - namely physical disintegration and chemical decomposition. Physical disintegration tends to be the more rapid of the two, and is responsible for the overall rate of weathering as it controls the amount of surface area available for chemical reaction, *see* for example Taylor and Spears (1970).

4.2.1 Physical Disintegration

The physical component of weathering involves *in situ* fragmentation of rock or soil by application of cyclical stresses; for example wetting – drying, heating – cooling and freeze – thaw. The effect of application of man-made stresses is discussed later. Stress relief due to unloading has already been mentioned; this leads to swelling and increase in water content. Taylor and Cripps (1987) give a useful summary of swelling processes which include *intra* particle swelling due to rehydration of clay minerals, and *inter* particle or osmotic swelling between clay minerals. The latter arises when separation between clay minerals is greater than that at which attraction by Van der Waals forces occurs (usually about 15Å). A net repulsion between particles causes water to be drawn into the system, which leads to further increase in volume.

The amount of physical disintegration experienced by a mudrock under given conditions is controlled by a number of factors which include:

1. Presence of sedimentary structures and discontinuities.
2. Slaking of the rock - this depends on suction characteristics.
3. Amount of expansion of expandable clay minerals.

4.2.2 Chemical Deterioration

Under any given climatic regime, minerals will tend to dissolve or precipitate in order to achieve equilibrium. During diagenesis mineralogical changes lead to an increase in mechanical stability of clays due to loss of expandable layers (*see* for example Dunoyer de Segonzac, 1970). During weathering, prolonged leaching of clays generates simplified forms or, at the other extreme, certain conditions may lead to the precipitation of minerals.

The processes by which chemical decomposition of rocks and soils take place are summarised in many texts (*see* for example Krauskopf, 1967). These are listed below.

1. Solution and Precipitation

Minerals most likely to be dissolved are halite, gypsum, anhydrite, calcite and dolomite. Of these, calcite is most likely to be a constituent of mudrocks. Several cases where solution of calcite has led to a significant decrease in strength are recorded in the literature, for example the Carboniferous Shales used in the construction of Balderhead Dam (Kennard *et al*, 1967), and Oxford Clay (Russell and Parker, 1979). Precipitation of minerals leads to the formation of duricrusts and duricretes under certain conditions. This may cause problems in man-made structures by precipitation of iron hydroxides ('ochre') in filters and drains.

2. Hydration

Hydration of clay minerals such as montmorillonite may render them unsuitable for use in structures. Uptake of water between lattice layers in the mineral does not lead to any major differences in the compound itself but may cause significant amounts of swelling and hence contribute to further disintegration.

3. Dispersion and Cation Exchange

Minerals prone to dispersion disintegrate on exposure to water. They tend to be rare in the U.K., although the Gault Clay is one notable exception. They can generally be recognised by their high exchangeable sodium percentage (ESP) values. Structures in which such materials are utilised may be vulnerable to internal erosion or piping.

4. Oxidation and Reduction

The oxidation of sulphides such as pyrite, marcasite and pyrrhotite in mudrocks is presently receiving a great deal of attention due to its often dramatic effects on structures and buildings. Penner *et al* (1973) provide the following probable mechanism for the reaction.

$$(I)\ 2FeS_2 + 7O_2 + 2H_2O \rightarrow 2FeSO_4 + 2H_2SO_4$$

$$(II)\ 4FeSO_4 + O_2 + 2H_2SO_4 \rightarrow 2Fe_2(SO_4)_3 + 2H_2O$$

$$(III)\ 7Fe_2(SO_4)_3 + FeS_2 + 8H_2O \rightarrow 15FeSO_4 + 8H_2SO_4$$

(I) ferric oxide is produced during this stage. (II) ferrous oxide is generated , the reaction is catalysed by bacteria.

Sulphuric acid produced in the last part of the reaction is known to react with carbonates and clay minerals present to form secondary minerals such as gypsum (from calcite) and jarrosite (from illite). These reactions generally involve an increase in volume, and a great deal of heat is generated in the initial oxidation reaction. Acid generation may also have a significant effect on geotechnical properties – Steward and Cripps (1983) note that residual strength of Carboniferous

Shales at Mam Tor in Derbyshire reduced due to mineralogical and porewater composition changes resulting from pyrite breakdown.

5. Hydrolysis

Hydrolysis is the process whereby primary rock forming minerals are weathered to secondary clays, hydrous oxides and so on. Although this process is important geologically, it is generally thought to be too long term to influence man-made structures.

The processes most commonly reported to affect mudrocks used in structures in the U.K. are dissolution of calcite and oxidation of pyrite. Mudrocks formed under marine conditions are much more likely to contain these minerals than their non-marine counterparts. This, coupled with the increased incidence of fine laminae built up under slow rates of sedimentation in deep marine environments, causes marine mudrocks to be much more susceptible to both chemical and physical weathering than non-marine mudrocks (Taylor, 1988).

It must be emphasised that the distribution of calcite and pyrite in mudrocks is dependent on depositional and diagenetic factors (Russell and Parker, 1979). Therefore their absence in profiles otherwise containing calcite and pyrite does not necessarily imply weathering away of the mineral. Furthermore, Spears and Taylor (1972) concluded that pyrite content was not a prime cause of rock breakdown, after observations of weathering depths and presence of pyrite in Carboniferous rocks.

4.3 Weathering Classifications

Several visual classifications are available for use in the field as it is thought expedient to describe the condition of weathered rocks in a uniform style for comparitive purposes. General schemes are provided by the Engineering Group of the Geological Society (Anon, 1977) and in BS 5930(1981). Both schemes define the various grades according to the amount of altered material present rather than by the extent of alteration. Variants of the general classifications which refer to individual formations can be found in the literature, for example - Mercia Mudstone (Chandler, 1969), Upper Lias Clay (Chandler, 1972), Chalk (Ward *et al*, 1968).

The distribution of weathered material observed in a rock mass depends on, amongst other things, differences in the original character of the rock, particularly its porosity and the presence of discontinuities. Consequently, the full range of weathering grades may not be seen in any one rock mass. Moreover, mantling effects of drift deposits may preclude development of grades in a mass. Despite this, several authors have attempted to establish depths of *in situ* weathering based on the Zone II/ Zone III (*that is,* slightly weathered/moderately weathered boundary. For example, 5 – 15m in London Clay (Skempton, 1977), 6 – 10m in Oxford Clay (Russell and Parker, 1979), 2.4 – 4.0m in Carboniferous mudstone and 2.0m in Carboniferous shale (Spears and Taylor, 1972). The latter observed weathering down to depths of 20 to 25m where there existed a high frequency of rock mass discontinuities.

It is important to remember that such schemes are intended for *in situ* weathering of a rock mass and therefore cannot be directly applied to earth or rock fills. In such cases man-imposed physical weathering is inflicted on the material as described in Section 4.5, as well as bringing rocks of potentially different weathering states into the same new conditions.

4.4 The effects of weathering on geotechnical properties

The gradational changes in geotechnical properties which accompany weathering are widely known and well documented. The most obvious is, as already described, increase in void ratio due to volume increase and swelling and consequent increase in moisture content. The effect is more pronounced in mudrocks and shales than clays since their initial moisture content is generally lower. This is accompanied by an overall decrease in undrained shear strength which is probably related to increase in amount of fissuring besides the void ratio increase.

Effective shear strength parameters are more difficult to interpret due to the presence of fissures and discontinuities and curvature of the shear strength envelope. The following is based on Taylor and Cripps (1987) and summarised in Figures 4.1 and 4.2. Generally, a large drop in cohesion is observed, with a smaller, but nevertheless significant decrease in the angle of shearing resistance. The drop to a lower bound value of peak effective shearing resistance in the fully weathered state is comparable to the behaviour of a normally consolidated or fully softened (remoulded) clay. Weathering alone cannot produce the ultimate (residual) shear strength which is only attained after large strains.

Relationships between weathering and other variables such as clay mineralogy (and therefore Atterberg Limits) appear less clear cut overall, although correlations have been recorded in individual sites or for specific strata.

4.5 Weathering in earth fill dams

In earth fill dams and similar structures any weathering which takes place may have serious implications in one of two ways. Firstly, weathering may cause a change in material and physical properties - in dam design it is generally assumed that the properties stay constant during the lifetime of the structure (Anon, 1986). Secondly, by-products of chemical reactions may be polluting and hazardous to health and/or the environment. The effects of acid generation at Carsington Dam have been mentioned in Chapter 1.

The extent and rate of weathering on earth or rock fill structures is controlled by material susceptibility, by the degree of leaching taking place within the dam and by the chemistry of water within the fill. In the U.K., waters are not normally aggressive although tailings dams in industrial, urban or mining areas have to be designated to allow for highly acidic and/or chemically contaminated water. Normally, due to the effects of running over hill peat, groundwater in upland areas tends to be slightly acidic whereas that in limestone areas may be highly alkaline and contain high concentrations of calcium. Water in lowland areas is generally neutral or slightly alkaline. Water entering the system as precipitation will have a different chemical composition from that already present. This may be an important factor in influencing chemical and physico-chemical reactions.

Weathering in earth fill dams tends to be restricted to the unsaturated part of the shoulders. The core is, by design, impermeable and effectively chemically inert when fully saturated. A large part of the shoulders, that is, from the water level on the upstream slope to the downstream toe, is also fully saturated but reactions may occur above the ground water level. The scenario may be complicated if cycles of drawdown and refilling occur. During dry summers the presence of a lower than usual ground water table leads to a larger volume unsaturated for weathering.

Leaching through the body of the dam is largely controlled by the permeability of the core, and may be further hindered by the placing of impermeable membranes on the upstream face. The placing of drainage blankets speeds up any pore water pressure dissipation, but may also allow the leaching of chemical by-products away from the reaction site. Furthermore, the presence of drainage blankets may introduce chemically reactive material into the system.

Despite the potential for degradation, a survey conducted by Babtie Shaw and Morton (Anon, 1986) for the Department of the Environment, recorded little evidence of dam failure caused by chemical deterioration of the fill. They did note however that changes to rockfill dams were more rapid and obvious, and that such dams may degrade to earthfill dams with time (see Chapter 2 for definition). It would appear that dams constructed from highly weathered material are more susceptible to chemical decomposition and tend to degrade more rapidly during their lifetime than those built from fresh and slightly weathered rock.

A great deal of work has been carried out on the condition of colliery spoil heaps following implications made in the Aberfan Enquiry that degradation may influence shear strength characteristics of the spoil (Taylor, 1973). Spears *et al* (1970) recorded the rapid breakdown of underground samples on exposure to the atmosphere and concluded that colliery discard would reach its level of degradation fairly quickly. Little weathering was noted below a shallow surface zone less than about one metre thick on the 50 year old Yorkshire Main Tip (Spears and Taylor, 1972) and at the 100 year old Brancepeth Tip in County Durham, little change was noted other than that attributed to combustion in the spoil heap (Taylor, 1973).

4.6 Weathering Studies

Samples taken from the sites described in Chapter 3 were tested as follows in order to view their condition and to determine whether any significant weathering processes appeared to be operating.

4.6.1 Moisture Content

On arrival in the laboratory, samples from weathering profiles were tested for moisture content according to the method outlined in BS 1377:1975 *. Plots of moisture content against depth are presented in Figure 4.3.

The moisture content – depth relationships for the five year and fifteen year weathered profiles at Gale Common are both sinusoidal in shape with the higher (wetter) values occurring between approximately 0.3 and 1.3m below the ground level. Samples from the surface are drier, presumably due to slight desiccation, as are those below 1.3m in the five year wethered profile. In both cases the values range between $10 and 22\%$. The similarity in both shape and numerical range of the two suggests that little change occurs to the *in situ* material during this time range.

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* Work was carried out prior to publication of the 1990 edition

The data from Balderhead Dam and Grassholme Dam show moisture contents to be in the ranges 8 – 12% and 16 – 23% respectively. In both locations slightly higher values are observed at the 'top' of the profiles followed by a decrease in moisture content with depth. Both dams were topsoiled and grassed shortly after construction and this layer, approximately 0.3m in thickness, appears to prevent drying out of the upper surface of the shale fill. On initial excavation, material from Grassholme dam appeared more highly degraded than that from other dam sites. It could be described as brown clayey sand to gravel sized shale fragments, whereas other sites yielded grey sand to cobble sized fragments with occasional boulders. This visual difference is reflected in the higher moisture contents observed.

The two spoil heaps formed during the excavation of the Grassholme Hury tunnel have been partially colonised by grass and low plants. Moisture contents of samples from this area are higher than those from ungrassed parts of the heap with values of 13 – 28% as opposed to 7 – 21%. In neither case is the surface desiccation observed in Gale Common samples apparent, and the shape of the profiles are irregular. One isolated point, denoted 'shale' on the plot in Figure 4.3, is from a sample of arisings at the mouth of the rabbit burrow. Its low value suggest that lower moisture contents exist at depth within the spoil heap although it is obviously not possible to ascertain the original depth of the sample, or the length of time to which it has been exposed to the atmosphere.

4.6.2 Particle Size Analysis

Particle size analysis was carried out on bulk samples from weathering profiles by wet sieve analysis and sedimentation (pipette) analysis according to BS 1377:1975. Care was necessary to overcome the following problem encountered whilst undertaking particle size analysis: Wet sieving is the preferred method in order to separate clay bound clasts, if this is not effected then the distribution curve will overestimate the amount of coarser particles present. However, it is well known that mudrocks and especially shales, tend to slake when wet, and it was found that the addition of water during the sieving process caused clasts to disintegrate. It is important that larger clasts are represented in the distribution curve because they may have an important influence in the shearing process, as shown in Chapter 6. Slake durability tests were carried out on clasts from the four bulk samples taken at Gale Common, according to the method specified by I.S.R.M. (1974). This method is not ideal (Taylor and Spears, 1981), but it provides a useful illustration of the rate and ease of breakdown of these shales. The results are summarised in Table 4.1 below. Further problems were encountered in trying to find appropriate clasts to use in the tests – they had to be clean in order to prevent extraneous particles being washed off them during the test, but cleaning them could easily cause them to disintegrate.

	Slake Durability Index (%)	
Material	I_{d2}	I_{d1}
fresh	43	76
stockpiled	51	78
5 year weathered	58	71
15 year weathered	80	85

Table 4.1: Slake durability indices for four bulk samples of shale from Gale Common.

Partical size distribution curves for the sites are presented in Figure 4.4, and relevant parameters summarised in Table 4.2 below.

location	sample	clay fraction (%)	D_{10} (mm)	C_u	C_c
Gale Common	fresh	1	0.8	25	2.25
	stockpile	2	0.015	1133	8.82
	5 year	4	0.009	522	8.51
	15 year	5	0.005	200	8
Grassholme Dam	GH1	1	0.025	112	1.75
	GH3	1	0.021	119	1.19
	GH5	<1	0.15	22.7	0.96
Balderhead Dam	BH2	<1	2.50	13.6	1.2
	BH4	<1	0.15	106.7	3.8
	BH6	<1	0.19	136.8	8.3
	BH8	<1	0.02	789.5	17.0
	BH10	<1	0.03	128.6	96.6
Grassholme - Hury tunnel spoil heap	1-1	6	0.004	150.0	0.14
	1-5	7	0.004	95.2	17.8
	1-7	3	0.008	56.2	0.25
	1-9	2	0.023	282.6	0.15
	1-11	3	0.020	500	4.1
	2-1	1	0.02	100	0.81
	2-2	1	0.10	50	8
	2-3	1	0.05	120	6.5
	2-4	1	0.60	33	3

Table 4.2 : Clay fraction ($< 2\mu m$), effective particle size (D_{10}), coefficient of uniformity (C_u) and coefficient of curvature (C_c) for samples from Gale Common, Balderhead Dam, Grassholme Dam and Grassholme Hury tunnel spoil heaps.

Particle size analysis of bulk samples from Gale Common clearly indicates a decrease in overall particle size of the shale fragments with time of exposure. Fresh shale arriving from Kellingley Colliery was found to contain the highest proportion of coarse material and shale fill from older parts of the embankment the lowest. It is not possible to determine how much of this apparent breakdown is due to natural weathering processes, and to what extent mechanical processes involved in transporting the material and constructing the embankment are responsible. Furthermore if one is to use decrease in particle size as an indicator of weathering then it has to be assumed that all material arriving at the site is similarly graded, and that any processes utilised in embankment construction do not cause any segragation or grading of the material.

Data from Grassholme Dam shows little variation with depth, although it should be noted that the samples were taken from a limited depth range. Overall the samples near to the top of the profile are slightly finer. This trend is shown very clearly by particle size distribution curves for both grassed and ungrassed parts of the Grassholme Hury tunnel spoil heaps. In both cases surface samples are considerably finer grained than those taken from depth, suggesting that weathering processes have been acting on the surfaces of the spoil heaps to break down the shale. Again it must be assumed that tipping spoil onto the heap did not cause any segregation of the different sizes.

No obvious trend is visible in the data from Balderhead Dam. Presumably the presence of the topsoil layer has a blanketing effect, and protects the shale against atmospheric weathering agents. The same process is probably responsible for the small variation seen in samples from Grassholme Dam.

4.6.3 X.R.F. Analysis

Samples were analysed for major and trace elements using standard X-Ray Fluoresence method, after Jenkins and de Vries (1967). Approximately 4.0g of powder of each sample produced in a TEMA swingmill with tungsten carbide vial, was pelletised by mixing with a few drops of PVA solution (MOVIOL) and compressing in a hydraulic press at approximately 6 tonnes. Pellets were run through the machine, a Phillips PW212 Automatic Sequential X-Ray Fluorescence Analyser using a Rhodium 3kW X-Ray tube at 80kV and 35mA, and the counts compared to a number of standard samples of known composition using computer software developed in the Department of Geological Sciences of Durham University. Readings were further corrected for loss on ignition (CO_2 and H_2O). Four sets of twelve identical samples were run through the machine in order to confirm precision of the results - these are tabulated in Appendix I. Three standards of known composition were also run blind to check the accuracy of the machine. Results are tabulated in Appendix II and plots of elemental concentration against depth given in Figures 4.7 to 4.12. It should be noted that although vertical scales are the same in each of these figures, the horizontal scales vary and thus care must be taken if comparing profiles from different sites.

4.6.4 X.R.D. Analysis

Standard methods of X-Ray Diffraction Analysis (Hardy and Tucker, 1989, Brindley and Brown, 1980) were used to identify mineralogical composition of samples. Samples were crushed and smear mounts prepared by mixing with acetone and placing on a glass slide. Some workers recommend use of water rather than acetone as this allows slower settling, partial orientation of clay particles and theoretically a stronger response. However Billing (1987) found that measured intensities were not significantly improved by using distilled water. Therefore acetone was chosen because the method of preparation is quicker and more convenient. The machine used was a Phillips PW1130 2kW Generator Diffractometer, using CoKα radiation at 40kV and 20mA.

Qualitative analysis is relatively straightforward for clay minerals and relies on comparison of peak positions on the diffractogram to data given by the Joint Committee on Powder Diffraction Standards (JCPDS) to identify individual minerals. However for weathering studies (semi-)quantitative data is necessary. The X-Ray response of a mineral, and therefore the intensity of peak produced depends on

a number of variables and therefore direct comparison of peak height or area is not possible. A series of calibration curves (Klug and Alexander, 1948), in which the responses at different concentrations against the concentration of a mineral are plotted, is neccessary. There are several methods of doing this. A mineral present in the system can be varied and then a direct calculation method applied (Hooton and Giorgetta, 1977). Alternatively, a constant amount of a new mineral can be added to the system to act as a standard and the response of varying concentration of any mineral in the system can be compared to this known constant. This method was used succesfully by Smith (1978). Boehmite was used as the internal standard (Griffin, 1954), its advantages being that it shows a convenient peak reflection which does not interfere with any of the peaks of minerals being analysed for. It has a similar total mass absorption coefficient to the minerals under analysis and gives a strong intensity when small amounts are present. X-Ray diffractograms for samples of shale with and without 10% boehmite present are shown in Figure 4.5. A series of samples with the following compositions was made up:
$(90 - x)\%$ shale + $x\%$ mineral + 10% boehmite
with $x = 1\%, 2\%, 5\%, 10\%, 15\%, 20\%, 30\%, 40\%$ and 50%

For each different mineral, plots of the peak area ratio of the mineral to that of boehmite, were plotted against the concentraion of the mineral. The resulting calibration curves are given in Appendix III. A typical Carboniferous shale was used as a basis for the standard samples with additional appropriate commercially available minerals. Gibbs (1967) suggests that the minerals to be added should be extracted from the samples to be tested to ensure maximum accuracy. Measurements of peak areas (rather than heights) was made using a polar planimeter. One disadvantage of this method is that the areas measured from a baseline, the drawing of which is subjective – errors were reduced by keeping its position standard on each occasion.

10% boehmite was added to all unknown samples and the calibration curves used to estimate the proportions of major minerals present. Results are tabulated in Appendix IV and plots of mineral composition against depth given in Figure 4.7 to 4.12. In most cases the total percentage measured was less than 100% and the remainder has been labelled 'miscellaneous'. This comprises a mixture of carbon, degraded clays and mixed layer clays although secondary minerals such as goethite and siderite have also been identified – these are indicated on the tables of results in Appendix IV. The amount of carbon present is determined by loss on ignition analysis in which the sample is heated and the weight loss due to the driving off of volatiles is measured. The results are given in Appendix II with the major element chemistry. On most charts, the illite 10Å peak is asymmetrical with a tail on the low 2θ angle side, this is caused by reflections from the degraded illite and mixed layer clays, and is illustrated in Figure 4.6. The tail can be removed during glycolation and heating as explained below, to leave a symmetrical illite peak. The quoted value for illite is taken from this peak, that is pure illite, and the degraded illite and mixed layer clays included within miscellaneous. A number of geometrical indices are available to measure the crystallinity of illite based on the symmetry of its peak, see for example, Hardy and Tucker (1989). Several workers have used such indices as weathering indicators (Taylor (1971), Smith (1978), Russell and Parker (1979), Billing (1987) amongst others) noting that increased weathering towards the top of profiles at both natural and man made sites leads to degradation of illite. Smith (1978) noted similar phenomena in kaolinite although this was not obverved in samples from this study. Illite shape factors for the samples analysed given in Table 4.3.

sample	shape factor	sample	shape factor	sample	shape factor	sample	shape factor
GCA1	0.36	GCL1	0.95	BH1	0.96	Beeston	0.23
GCA2	0.50	GCL2	0.79	BH2	0.23	Silkston	0.14
GCA3	0.10	GCL3	1.59	BH3	0.64		
GCA4	0.27	GCL4	0.61	BH4	0.33	GCF coarse	0.15
GCA5	0.33	GCL5	0.47	BH5	0.31	GCS coarse	0.18
GCA6	0.30	GCL6	0.49	BH6	0.19	GC5 coarse	0.24
GCA7	0.20	GCL7	0.58	BH7	0.50	GC15 coarse	0.82
GCA8	0.33	GCL8	0.54	BH8	0.20		
GCA10	0.23	GCL9	1.05	BH9	0.21		
GCA11	0.22	GCL10	0.42	BH10	0.19		
GCA12	0.25	GCL11	0.77				
GCA13	0.33	GCL12	0.39	GH1	1		
GCA14	0.21	GCL13	0.59	GH2	1		
GCA15	0.15	GCL14	0.75	GH3	1		
GCA16	0.23			GH4	1		
GCA18	0.20			GH5	1		
GCA20	0.24			GH6	1		

Table 4.3 : Illite shape factors for samples from Gale Common five year weathered (GCA) and fifteen year weathered (GCB), Balderhead Dam (BH) and Grassholme Dam (GH).

Certain workers recommend separate analysis of the clay fraction (passing $2\mu m$). Towe (1974) points to possible dangers in this. Methods of clay analysis are given in Hardy and Tucker (1989) and in Wilson (1987). The method adopted in this work was as follows: The passing $2\mu m$ fraction obtained from sedimentation analysis was allowed to settle on to a glass slide by evaporation in a warm oven. This produced oriented sections with stronger reflections. Some interference of peaks occurs, so after an initial run, the slides were first glycolated by placing in a desiccator containing glycerol in an oven at $60°C$ for 4 hours. This expands smectites to 17Å so that the reflection does not interfere with that of chlorite 14Å. Heating to $375°C$ collapses smectites and illites to 10Å and further heating to $550°C$ destroys kaolinites whose 7Å peaks may interfere with second order chlorite peaks. These methods were used to confirm the clay mineralogy as measured on the whole rock samples.

4.6.5 Mineralogy and Chemistry of the sites studied

In general, at first glance, there appears to be little variation in mineralogy with depth. The distribution of trace elements in particular is remarkably consistent with depth. The average mineralogy and major element chemistry for the sites is shown below in Tables 4.4 and 4.5. In most cases there is an apparent inverse correlation between illite and degraded illite/mixed layer clays, but only in the case of Balderhead Dam does this also appear to be influenced by depth. This also correlates with the illite shape factors, and shows there to be more weathering and degradation at the tops of the profiles.

sample	Mineralogy (% weight)					
	quartz	illite	kaolinite	chlorite	mixed layer clays	other
fresh Coal Measures shale (Tanners Hall)	11	55	15	5	14	
fresh Kimmeridge Clay	15	20	10	-	20	35% calcite
fresh Coal Measures shale (Kellingley)	23	54	19	1		
Gale Common 5	22	44	12	1		
Gale Common 15	22	27	12	1		
Balderhead	25	21	22	1		
Grassholme	28	16	15	1		
Burnhope	28	19	15	1		
Tunstall	28	13	8	1		
Waskerley	26	27	16	1		
Smiddy Shaw	41	31	14	1		
CD (model dam)	18	30	24	1		

Table 4.4 : Average mineralogy of samples.

	Element (% weight)										
sample	SiO_2	Al_2O_3	Fe_2O_3	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
BR	57.6	21.8	4.7	1.7	0.3	0.8	3.6	1.1	tr	0.1	8.3
SR	59.5	20.4	5.0	1.5	0.3	0.8	3.6	1.1	0.1	0.2	7.8
GCF	40.3	18.1	4.7	1.1	0.6	0.4	2.8	0.8	tr	0.1	19.2
GCS	40.3	18.4	4.0	1.0	0.4	0.5	2.8	0.8	tr	0.1	30.6
GC5	42.9	18.9	5.1	1.3	0.4	0.4	3.2	0.8	tr	0.1	27.5
GC15	43.9	18.9	5.9	1.2	0.3	0.3	3.0	0.9	tr	0.1	24.6
BH	49.8	20.5	6.1	1.5	3.1	0.2	2.3	1.0	0.1	0.1	14.8
GH	52.3	20.9	8.7	1.3	0.2	0.2	2.1	0.8	0.1	0.2	12.8
GHSH	59.6	21.2	7.8	2.3	4.4	0.3	2.8	0.9	0.1	0.2	0.4
CD	40.4	19.0	3.7	0.8	0.4	0.1	2.4	0.9	tr	0.1	31.5

Table 4.5 : Average chemistry of samples.

It is interesting to compare the mineralogy of samples from Beeston and Silkstone roof rocks taken direct from Kellingley Colliery (underground samples were supplied by British Coal) with other samples from Gale Common. The underground samples contain few mixed layer clays and the illite peaks are sharp and symmetrical. Bulk samples from fresh, stockpiled, five year and fifteen year weathered samples contain progressively higher amounts of mixed layer clays, less pure illite and have higher shape factors. The underground samples contained little calcite and no pyrite or gypsum; various amounts of all three are present in the weathered rocks, although as mentioned earlier the presence of such minerals is generally sporadic and therefore their presence or absence cannot be used to infer any weathering effects. Samples near the surface of the five and fifteen year profiles show enhanced quartz contents, that is, higher than those measured in the fresh and underground sampes, although this tends to decrease to the expected value with depth and may represent leaching of clays at the surface. Coarse gravel sized clasts from the bulk sampes were also analysed separately – the results are in Appendix IV. In fresh, stockpiled and five year weathered samples the mineralogy is close to that in the underground samples. However, in the fifteen year weathered sample the clasts contain more degraded illite and mixed layer clays and it appears that, given time, the larger clasts start to become weathered as well as the matrix. Furthermore samples of shale from the Gale Common embankment itself contain significant amounts of carbon – this is derived from coal fragments which are visible in the fill material and are indicated by high loss on ignition values.

Namurian shales from Balderhead and Grassholme Dams contain approximately equal amounts of illite and kaolinite, although the latter also contains a high proportion of mixed layer clay and degraded illite, this is reflected in the higher shape factors. Samples from the remaining dams are similar in composition to the Coal Measures shale with the clay mineral assemblage being dominated by illite. The high quartz value observed in the fill from Smiddy Shaw Dam is probably related to fragments of sandstone which were observed in excavations on the dam. The lack of calcite in samples from Grassholme compared to that in samples from Balderhead may be an expression of the longer time of exposure, but it is not possible to say for sure whether the mineral was present in shales used at Grassholme in the first instance.

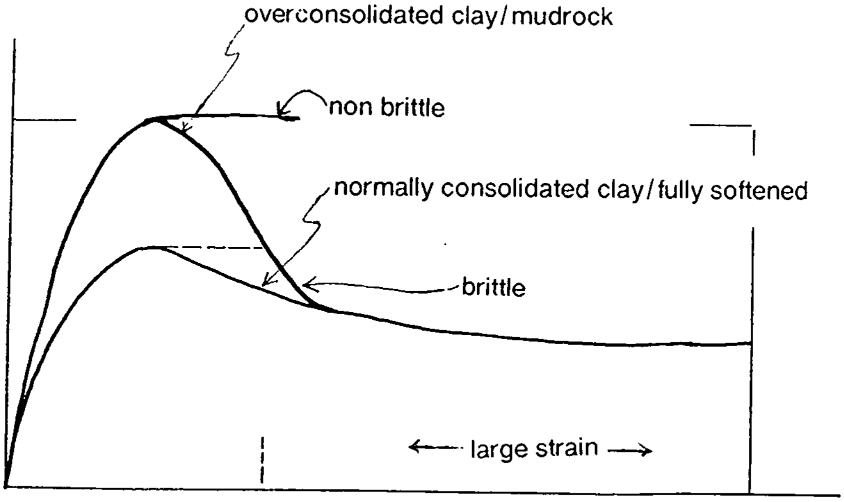


Figure 4.1 Effective peak and residual shear strength parameters for progressively weathered mudrocks.

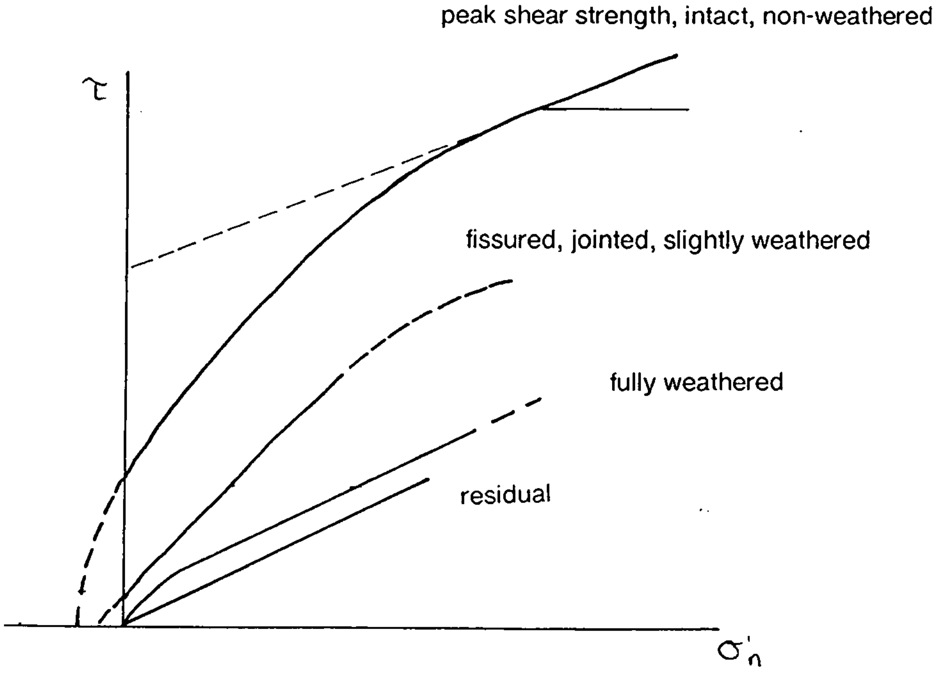
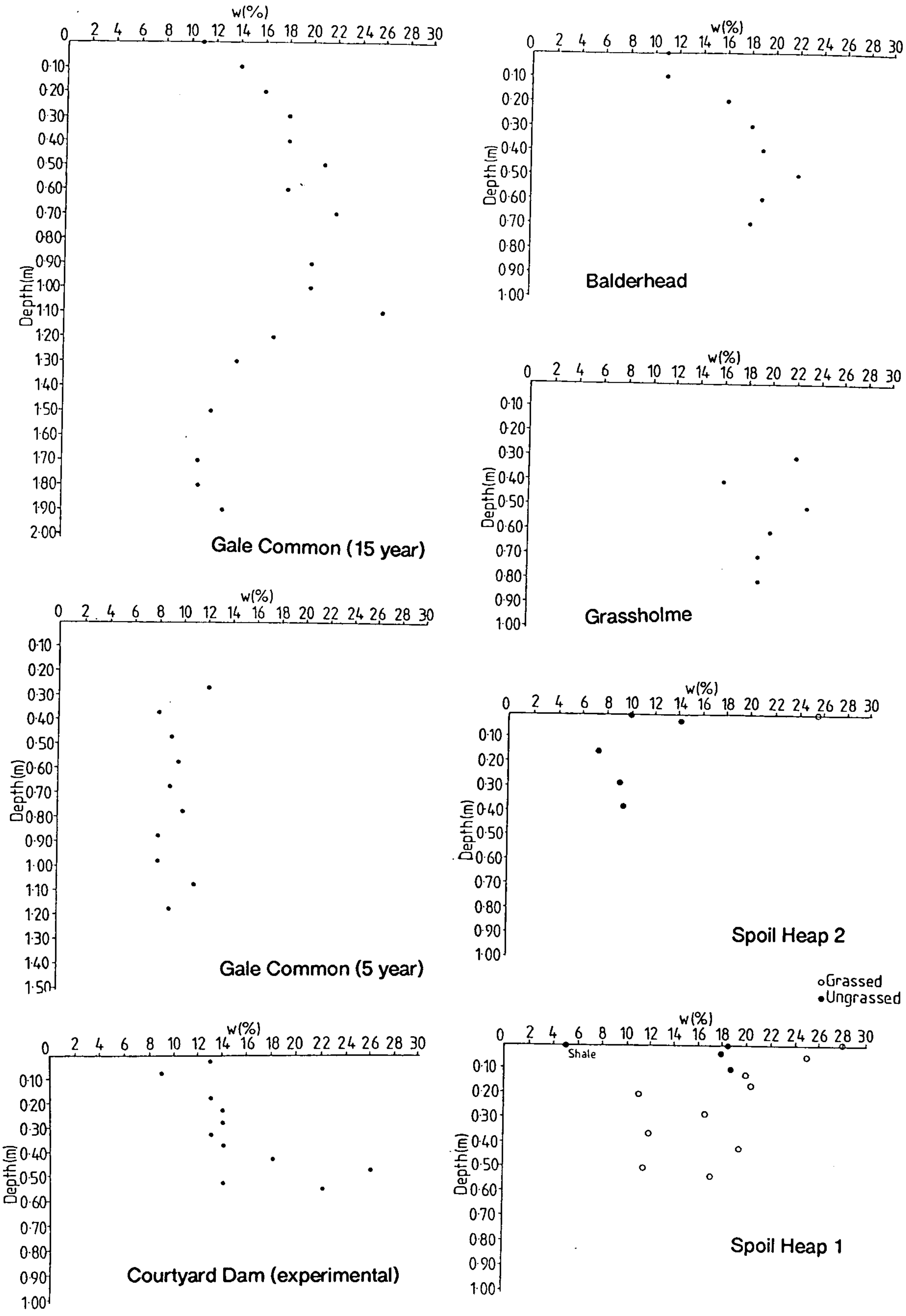


Figure 4.2 Shear strength – displacement curves for overconsolidated and fully softened clays.



Gale Common (15 year)

Balderhead

Grassholme

Gale Common (5 year)

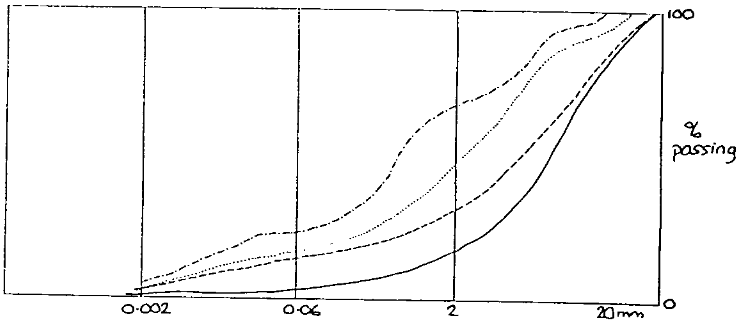
Spoil Heap 2

Courtyard Dam (experimental)

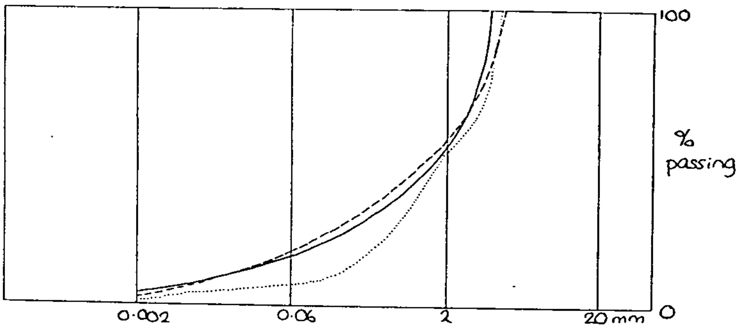
Spoil Heap 1

Figure 4.3 Moisture content – depth relationships.

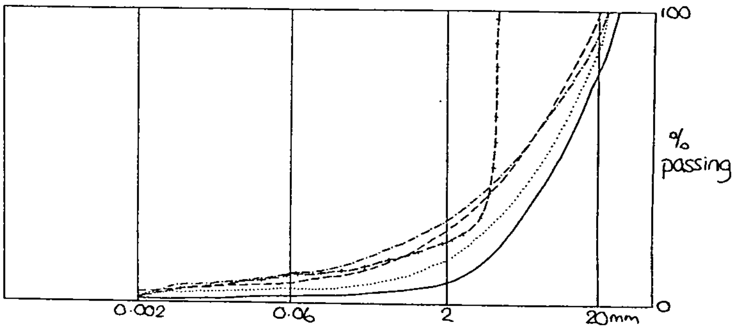
(a) Gale Common



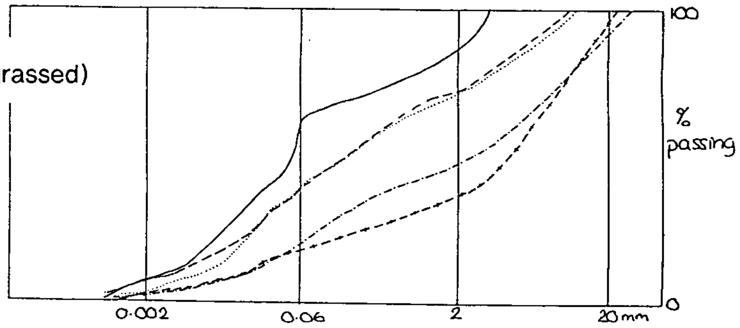
(b) Grassholme Dam



(c) Balderhead Dam



(d) Grassholme Hury tunnel spoil heap (grassed)



(e) Grassholme Hury tunnel spoil heap (ungrassed)

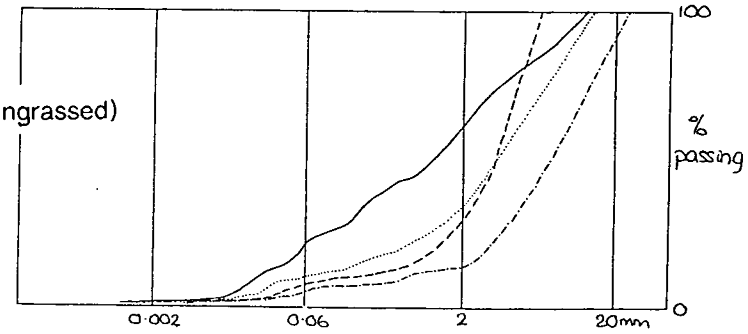


Figure 4.4 Particle size distribution curves.

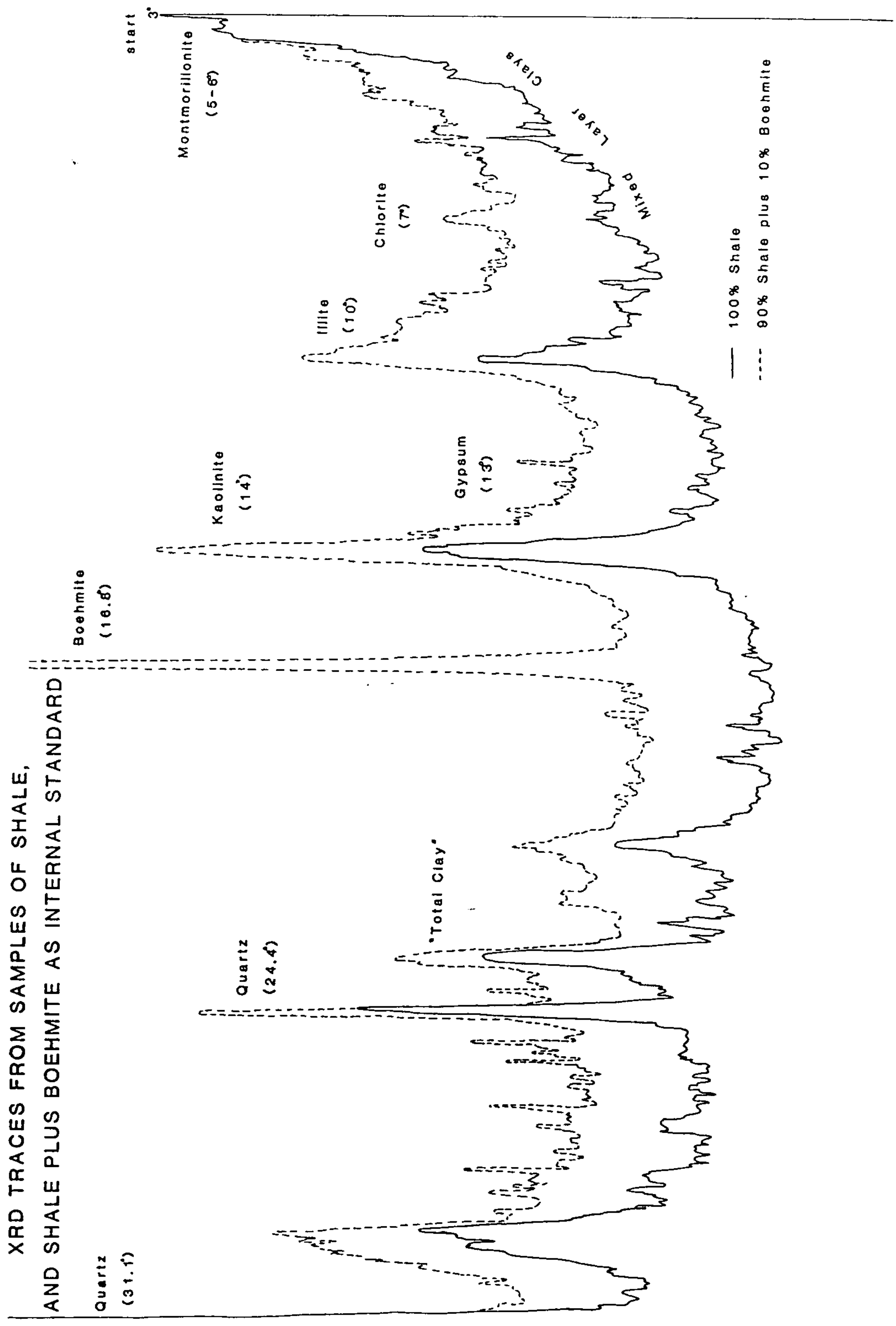


Figure 4.5 X-Ray diffrctogram for Carboniferous shale with and without the addition of 10% boehmite.

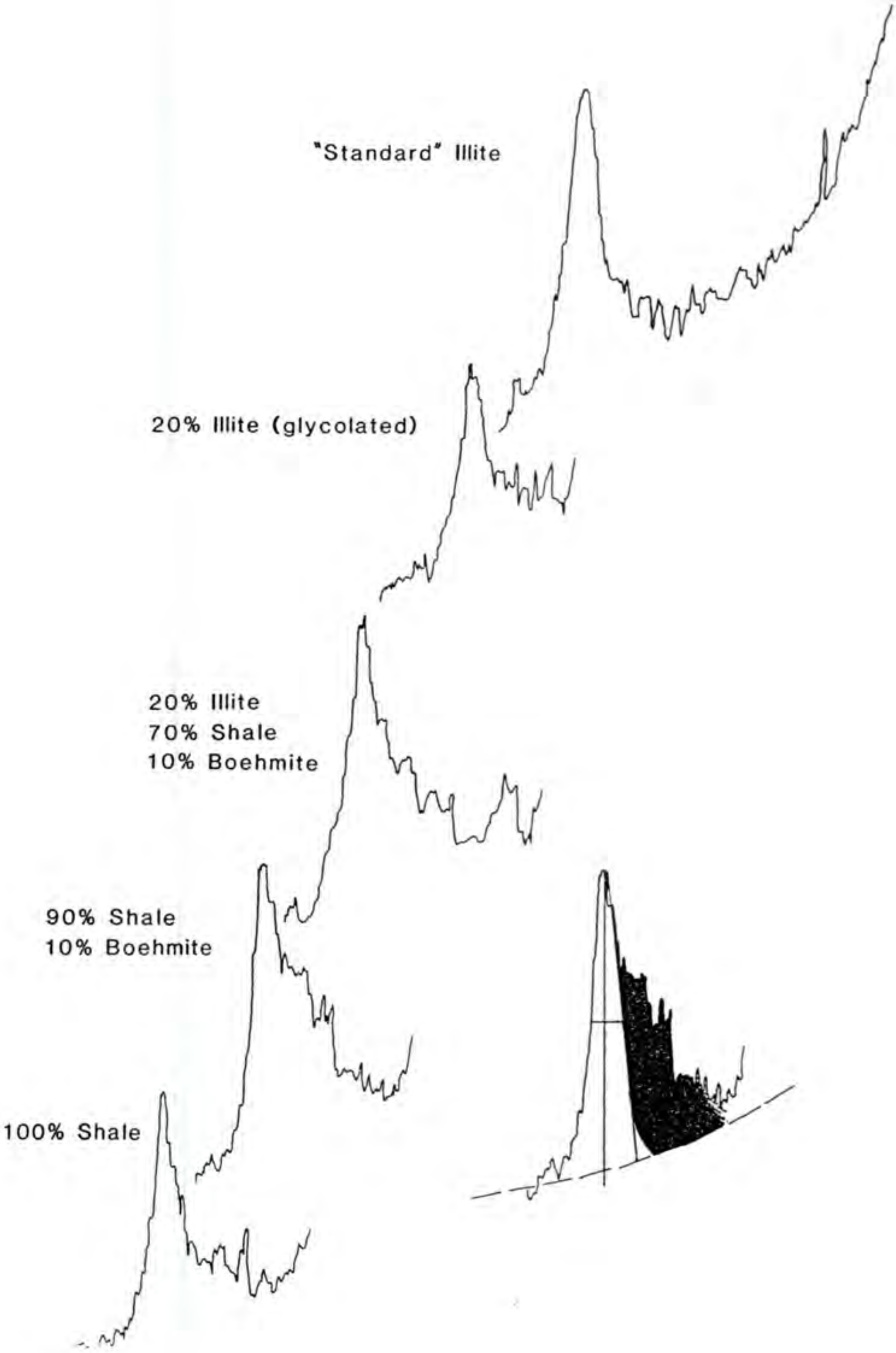


Figure 4.6 Illite Crystallinity (not to scale - diagrammatic representation of change in shape only)

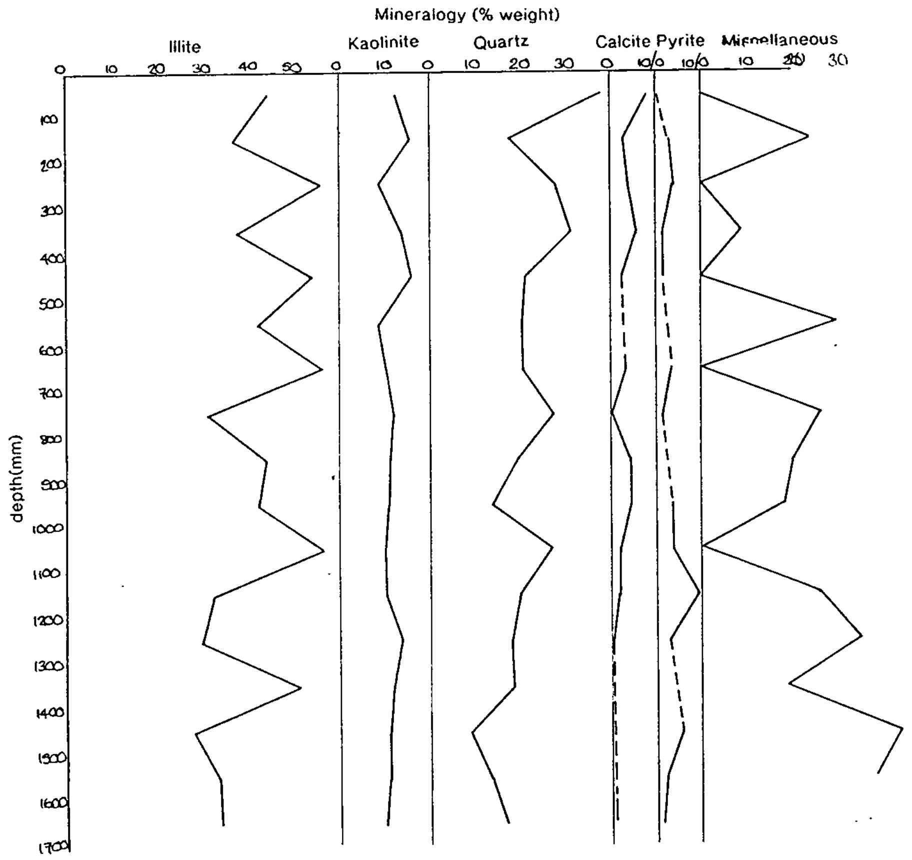
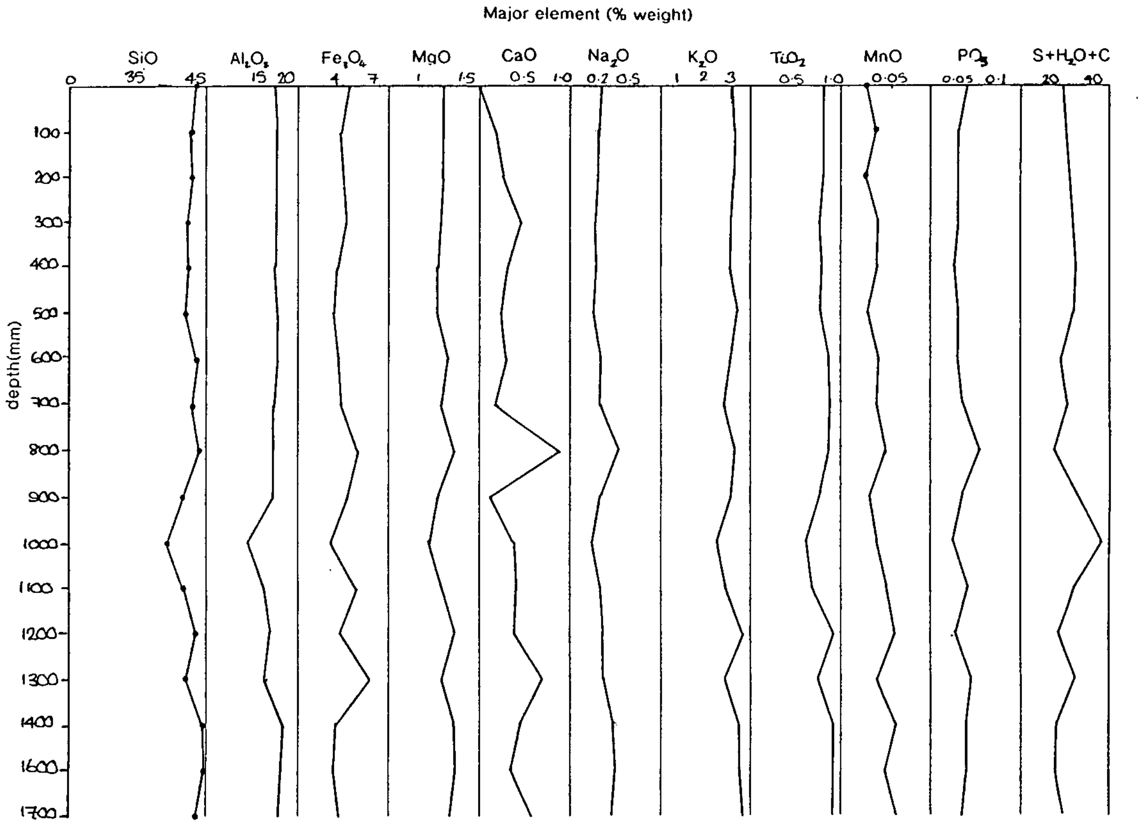
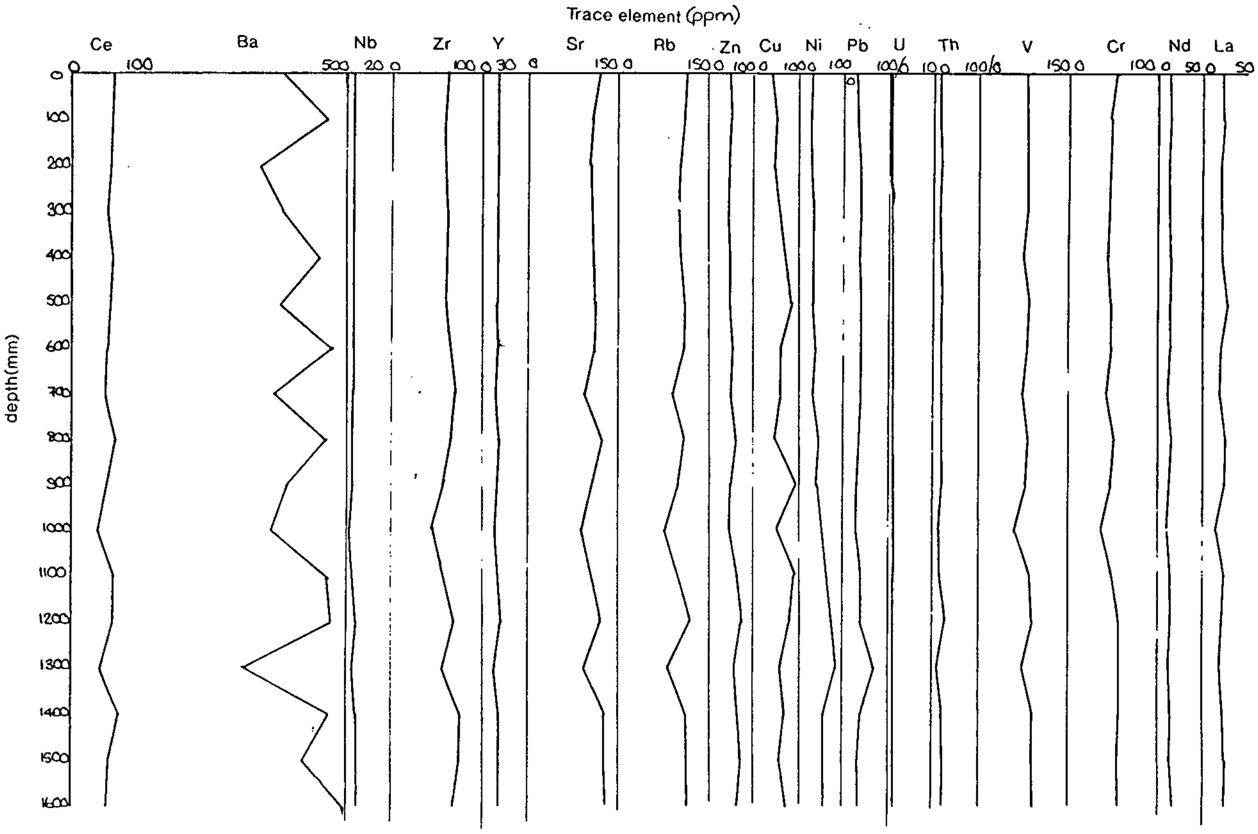


Figure 4.7 Mineral content, major element and trace element – depth: Gale Common five year weathered.


Major element (% weight)
SiO
Al₂O₃
Fe₂O₃
MgO
CaO
Na₂O
K₂O
TiO₂
MnO
P₂O₅
S+H₂O+C
0
35
45
15
20
4
7
1
1.5
0.5
1.0
0.2
0.5
1
2
3
0.5
1.0
0.05
0.05
0.1
20
40
depth (mm)
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1600
1700


Trace element (ppm)
Ce
Ba
Nb
Zr
Y
Sr
Rb
Zn
Cu
Ni
Pb
U
Th
V
Cr
Nd
La
0
100
500
20
0
100
0
30
0
150
0
150
0
100
0
100
0
100
100
10
0
100
0
150
0
100
0
50
0
50
depth (mm)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600

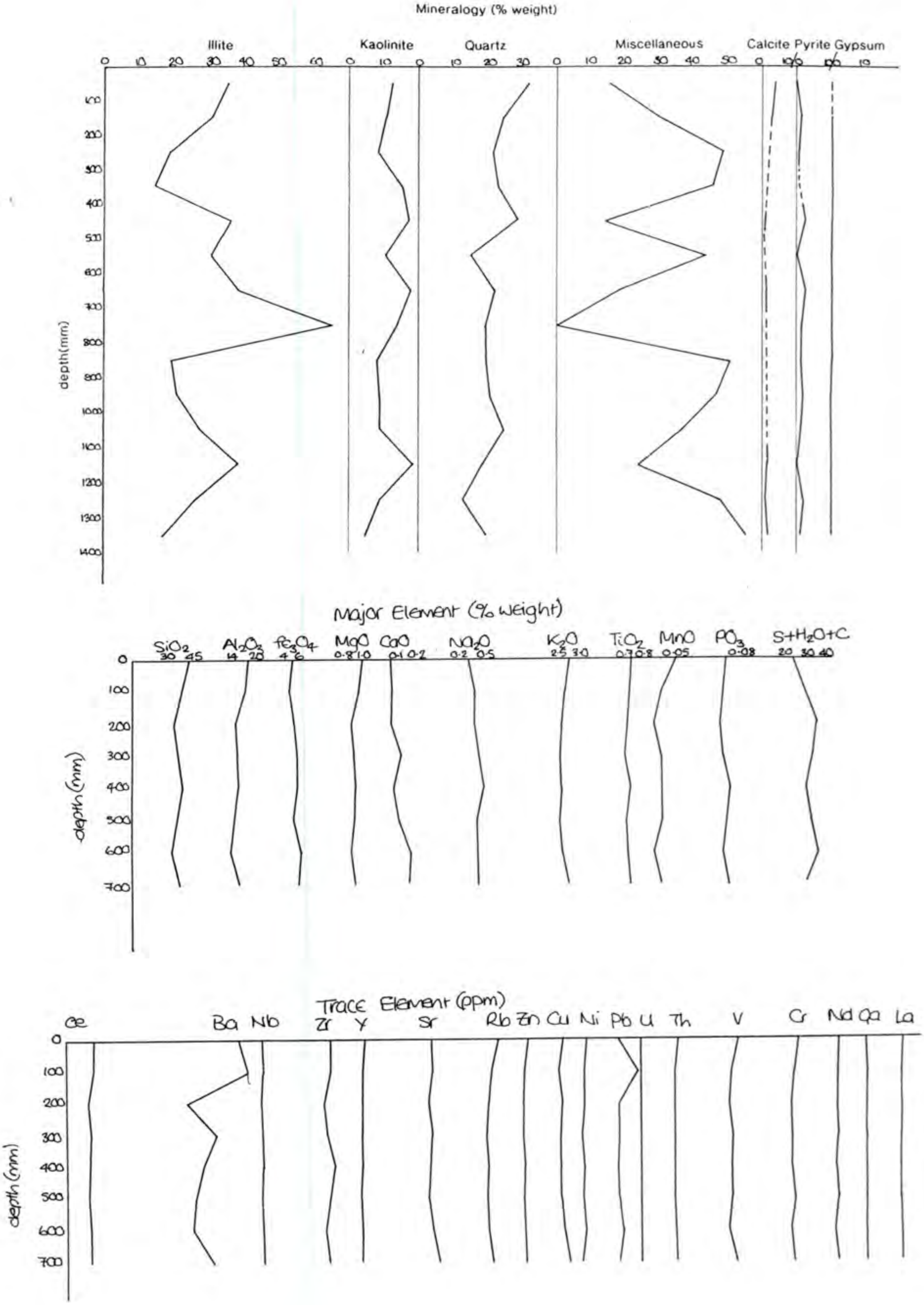


Figure 4.8 Mineral content, major element and trace element – depth: Gale Common fifteen year weathered.

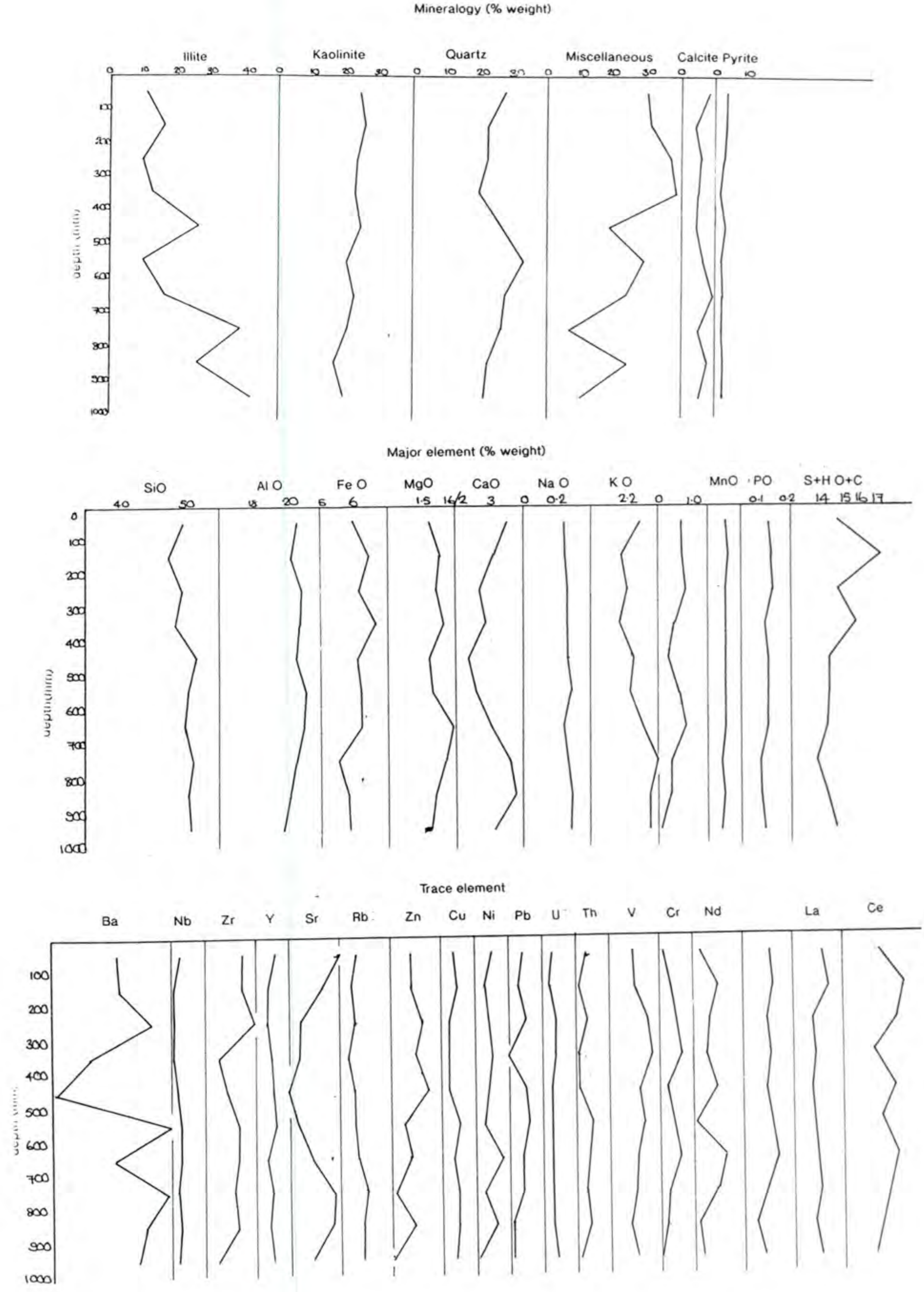


Figure 4.9 Mineral content, major element and trace element – depth: Balderhead Dam.

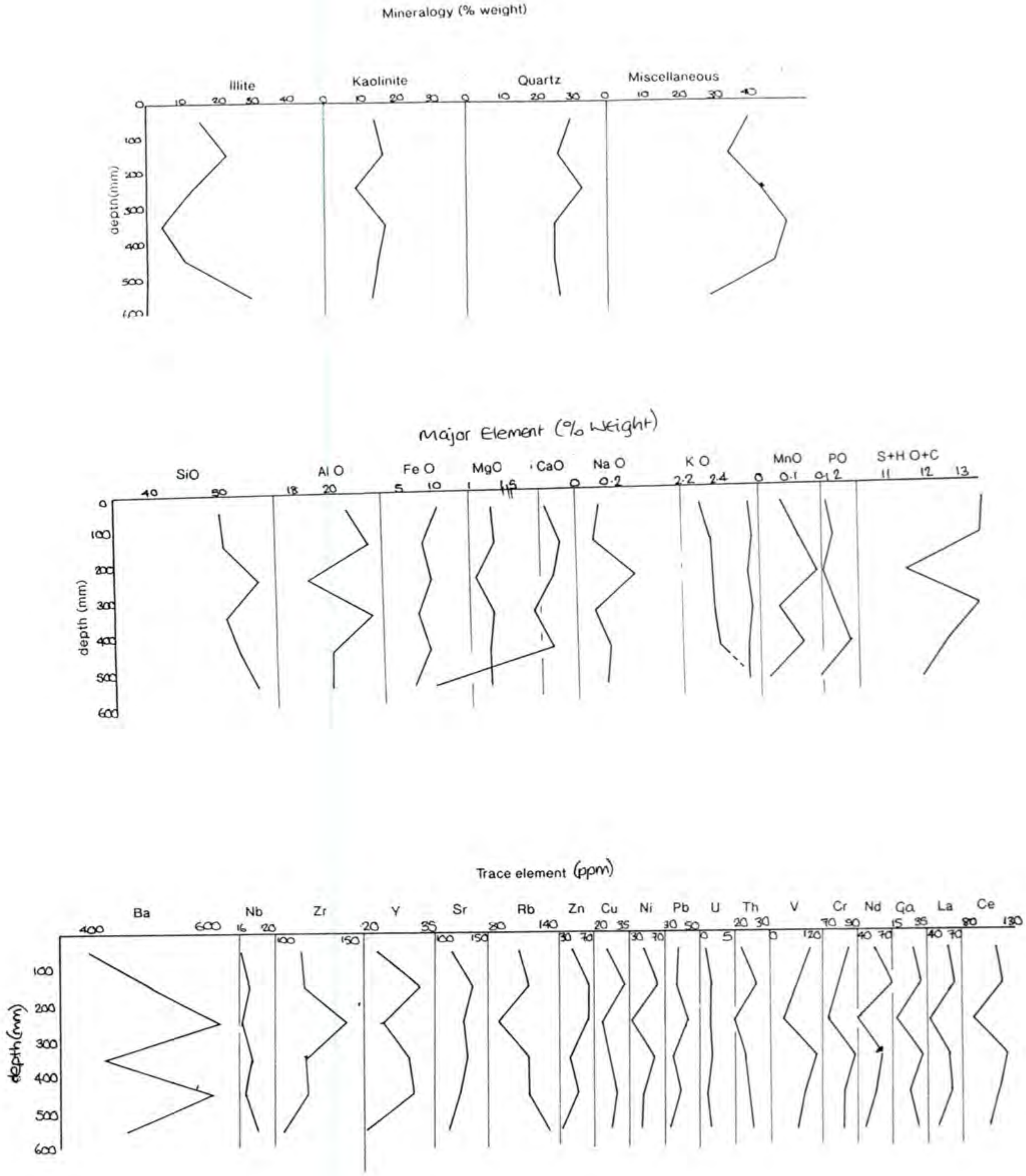


Figure 4.10 Mineral content, major element and trace element – depth: Grassholme Dam.

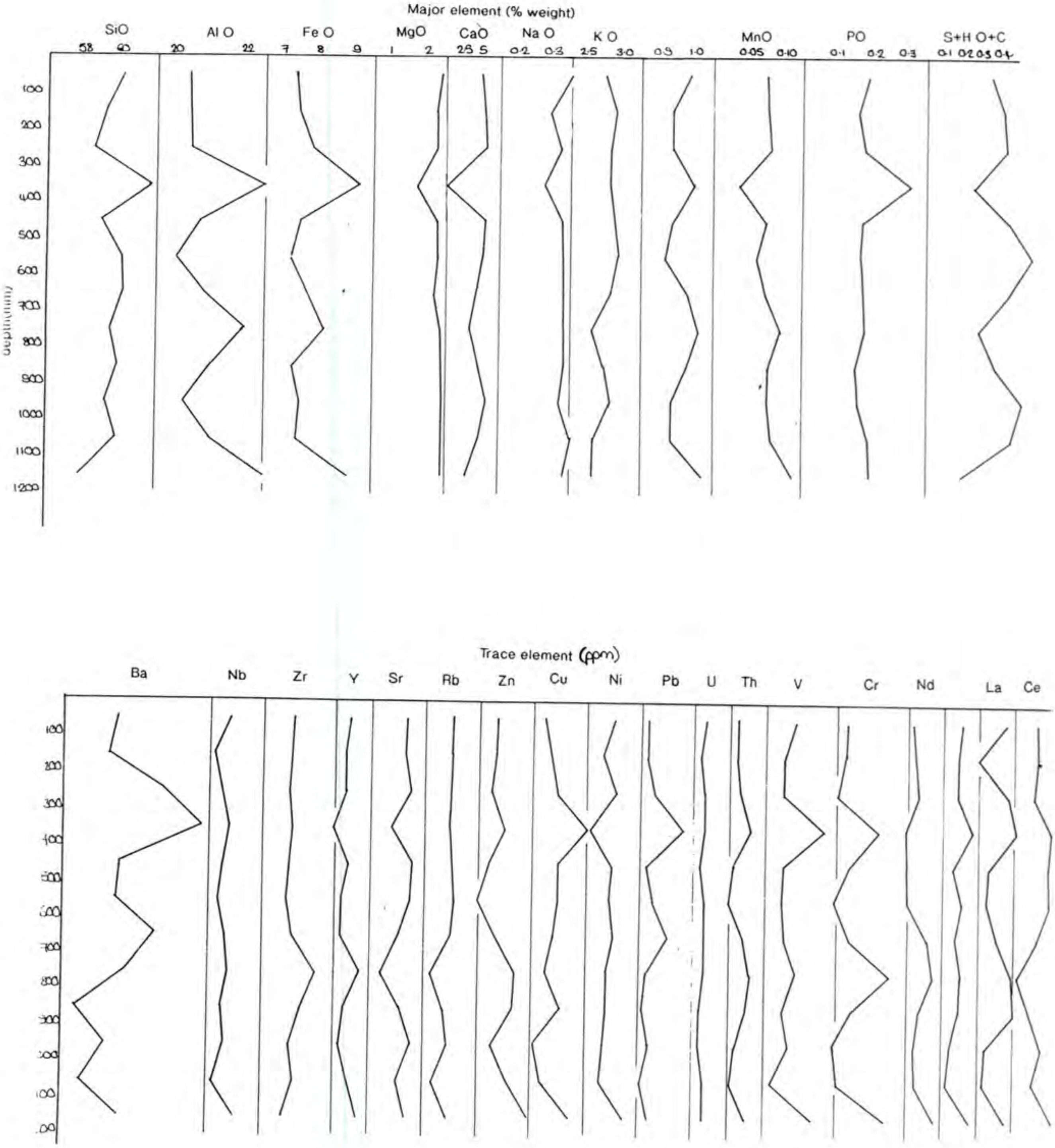


Figure 4.11 Mineral content, major element and trace element – depth: Grassholme Hury tunnel spoil heap

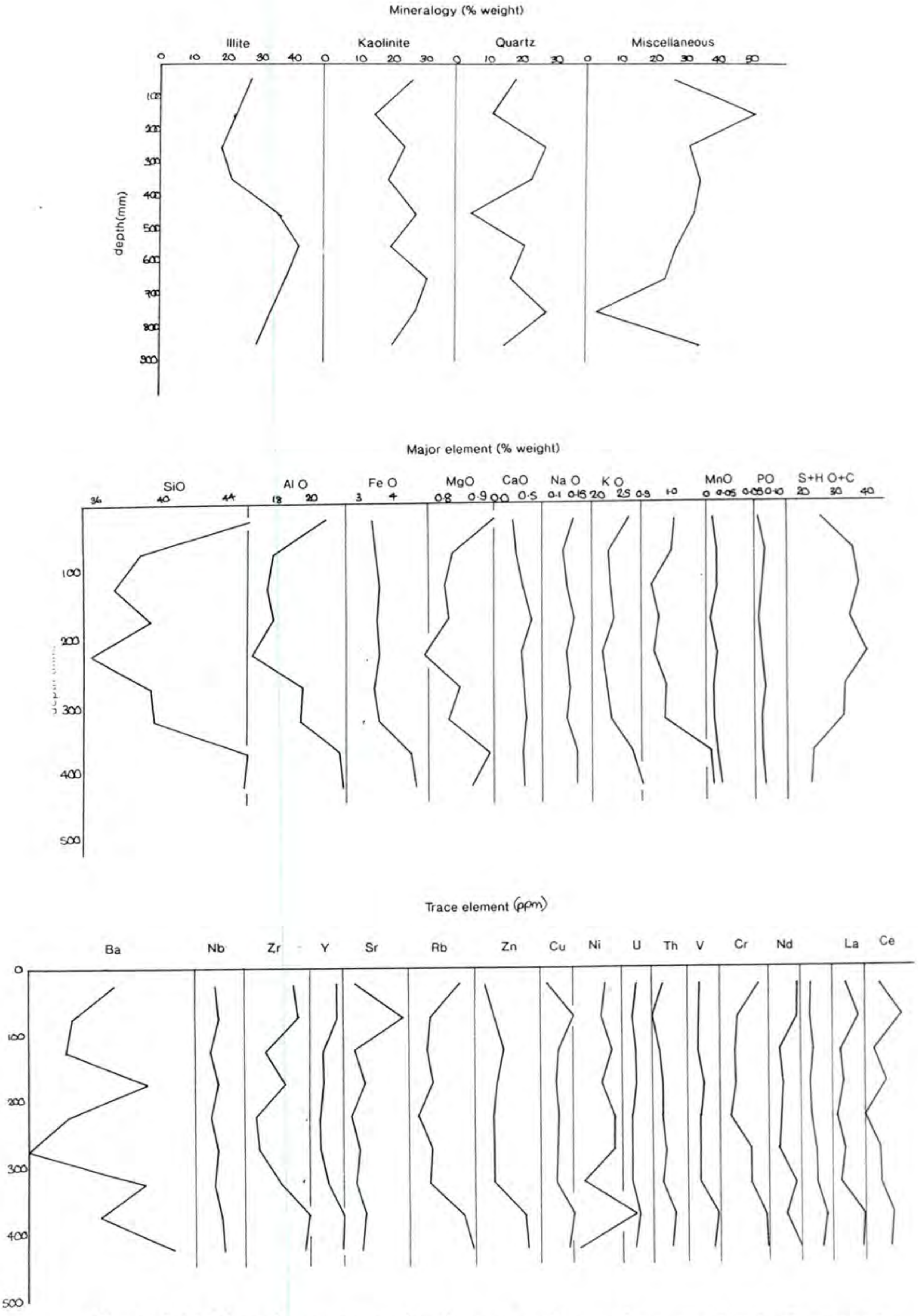


Figure 4.12 Mineral content, major element and trace element – depth: Courtyard model dam.

5. BRITTLENESS AND PROGRESSIVE FAILURE

5.1 Introduction

5.1.1 The property of brittleness

According to Gordon (1968), the worst sin that an engineering material can have is not lack of stiffness or strength, but lack of toughness: that is, to be brittle. Whereas weakness in a material can be predicted and catered for at the design stage by allowing for excessive elastic deformation and plastic yielding, brittleness cannot be treated in this way.

By studying the shape of the stress – strain graphs for various materials, three types of post-failure behaviour as shown in Figure 5.1 have been recognised:

1. work hardening
2. perfectly plastic
3. work softening or brittle

Brittleness has also been defined as an inability to resist the propagation of cracks and is characterised by unstable situations in which a crack, once initiated, will accelerate rapidly. This may lead to sudden and catastrophic failures.

The Griffith Theory (1921) illustrates one of the essential factors of brittle behaviour, that is, that a material will fail at unpredictable levels of applied stress which may be significantly lower than the general yield strength of the material. Griffith accounts for this by assuming the presence of many fine elliptical cracks throughout the material. Calculations of stress concentrations at the tips of these cracks show that locally the yield strength is reached, thus leading to failure at that point and progressive propagation of the crack. In reality, microflaws in a material act as stress concentrators. Fracture mechanics theory shows that cracking tends to proceed along the most energetically favourable route available. This is towards the major principal stresses in an isotropic and homogeneous material, but tends to be parallel to cleavage planes or along grain boundaries where these are present.

To summarize, a brittle material is one in which the ability to resist load decreases beyond the yield point ('failure'). This may occur at isolated points within the body of the material, thus allowing a chain reaction of localised failures. This leads to brittle behaviour which occurs at lower than anticipated stress levels.

5.1.2 Rock deformation in the laboratory

Jaeger and Cook (1969) describe the 'brittle state' in rocks as that in which the ability to resist load is lost with increasing deformation. During a typical conventional unconfined compression test on a cylindrical rock specimen a graph of stress against radial strain can be plotted and will show the following features. First, as stress is applied there is an overall decrease in volume thought to be caused by the closure of pre-existing cracks. Second, however, at approximately 40% of the peak stress value, dilation begins to occur as micro cracking proceeds – once the amount of dilation exceeds that of contraction the overall volume of the sample begins to increase. During this brittle or strain softening stage, failure is occurring continuously throughout the sample which, despite its highly disordered internal structure, remains intact. Further application of stress causes a large permanent set. Lastly, at its residual state, the sample is no longer intact, and any strength is largely derived from inter-particle friction.

By fixing microphones around such test specimens, Obert and Duval (1957) measured what they term 'micro-seismics'. These are the result of vibrations produced when elastic strain potential energy is dissipated during crack growth and is analagous to the cracking sounds emitted from glaciers. It has been shown that the amount of micro-cracking is directly responsible for the micro seismic activity and correlates with the dilation effects discussed above. The frequency of these acoustic events is apparently random up to stresses at around 90% of the peak value, and then is directly proportional to the stress up until failure.

One problem with attempting to observe brittle behaviour is that of machine – sample interaction (Hudson, 1972). As elastic energy is stored up in the sample it must also be held in the machine so that the whole system remains in equilibrium. When the specimen fails, the elastic energy from the machine columns is dissipated through the specimen, hence a fracture is induced almost immediately after peak stress is reached. If a stiff testing machine is employed then the strain energy * accumulated is much lower, hence less energy is available to be put back into the system at failure and the whole range of behaviour can be observed – the ability of a partially failed rock to withstand load is greater in stiff machines or when surrounded by rock. In the field, surrounding rock provides the necessary stiffness required to prevent sudden and explosive failure from occurring. This is one reason why laboratory testing does not adequately reproduce field conditions. Others include the rate of and time taken for loading, and the fact that results from tests are strongly affected by both the size and shape of specimen as well as end effects.

Although laboratory testing may be valid in terms of short term structures, care must be taken especially when natural structures such as faults are being considered - a technique criticized by Price (1966), amongst others, who suggests that it is wrong to invoke such criteria and mechanisms in the field. The processes behind fault development may be at least partly ductile, especially in the lower crust where temperature and pressure conditions are very different from those used in laboratory testing.

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* Strain energy is equal to $\frac{\sigma^2}{2E}$, where σ = stress and E = Young's Modulus

5.2 Brittleness in Soils

The preceding discussion refers to solid materials. The following section deals with cohesive and granular soils, and it is assumed that earth and rock fills, being particulate in nature, will also be governed by the same factors.

5.2.1 Cohesive Soils

The strength of cohesive soils, that is soils where interparticle bonds and forces make a significant contribution to their behaviour, is derived from a combination of cohesion and interparticle friction. Cohesion itself is controlled by interparticle bonds and physico-chemical properties of the clay minerals present (for example, *see* Smith and Taylor, 1978). The destruction of such bonds during stressing leads to irreversible loss of strength and thus brittle behaviour.

Most naturally occurring clays will behave in a brittle manner under drained conditions with large strains being accommodated in relatively thin concentrated shear zones in which particles are orientated (Skempton, 1964). Peak shear strength envelopes are typically curved with a small cohesion intercept whereas residual shear strength envelopes tend to be linear with zero cohesion intercept. In this latter condition cohesive bonds are destroyed and strength is derived solely from interparticle friction. As the plasticity index (this reflects clay content) increases so particles tend to undergo sliding as opposed to turbulent behaviour (Lupini *et al*, 1978). As this effect increases so the residual strength becomes increasingly lower than the critical state strength.

Brittleness in undrained conditions is much more difficult to evaluate as, along with the factors discussed above, changes in pore water pressure also contribute to the drop in strength.

5.2.2 Granular (non-cohesive) soils

The behaviour of granular soils is somewhat simpler to assess since the presence of interparticle bonds and the effect of clay mineral reorientation are not significant and strength depends on the mean applied stress. Sands, gravels and rock fills fall into this category. Shearing resistance derives from interparticle friction and depends on the density, distribution and shape of the particles.

The post peak drop in strength under drained conditions is due to the dilation of particles until critical state is reached (residual and critical strength are equal in soils comprising rotund and non deformable particles). Dilation will not occur in very loose materials, and under high confining pressures the increase in volume is prevented. Hence in both cases non brittle behaviour is observed.

Again, undrained conditions present a somewhat more complicated scenario as porewater pressure is a contributing factor. Dense soils tend not to be brittle unless under enormous pressure, whilst loose materials may exhibit extreme loss of strength as liquefaction occurs.

Thus it can be seen that brittleness is an observable phenomenon in soils. Bjerrum (1967) used the term 'brittle' to describe stiff clays liable to progressive failure. In discussing brittle soils, Bromhead (1984), refers to the loss of strength which occurs when deformation is continued beyond the maximum load bearing capacity. Beyond this point, the shearing process is irreversible and the soil is left with a permanently impaired strength. Dounias (1987) concludes that most naturally occurring soils represent a hybrid of these two types and that all are brittle with the exception of undrained dense granular soils, and drained very loose granular types.

5.3 Definition and use of the Brittleness Index

Given that soils behave in a brittle manner, it is worth considering a method of quantifying the extent, and to give an indication of the likely magnitude of the consequences of brittle behaviour. Haefeli (1965) proposed the use of the Residual Coefficient (I_R) as a means of comparing peak and residual shear strengths.

$$I_R = \frac{\tau_R}{\tau_F} \qquad 5.1$$

where

τ_R = Residual shear strength

τ_F = Shear strength at failure

Bishop (1967) considered it premature to establish parameters to define this aspect of soil behaviour prior to any detailed study of shear surface formation under non-uniform stress conditions. The influence of such problems as fissures, intact lumps within a rock mass and suggested differences between clays and rock fills had yet to be ascertained. In the absence of such information the Brittleness Index was proposed. This expresses the reduction in strength in going from peak to residual state as a percentage of the peak value and is given in equations 5.2 a and b.

$$I_B = \frac{\tau_P - \tau_R}{\tau_P} \times 100\% \qquad 5.2a$$

where

τ_P = peak shear strength

τ_R = residual shear strength

or, for the undrained case;

$$I_B = \frac{C_{uP} - C_{uR}}{C_{uP}} \times 100\% \qquad 5.2b$$

where

C_{uP} = undrained peak shear strength

C_{uR} = undrained residual shear strength

An alternative way of observing this is to look at the additional energy involved in proceeding from the peak to the residual condition (Bishop 1967 and 1974). For example, the rupture index expresses the difference in work done to reach peak and residual as a percentage of that to reach peak.

Bishop (1973) quotes Brittleness Index when comparing case studies of a number of failure events. In a recent literature review some twelve papers were found which referred to the Brittleness Index. Of these, Highter and Page (1976) claimed to have found no values of the Index quoted in the literature. However, Bishop (1973), gives values for some typical soils, colliery waste and rockfill. These are summarised in Table 5.1. More recently there has been an upsurge of interest in the Brittleness Index, due to its application in the computer analysis of progressive failure (*see* Section 5.5).

Cohesionless soils, drained conditions				
material	state	test type	$\sigma_3(psi)$	$I_B(\%)$
Ham River Sand	very dense	axial comp	15	33
	very dense	plane strain	15	44
	dense	axial comp	100	18
	dense	axial comp	990	8
	loose	axial comp	40	3
Rockfill (granitic gneiss)	dense	axial comp	10	38
	dense	axial comp	41	22
	dense	axial comp	150	4
Rockfill (Silurian mudstone)	dense	axial comp	10	30,59
	dense	axial comp	40	14,50
	dense	axial comp	135	0,41
Colliery waste (Aberfan)	loose	axial comp	5 - 35	61
	loose	axial comp	10 - 90	52

Cohesionless soils, undrained conditions				
material	state	test type	$\sigma_3(psi)$	$I_B(\%)$
Banding Sand	v. loose	stress	4	94
Ottawa Sand	med loose	controlled	4	65
	med loose	axial comp	4	10
Huachipate Sand, Chile	v. loose	axial comp	4	91
	loose	axial comp	4	65
Ham River Sand	loose	axial	99.4	75
	loose	extension	90.0	75

Table 5.1: Values of Brittleness Index for cohesionless soils after Bishop (1973).

<table>
<tr><th colspan="6">Cohesive soils, drained conditions</th></tr>
<tr><th>material</th><th>state</th><th>PI</th><th>CF</th><th>$\sigma'_n(psi)$</th><th>$I_B(\%)$</th></tr>
<tr><td>Blue London Clay</td><td>intact</td><td>43</td><td>57</td><td>5</td><td>93</td></tr>
<tr><td></td><td></td><td></td><td></td><td>80</td><td>77</td></tr>
<tr><td></td><td>remoulded</td><td></td><td></td><td>30</td><td>52</td></tr>
<tr><td>Brown London Clay</td><td>undisturbed</td><td>42</td><td>40</td><td>5</td><td>81</td></tr>
<tr><td></td><td></td><td></td><td></td><td>40</td><td>71</td></tr>
<tr><td></td><td>slurried</td><td>43</td><td>64</td><td>131</td><td>63</td></tr>
<tr><td>Boulder Clay (Selset)</td><td>undisturbed</td><td>13</td><td>17</td><td>5</td><td>34</td></tr>
<tr><td></td><td></td><td></td><td></td><td>40</td><td>12</td></tr>
<tr><td>Studenterlunden Clay, Oslo</td><td>remoulded</td><td>20</td><td>38</td><td>0.5</td><td>6</td></tr>
<tr><td></td><td></td><td></td><td></td><td>2</td><td>5</td></tr>
<tr><td>Toulnustuc Estuarine Clay</td><td>undisturbed</td><td>4.2</td><td>24</td><td>5</td><td>>77</td></tr>
<tr><td></td><td></td><td></td><td></td><td>80</td><td>0</td></tr>
<tr><th colspan="6">Cohesive soils, undrained conditions</th></tr>
<tr><td>Blue London Clay</td><td>undisturbed (0.7")</td><td>45</td><td>56</td><td>28</td><td>55</td></tr>
<tr><td></td><td>undisturbed (1.5")</td><td></td><td></td><td></td><td>16</td></tr>
<tr><td></td><td>undisturbed (12")</td><td></td><td></td><td></td><td>86</td></tr>
<tr><td></td><td>remoulded</td><td>55</td><td></td><td>35.5</td><td>43</td></tr>
<tr><td></td><td>remoulded</td><td>39</td><td>55</td><td>31.7</td><td>70</td></tr>
<tr><td></td><td></td><td></td><td></td><td>31.5</td><td>67</td></tr>
<tr><td>Boulder Clay (Tynemouth)</td><td>undisturbed</td><td>21</td><td></td><td>14</td><td>0</td></tr>
<tr><td>Norwegian Quick Clay</td><td>in situ</td><td>3</td><td></td><td>40</td><td>97</td></tr>
<tr><td></td><td>vane</td><td></td><td></td><td></td><td>99</td></tr>
</table>

Table 5.2: Values of Brittleness Index for cohesive soils after Bishop (1973). (PI = plastic index, CF = clay fraction)

It is important to emphasize that brittleness varies with mean normal stress and therefore should be regarded as a parameter of soil behaviour rather than a soil property *sensu stricto*. This point is important when dealing with slopes, for example, because the normal load in the centre of the profile is higher than at either end. Consequently the brittleness index will be higher at the edges of a slip surface, where as previously mentioned, the stress ratio is at its highest.

A second essential point is that the measured brittleness of a soil is strongly affected by the method of measurement, and by such factors as sample size and sample end restraints which are known to influence the measured strength of a specimen. There are problems associated with developing parameters using stress-strain graphs as their shape is affected by a number of factors including the type of test, size and shape of the sample, amount of volume change during shearing and whether a single shear surface or a more complex zone develops. The stiffness of the testing system and the need for strain controlled testing to observe post peak behaviour has already been mentioned. Lo (1972) observed that post peak drops in strength on a number of samples were proportional to the recorded stiffness of the test system used. Stress path shape can also have a strong influence. In an investigation of the effect of stress path on drained brittleness, Law (1981) found that if confining stress was kept constant then the highest brittleness values were measured in a conventional drained triaxial test followed in decreasing order by tests with constant p', constant s' and constant σ'_1, respectively. He also showed that Brittleness Index was proportional to cohesion and inversely proportional to the confining stress.

In determining brittle or strain softening parameters for a soil it is therefore necessary to establish both peak and residual shear strength envelopes, thus avoiding dependency on the shape of the stress path. It is difficult, however, to measure undrained residual strength. The post peak behaviour of the soil must be observed – although strain controlled testing is necessary for this. Such behaviour is difficult to assess in terms of strain since conditions within samples become non-uniform as shear zones begin to form.

Little work may be found directly relating to Brittleness Index in the literature.

In order to obtain data on the brittleness of a number of materials, in an attempt to identify any further trends, two approaches were employed. Firstly MSc dissertations completed by students in Engineering Geology at the University of Durham were examined and all appropriate data relating to shear strength testing on mudrocks extracted. Data chosen was from shear box testing (see Chapter 6 for reasons) and in each case peak and residual strengths at various normal loads recorded. From this information, Brittleness Index was calculated according to Equation 5-2. All results are shown in Table 5-2 along with other data where available. Unfortunately many of the students were interested only in the peak shear strength data and consequently the amount of information available for this study is less than hoped for if the total amount of shear box testing is taken into consideration. Furthermore, data are heavily biased by the large amount of research on Coal Measures material undertaken by this Department. Nevertheless, several interesting points are apparent. For each set of tests the calculated Brittleness Index varies according to the normal load – in most cases there is an inverse relation although in some cases scatter appears random. An inverse relationship showed more conclusively in data collected by Bishop, Table 5-1. The highest measured values of Brittleness Index are from the Kimmeridge Clay and the Ampthill Clay and is probably a reflection of low residual shear strength values compared to the rest of the data. Peak values are fairly average. Lowest values are seen in a Quaternary laminated clay from County Durham which has low values of both peak and residual shear strength. These examples illustrate firstly that there is no influence of geological age on brittleness and secondly that brittleness is only related to magnitude of peak and residual strength inasmuch as the difference between the two values is important. A high peak strength does not imply a high brittleness for example.

material	state	PI (%)	CF (%)	$\sigma_n (kNm^{-2})$	I_B	Reference
Upper Devonian Slate	powder	7	80	50	3	Peacock (1983)
				116	7	
				224	15	
Namurian Shale	rem			144	17	Middleton (1985)
				207	30	
				215	9	
	rem (L)	37		300	64	Fytis (1986)
	rem (S)			300	45	
Coal Measures	rem			211	17	Middleton (1985)
	rem (L)	12		300	48	Fytis (1985)
	rem (S)	12		300	26	
(Lumley)				69	30	Starr (1970)
				138	19	
				207	26	
				276	44	
(Langley Park)				69	28	
				138	43	
				207	27	
				276	30	
(Hett - shear plane parallel to bedding)				69	47	
				138	29	
				207	26	
				276	29	
(Hett - shear plane at 90° to bedding)				69	42	
				138	30	
				207	34	
				276	39	

rem = remoulded sample, (L) = large shear box, (S) = small shear box

material	state	PI (%)	CF (%)	$\sigma_n (kNm^{-2})$	I_B	Reference
(Hirwaun - Bassett roof rocks)	1200-600			7	43	Green (1970)
				105	38	
				140	44	
				175	60	
				210	57	
	600-300			69	4	
				105	52	
				138	62	
				140	56	
				175	62	
				207	62	
				210	55	
	300-76			69	26	
				105	13	
				138	13	
(Hirwaun - seat earth)	600 -1200			105	51	
				140	27	
				175	61	
				210	20	
(Bersham Colliery)				300	17	Ratsey (1974)
(Littleton Colliery)	rem		5-10	90	61	McWilliam (1975)
				150	40	
				230	43	
				90	59	
				150	54	
				230	47	

material	state	PI (%)	CF (%)	$\sigma_n(kNm^{-2})$	I_B	Reference
Kimmeridge Clay	SSB rem			210	67	Middleton (1985)
(fresh)	SSB rem			300	50	Fytis (1986)
	LSB rem			300	32	Fytis (1986)
Kimmeridge Clay	SSB rem			210	44	Middleton (1985)
(weathered)	SSB rem			300	54	Fytis (1986)
	LSB rem			300	26	Fytis (1986)
	SSB			107	29	Little (1972)
Ampthill Clay	und 3m	41		69	58	Harris (1971)
				177	52	
				263	50	
	und 6m			138	80	
				207	63	
	und 12m			69	76	
				138	67	
				207	54	
	und 15m			69	75	
				91	42	
				177	66	
				263	53	
Boulder Clay		13-26	25	100	19	Rozier (1977)
				200	12	
				300	8	
Glacial Clay	und	22-34	15-24	50	39	Lynn (1973)
(Carrville, Durham)				121	39	
				200	44	
				300	47	
Quaternary laminated				100	67	Clarkson (1974)
clay				200	59	
(Browney, Co. Durham)				294	45	

The data from Coal Measures mudrocks and shales is often scattered but on observation of original peak and residual shear strength data we find that this often does not fall on well defined failure envelopes.

There are many problems with attempting to use such data. Information was obtained from many sources hence there is much potential for operator error (backed up by poor correlation of points on failure envelopes) and in each case there is no guarantee that the test was taken to a true residual value. Quoted displacements and/or number of runs taken to reach residual value are often low (see Chapter 6).

In an attempt to eliminate some of the scatter a second approach was adopted. Equations of peak and residual shear strength envelopes for a series of mudrocks were taken, and peak and residual shear strength values calculated for defined normal loads. Brittleness Indices were then calculated for each normal load and plotted. The resulting graphs are shown in Figure 5-2 and 5-3. Data were taken from surveys of engineering properties in the literature for Tertiary and Mesozoic overconsolidated clays and mudrocks by Cripps and Taylor (1985 and 1987). Again, the data shows a large scatter some of which is undoubtedly related to the large variation in sample type, test method and operator. However, the following trends may be recognised:

1. Maximum unweathered values tend to be higher than weathered values - i.e.: greater difference between peak and residual values in unweathered mudrocks than in weathered derivatives.

2. High peak values of shear strength, cohesion and friction angle do not dictate a high or low brittleness.

3. Age is not a controlling factor.

4. In all cases brittleness index – normal load is a curved function.

5. In all cases brittleness index decreases with increasing normal load.

Of the MSc theses studied, one (Fytis, 1986) was specifically dedicated to the study of brittleness, and a second (Middleton, 1985) referred to rate (*that is,* the rate at which the strength decreases with increasing displacement) and index of brittleness. Table 5-4 lists Brittleness Indices measured by Fytis on four material types tested in both small and large shear boxes at a normal load of $300kNm^{-2}$

Material	Brittleness Index (%) ($\sigma_n = 300kNm^2$)	
	small shear box	large shear box
Heavily remoulded Namurian Shale	44.8	64.4
Unweathered Coal Measures	48.0	25.6
Unweathered Kimmeridge Clay	50.4	32.3
Weathered Kimmeridge Clay	53.4	25.8

Table 5.4: Values of Brittleness Index (after Fytis, 1986)

The most obvious feature of these values is the discrepancy in results obtained from the large and small shear boxes. For the sample of Coal Measures Shale, and for both samples of Kimmeridge Clay, the small shear box gave significantly higher indices than did the large box although the reverse is true for tests on remoulded Namurian Shale. When the 'rate' of Brittleness *i.e.*: the decrease in shear strength per mm displacement in going from peak to residual shear strength was measured, it was found that the Namurian Shale showed the highest rate, followed by the samples of Kimmeridge Clay, Coal Measures Shale and Weathered Kimmeridge Clay in decreasing order. Again, the age of the material bore no relation to the measured index or rate of brittleness. Other interesting conclusions from this particular study include the following: shear surfaces produced in shear box testing were smoother and similarly residual shear strengths were more easily obtained in weathered clays than in shale fills. As expected, all samples showed an increase in moisture content adjacent to the shear plane.

5.4 The influence of brittleness on mode of failure.

Until now, brittleness has only been considered in terms of the state of the soil at or near limiting equilibrium of the shear surface as a whole. However, it is the behaviour of the slipped mass of the soil after this stage which poses a potential threat to public safety. The magnitude of post failure displacement, that is, the distance travelled by the slipped mass, is dependent on brittleness as well as geometry, local topography and the scale of the problem.

Soils with low brittleness show small post-rupture movements, whereas those with high indices show large movements, and progressive failure is well established by the time these displacements are observed. Flow slides, as opposed to the more usual gravity slides, may be regarded as extreme forms of brittle behaviour where high brittleness values are associated with stress transfer on to pore fluid.

It is important to remember that it is difficult to assess brittleness under field conditions as discussed above. A change in strength due to small displacements near to the point of failure may occur under either drained or undrained conditions not necessarily identical to those which controlled the earlier history of the structure. This presents two problems - not only the choice of whether to employ drained or undrained values, but the fact that undrained residual strength depends on the rate of displacement further complicates the issue, the most critical case being the shearing of loose granular materials where the brittleness is due to the rise in porewater pressure during structural collapse.

5.5 Progressive Failure

As mentioned in Chapter 2 and Section 5.3, one possible need for use of the Brittleness Index is in the computer modelling (finite element analysis) of progressive failure. Before looking at this, it is pertinent to look at what is meant by progressive failure.

5.5.1 Definition and Mechanism of Progressive Failure

One of the first references to progressive failure was by Terzaghi and Peck (1948). Since then many authors have looked at this aspect of soil failure, for example Skempton (1964), Bishop (1967), Bjerrum (1967), Vaughan *et al* (1978).

The mechanism for progressive failure, as discussed in such papers, can be summarised as follows. The process begins when the strength of the soil body is locally exceeded; the means by which this occurs is not relevant to the next stage which depends solely on the behaviour of the soil itself around that point. If the soil exhibits non-brittle characteristics then the failure propagates slowly as mobilized strengths in the adjacent regions reach the available strength of the soil. At general failure (state of equilibrium is reached) peak strength is reached along the whole of the failure surface. Such cases are typically characterised by large pre-failure but small post-failure displacements.

The alternative to this is the scenario in which the soil behaves in a brittle manner from the same starting point, that is, locally exceeding the material's yield strength, the soil at this point fails. The resulting state of non-equilibrium results in the stressing of adjacent soil which in turn fails and propagates the instability to the soil around it. Thus a chain reaction is initiated. The process is generally slow, with small displacements up to the point of instability or general failure. At this point parts of the shear plane will be substantially reduced in strength where failure has occurred, whereas in pre-peak regions, soil may not have mobilized strength up to peak value. Post-failure movements in such cases are typically large compared to those observed in non-brittle cases.

Thus for progressive failure to occur, it is essential that two conditions are met. First the soil body must behave in a brittle manner, and second, stress distribution along potential slip surfaces in the soil body should be non-uniform. Such conditions occur in both natural and man-made slopes, and so progressive failure is a common phenomenon.

5.5.2 Factors Affecting Progressive Failure

Dounias (1987) examines a number of parameters which may influence progressive failure. There are many potential sources of strain non-uniformity found in natural slopes. Because of their nature, they are not homogeneous and isotropic as is often assumed in engineering anaysis. In addition, many potential stress concentrations exist in man-made structures. These include interbedding of different layers, rigid boundaries, weak spots caused by softening due to weathering, high porewater pressures, geometric corners and irregular dam core shapes. Bjerrum (1967) claims that release of strain energy due to the weathering of overconsolidated clays and clay-shales can be a driving force for progressive failure in itself. This slow release of strain energy is also mentioned by Bishop (1967) who also includes factors such as porewater pressure re-adjustments following changes in loading of slope or in groundwater level. To be of use in design and analysis, a method is needed to quantify the effects of such parameters.

A further problem concerns the effects of scale; relative displacements are proportional to the size of the engineering structure, and so large structures are more susceptible to the effects of progressive failure and the use of models is questionable.

Using linear elastic finite element analysis Lo and Lee (1973) calculated Residual Factors (defined by Skempton, 1964) and calculated that on a drained slope the effects of progressive failure increase with height and inclination of the slope. Vaughan (1986 – quoted in Dounias, 1987) disagreed with these findings on the basis that Residual Factor is defined at a factor of safety of 1 and that Lo and Lee's analysis was not carried out at failure. Vaughan used a simplified wedge analysis at limiting equilibrium conditions to show that progressive failure effects increase with the height of a drained slope but decrease as the slope's inclination increases.

The rate of strain may be an influencing factor. It is known that loading rate affects the magnitude of peak and residual strength in laboratory testing, and it may therefore be assumed such conditions apply in the field. The magnitude of post-failure movement is also likely to be affected by strain rate.

5.5.3 Methods of Stability Analysis

Traditional stablility analyses employ limit equilibrium methods to derive factors of safety for stability. Such methods are comparatively quick and simple to use, and can be shown to be reasonably accurate for non-brittle materials. However, assumptions made in the analysis, such as the soil being a rigid material and deforming but not changing in strength after failure, are not valid. Furthermore, limit equilibrium analysis assumes that at the point of failure (state of limiting equilibrium) all shear stresses along the failure surface are exactly equal to the maximum available shear strength of the soil. This is in direct contradiction to the mechanism of progressive failure, in which stresses and strains are assumed to be non-uniform. A further disadvantage of such methods is that they yield no information cconcerning stress distribution or displacements. A separate analysis based on elastic methods and consolidation theory must be employed.

More recently, continuum analysis methods, in which stress distributions are calculated and then checked to determine whether strength criteria have been violated, have been used. These are useful in that they are simple to adopt, give information on stability, stress distribution and displacements, and have been shown to be reasonably accurate, albeit somewhat conservative. The simplest form is when linear elastic conditions are assumed in the calculations. However this is grossly oversimplified since soil behaviour is neither linear nor elastic. Non-linear methods, of which there are many forms in the literature, involve plasticity, so although they are more accurate they still cannot predict all aspects of soil behaviour.

Current practice in the design of earth fill dams employs a combination of traditional stability and non-linear stress analysis.

Various methods have been used to analyse progressive failure. It is possible to incorporate the effects of brittle behaviour into traditional methods of stability analysis; for example by reducing cohesion to zero and using critical state rather than peak strength (Schofield and Wroth, 1968).

Several examples of methods which postulate the method of failure and then model soil parameters accordingly can be found in the literature. Bjerrum (1967) proposed one of the earliest models in which peak strength was exceeded locally and residual conditions were reached. Although elastic energy was considered, strain was not modelled directly. Christian and Whitman (1969) modelled soil and represented stress-strain behaviour, and later, the Palmer and Rice (1973) model illustrated the importance of elastic energy release by using fracture mechanics and shear band propagation.

The use of finite element analysis, although recognised in the late 1960's (Peck 1967, Bishop 1967), has recently come to the fore since computational methods advanced sufficiently to surmount numerical difficulties.

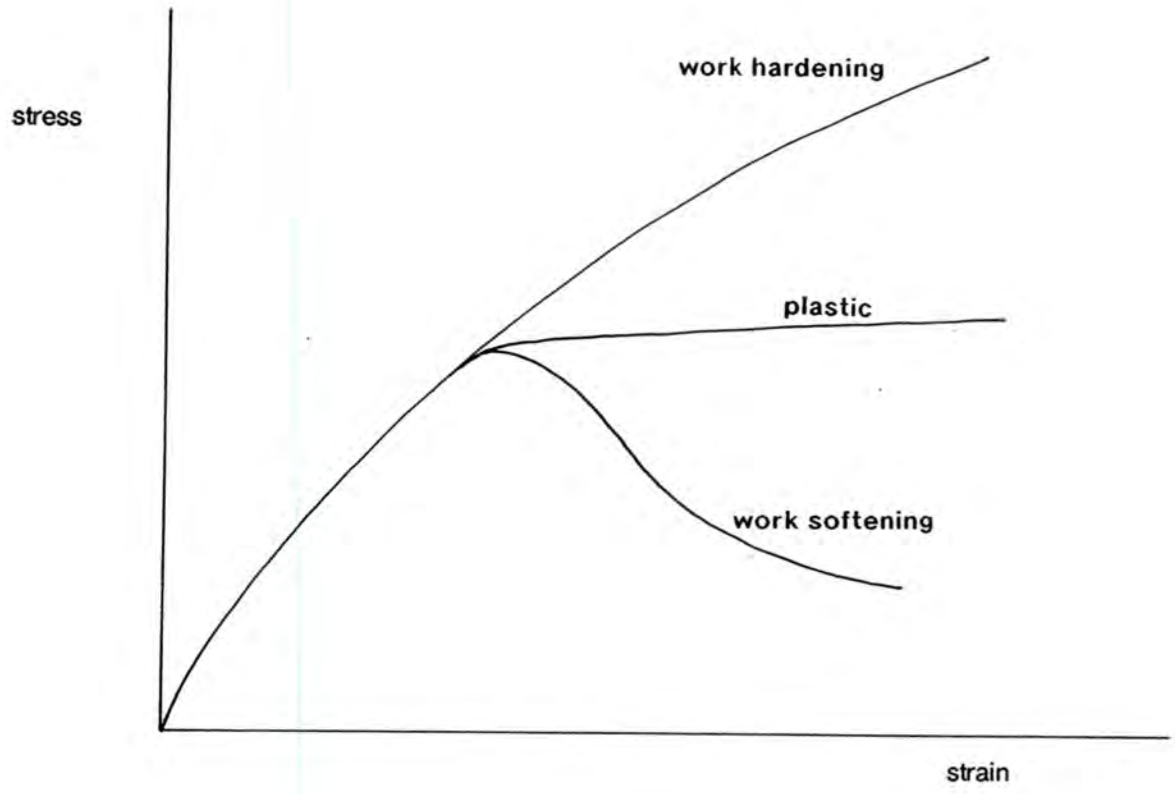

Figure 5.1 Summary of post failure behaviour observed in different materials.

not to scale

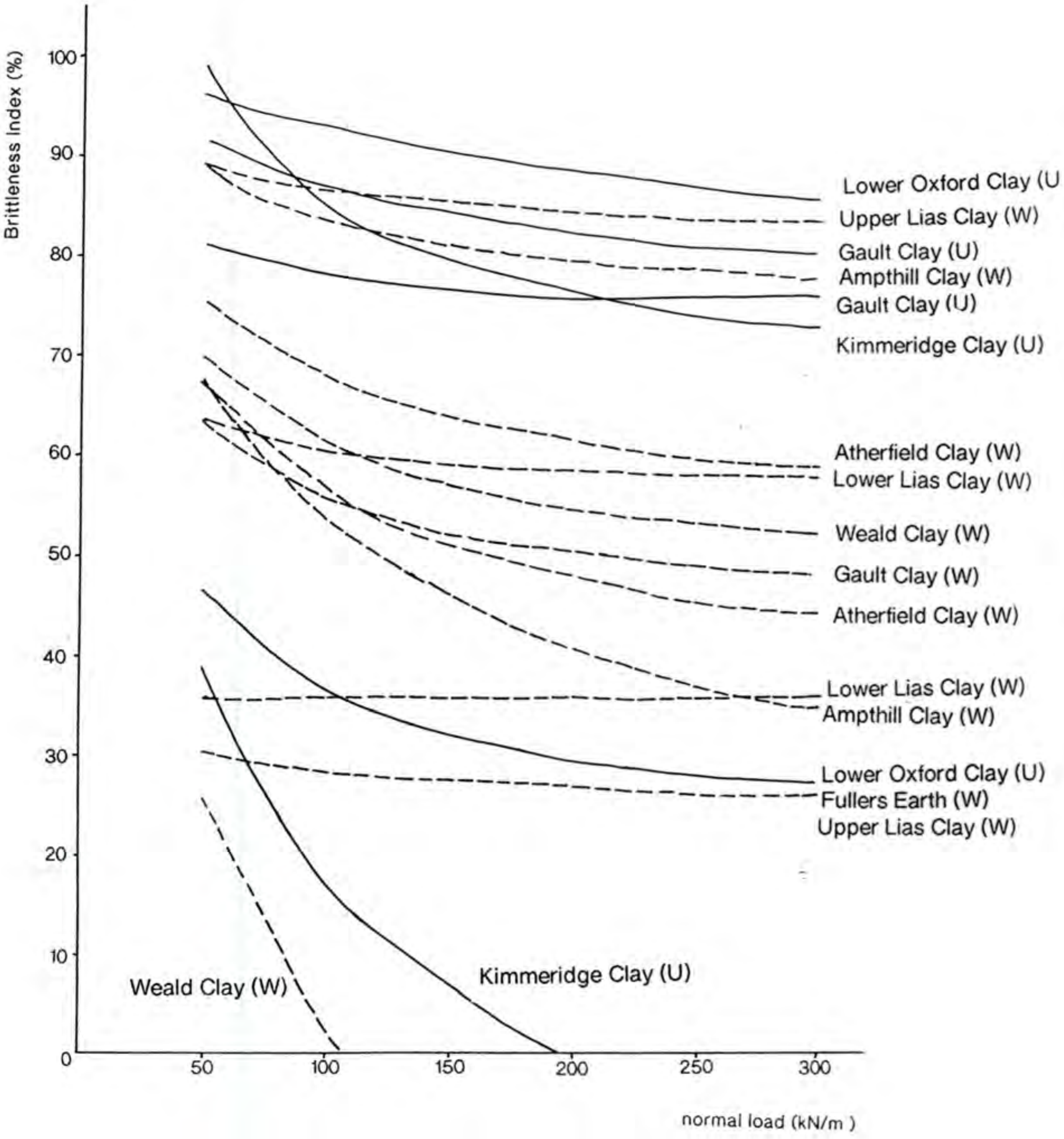

Figure 5.2 Brittleness index – normal load in Mesozoic overconsolidated clays and mudrocks.

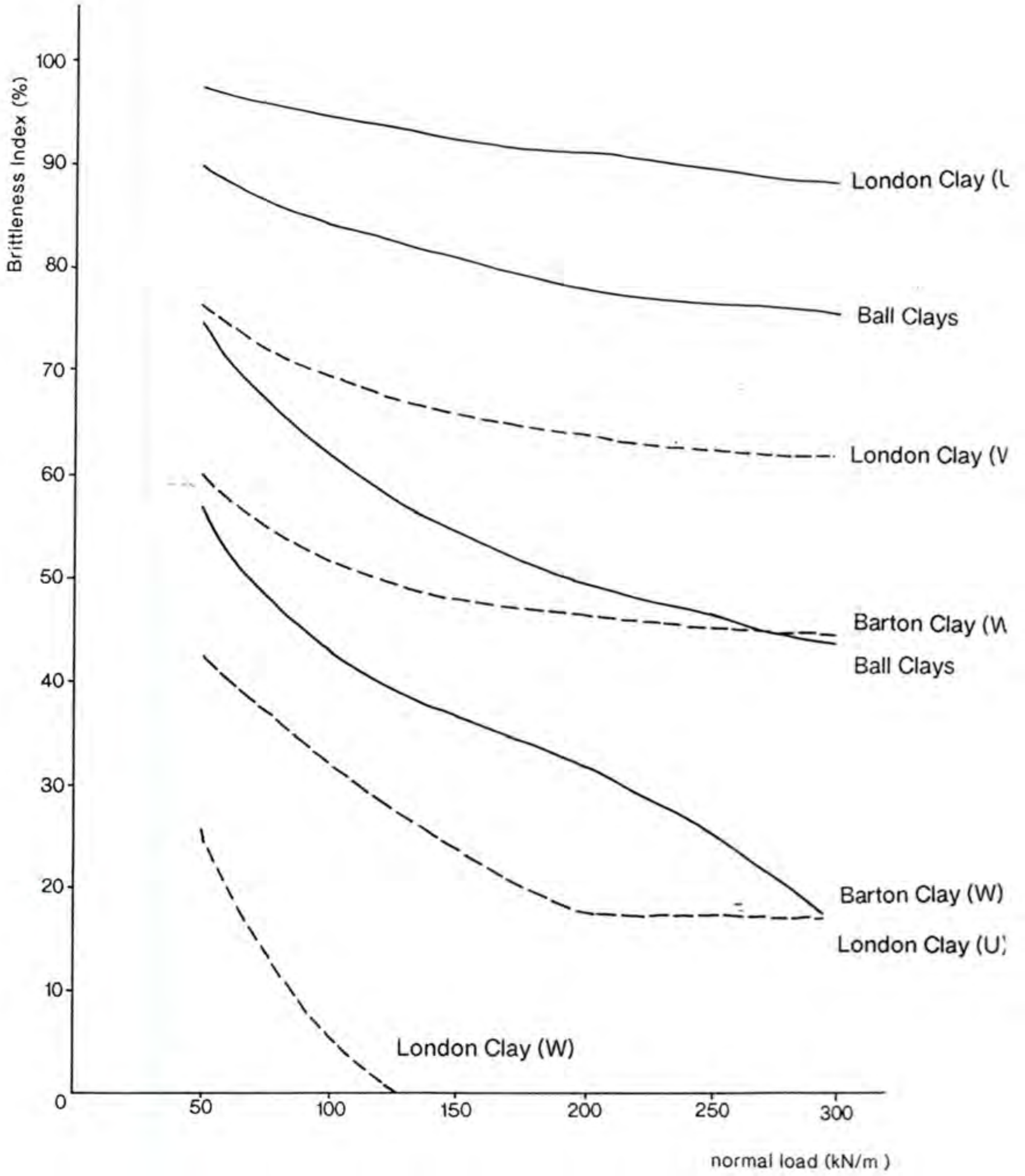

Figure 5.3 Brittleness index – normal load in Tertiary overconsolidated clays and mudrocks.

Chapter 6: DIRECT SHEAR BOX TESTING

6.1. Introduction

Direct shear boxes are the earliest and simplest form of shear strength testing apparatus used for soils, and despite their many shortcomings are frequently employed in commercial testing.
This Chapter describes the use of both small ('standard') 60 × $60mm$ size and large 300 × $300mm$ shear boxes in the current research and presents some of the problems encountered during testing. Test results are presented in Section 6.4 and will be further discussed in Chapter 7.

6.1.1. The Principle of Direct Shear Testing

The principle of direct shear box testing is based on coefficient of friction tests in which the resistance to sliding between two surfaces is measured. The first shear boxes were designed to measure the internal friction angle of recompacted sands and, despite modifications and the development of ring shear techniques, the design of the apparatus has changed little since the early twentieth century. The basic design is shown in Figure 6.1. It will be explained later why the shear box is not appropriate for studying stress–strain relations, but is probably the most convenient means of measuring the average shear strength of soils.

The soil sample is placed in a horizontally split box in which the top half is free to move along the horizontal shear thrust axis of the apparatus. The shear force is applied to the lower half of the box and the resistance to shear movement transmitted through the upper half is measured by a proving ring or load cell.

The method of testing is given by Ackroyd (1962) and Head (1982) based on the ASTM (1979).

It is important to note that shear strength is related to prevailing ground conditions, and therefore the measured value is strongly dependent on the test type and conditions. Pore water pressure in particular can have a significant effect on measured values. This obviously has some bearing on the method of shear strength testing and it is well known that values measured in different tests themselves differ consistently. This point is further discussed in Section 6.2.

6.1.2. Limitations of Direct Shear Box Testing

Despite the simplicity of direct shear box testing in basic theory, sample preparation and testing procedure, many workers have indicated drawbacks in this type of testing. Some of these disadvantages are listed below, and explained in more detail in the following paragraph.

a. Non-uniform stress-strain distribution within the sample,
b. Problems with drainage from the sample,
c. Problems related to box geometry, and
d. Problems associated with reversing shear direction (*see* section 6.1.3).

The only known stresses acting on the sample are the applied normal and shear stress. Therefore a discrete point can be drawn on a shear stress – normal stress graph, but not a unique Mohr Circle. The principal stresses cannot be determined and have been shown to rotate during testing. The stress distribution throughout the sample is known to be non-uniform – this being largely due to box shape and to the presence of rough loading plates (*see* for example, Atkinson and Bransby, 1978). Since only the boundary displacements of the box can be observed and the exact volume of the shear zone to which deformation is restricted is not known, the strains calculated are average values and do not adequately describe that part of the soil which is undergoing failure. It is thought that stress concentrations build up at the box edges and failure moves into the centre of the box from there (Craig, 1983). However Potts *et al* (1987), using finite element analysis with severely strain softening conditions, have demonstrated surprisingly uniform stresses and strains in the final shear zone and very little progressive failure.

Unlike the triaxial apparatus, the shear box includes no means of either porewater pressure measurement or of drainage control from the box. This problem can be largely overcome by using a slow rate of deformation which allows excess pore pressure to dissipate. Most shear boxes employ thin samples in which this process is obviously more rapid than in thick samples where consolidation will be slower. Hence, the porewater pressure can be assumed to equal zero, the test considered fully drained and the effective stress numerically equivalent to the measured total stress. Some workers have studied the increase of moisture in the area of the shear zone (*see* for example, Skempton, 1964), while others have examined the overall effect of water content on shear strength. Cullen and Donald (1971) noted that the residual friction angle was dependent on the moisture content in a series of tests on an over-consolidated kaolinitic clay. A partially saturated soil may show an anomalous cohesion which can be attributed to suction effects (Head, 1982).

One strong objection to the test apparatus is that of the split nature of the box, which imposes both the location and direction of the shear plane upon the sample. This however can be of use in the testing of pre-cut and naturally sheared or fissured samples when the plane of discontinuity can be orientated parallel to the split in the box. Other geometry-related problems include a limit on maximum displacement determined by the size of the box – this is particularly inconvenient in residual strength determinations which require large displacements. Finally, the area of contact between the two sample halves decreases during the course of a run, so the stress values consequently increase. Some authors argue that since both shear and normal stresses are affected equally, the end result of this is neglible, (for example, a 15% reduction in area will result in only a 5% increase in shear strength, (Petley, 1989 *personal communication.*), although it is simple to incorporate area corrections into the calculations. Figure 6.2 shows raw and treated data for comparison.

6.1.3 Use of the Shear Box to measure Residual Strength

One of the advantages of the direct shear box is its applicability to both drained and multi-reversal testing, hence its popularity in testing for residual strength of over-consolidated clays. By reversing the box halves back to their original position (either by hand or mechanically) and re-shearing several times, a cumulative displacement is built up and, after relatively large strains, a constant, steady state value of shear strength is reached. Modifications to the basic equipment, such as those described by Marsh (1972), are required. These include microswitches, locating pieces on the box, and automatic data recording. There are problems with this technique. Most obviously, the reversal technique cannot replicate large shear movement along one constant direction. A certain amount of material is inevitably lost from the sample, and, on re-shearing a small 'peak' at the beginning of subsequent runs is commonly observed. This is attributed to local reorientation of clay minerals (Ratsey, 1974) or to relocation of the shear plane (Cullen and Donald, 1971). It is important to ensure that a sufficient number of runs have been used and that the sample has reached a true steady state. Recent work by Chandler and Hardie (1989) employs the use of thin samples. Using a minimum thickness of $2mm$ they found that the residual strength was reached more rapidly than in conventional $25mm$ thick samples. The authors attribute this to the quicker concentration of shear stress on to the shear surface, although it is possible that there is a danger of the influence of physical boundaries on the shear plane.

'Cut plane' tests can also be employed to find residual shear strengths – here the sample is sliced through along the joint between the box halves in order to reduce the number of runs required. Head (1982) has reported loss of fine material from the shear plane during such tests which has a significant affect on measured shear strength.

6.2 Use of the Shear Box in the Current Research

Of the soils testing apparatus available, the direct shear box was considered to be the most appropriate means of examining the samples for the following reasons:

a. Peak and Residual strengths are obtained from the same specimen.

b. Ease of sample preparation and testing.

c. Repeatability of results.

d. Greater accuracy of results.

From the shear stress–displacement graph produced at the end of a shear box test on a single sample it is possible to obtain both the peak and residual shear strengths. Thus the brittle behaviour of the sample as an expression of these two relative strengths can be observed directly. Neither triaxial nor ring shear apparatus are capable of presenting this overall picture because, although separate peak and residual strengths are obtained, the transition between the two cannot be observed. For comparative purposes it is essential to maintain constant conditions throughout the test programme, especially with a material as inhomogeneous as a rock fill. By using the same sample, possible differences arising from the variation in clay fraction and in composition are reduced and thus the measured brittleness can be considered representative of the sample and not due to differences between samples.

There is some debate in the literature as to which type of test most closely resembles the natural strength of material *in situ*. It is generally considered that triaxial tests tend to give slightly higher values of residual shear strength, while ring shear boxes yield lower values than those obtained from direct reversing shear boxes (Bromhead, 1979). Hawkins and Privett (1985) attribute this discrepancy to the relative ease with which residual conditions are reached. Lupini *et al.* (1981) claim that the differences in strength are apparent only. Both Chandler *et al.* (1973) and Taylor and Spears (1985), amongst others, agree that ring shear tests tend to underestimate and reversing direct shear boxes overestimate the *in situ* residual shear strength. Bishop (1971) quotes differences of $2 - 6°$ in London Clay. Whilst Townsend and Gilbert (1973) accept this conclusion for hard, overconsolidated shales they, with Bromhead and Curtis (1983), show evidence that the two types of box yield similar results. Taylor and Garrard (1984) in the analysis of data from the site investigations of colliery discards found that triaxial tests on samples of 100mm diameter gave higher peak shear strengths than *in situ* and laboratory shear box tests. It follows therefore that a calculation of Brittleness Index using values derived from triaxial and ring shear tests will tend to give an exaggerated value. Consequently it was decided to employ direct shear boxes throughout the test schedule.

Because a wide variety of clast sizes is found in a typical rock fill, both large and small boxes are appropriate. The maximum appropriate particle size for an apparatus is equal to one sixteenth of the equivalent spherical diameter of the test specimen (Taylor and Spears, 1985). The large ($300 \times 300mm$) box is thus suitable for particles up to $37.5mm$ – this is illustrated in Plate 6.1 – (compared to $2mm$ in the $60 \times 60mm$ 'standard' small box) and is commonly used to test fills, road construction materials, colliery spoils as well as industrial slags and rubble. The large box also allows a longer uninterrupted distance of travel which it was thought may contribute to better development of the shear plane. The affect of box size on the results is discussed later in this Chapter.

Due to the length of time (typically three days to peak and two to three months to residual) for a single shear box to test a sample to residual state only a limited number of samples could be tested in the large box. Accordingly tests were also conducted using the small boxes.

6.3 Practical Method

6.3.1 Sample Sites

The locations from which samples were collected have been identified and described in Chapter 3. At each site two types of sample, undisturbed and bulk disturbed, were taken from the pits for shear strength testing, in addition to those collected for geological and chemical analysis. All samples were sealed in polythene cling film in the field, then further wrapped and waxed in the laboratory (after taking moisture contents) to ensure air tightness during storage. In all cases testing was carried out as soon as possible after collection.

6.3.2 Sample Collection

Ideally, undisturbed samples taken by pressing a cutting shoe into the fill should be used throughout the programme in order to test for the effects of compaction within the structure. Although such sampling was possible in a few cases, the majority of tests had to be undertaken on remoulded samples due to the following problems encountered whilst attempting to take undisturbed samples:

a. unsuitability of large clast size for apparatus,
b. difficulty in insertion of cutting shoe by hand,
c. collapse of poorly compacted samples,
d. potential damage to Water Board property.

By the nature of undisturbed samples, it is not possible to discover the particle size distribution until after the test is complete. Many of the pits which were dug contained clasts in excess of $37.5mm$ equivalent diameter, and consequently bulk samples were collected to be later remoulded in the laboratory. Plate 6-2 shows the clast size range in a typical pit, while Plate 6-3 demonstrates the effect this may have on a large shear box test result.

Large clasts can cause problems in other ways; the cutting shoe must be pushed into the material by hand, which is straight forward when the material is fine grained, but the presence of large clasts hinder the operation since these must be either broken or pushed out of the way by the cutting shoe.

Whilst most samples taken from dams were well compacted, a few were less densely packed and therefore prone to collapse. The spoil heap samples also fall into this category. Once clasts and matrix have fallen from the cutting shoe and been pressed back in, the sample cannot be considered truly undisturbed.

The cutting shoe, plus sample, must be dug from the ground; this involves the removal of a quantity of material from around the pit until the shoe can be levered out. On a 1 *in* 2 or 1 *in* 3 sloped dam face this disturbed area inevitably covers several square metres, and this may cause difficulty because damage to Water Board property had to be kept to a minimum.

6.3.3 Preparation and testing of Samples

The method that was used is essentially that quoted earlier. Preparation depends on the type of sample undergoing testing.

a. Undisturbed

In this case the samples were transferred directly from the cutting shoe to the large shear box either by hand or by using a hydraulic jack. Particle size analysis and Atterberg limits were determined at the end of the test.

b. Bulk Samples

The bulk samples were air dried prior to testing, thus preventing excess breakage by oven drying. Samples were then dry sieved to remove particles of unsuitable particle size and the mass required to give the correct density weighed. The appropriate amount of distilled water was added; then the sample was wrapped in polythene and left to equilibriate for at least 24 hours before being compacted into the shear box by hand. The compaction densities that were used were determined by Proctor compaction tests according to B.S. 1377 (1975 *).

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* work was done prior to publication of the 1990 edition.

Compaction in the large shear box can also be achieved by using a Kango hammer although this did not prove significantly more efficient than completing the job manually. There is also a risk of introducing hard layers into the box and these may act as potential shear surfaces.

Due to the restraints of time and number of samples, most of the large shear box tests were of multi-stage form (method according to Head (1982)) with four normal loads being applied in increasing magnitude in order to obtain peak strengths, and then in decreasing magnitude for residual strength. The box was run at a higher speed between these two stages in order to form the shear plane. A few tests were conducted at single loads for comparative purposes since multi-stage tests are often thought to be unreliable.

6.4 Results

The results of both large and small shear box tests are presented in this section although the accompanying discussion is limited to brief comments. The overall relevance, and significance of the tests with respect to earlier weathering and chemical studies is discussed in Chapter 7. The results are grouped according to material type as previously used in Chapter 4.

For each test, shear stress – displacement curves can be drawn for each increment of normal load. A shear stress – normal stress plot is then drawn for a series of tests to produce the failure envelope. Ideally, a vertical displacement – horizontal displacement curve should be drawn so that any expansion or contraction during shearing can be monitored. Although vertical movements were recorded these showed a decrease in sample height, a feature which is inconsistent with the observed losses in shear strength and increases in moisture content (with corresponding decreases in bulk and dry density). Loss of material from between the two box halves is a common problem in shear box testing (Section 6.1.3) and it is this which is thought to be responsible for obscuring true dilation of the samples, which occurs as critical state is approached. A small amount of contraction due to particle breakage and crushing, and to clay mineral alignment, does take place, but this is thought to be small in magnitude and restricted to the relatively narrow shear zone.

6.4.1 Tests to check apparatus precision

A series of 16 tests was performed, using the small shear boxes, in order to study the limits of precision of the apparatus. In total, four boxes were available for use and it was therefore necessary to confirm their ability to produce consistent results. A sample of fresh Coal Measures shale, crushed to pass a $2mm$ sieve was remoulded at 11.5% water content (determined by Proctor compaction) and then used for tests at normal stresses of approximately $50, 100, 150$ and $200kNm^{-2}$ on each box. Figures 6.3 a-d show the stress - displacement curves. Figures 6.4 a and b show the resulting failure envelopes. The average initial and final water contents are plotted against normal stress in Figure 6.5.

Figures 6.3a to d and 6.4a and b both show a reasonable agreement in peak shear strength between boxes at each normal load. Peak shear strength values ranged from $47 - 50kN/m^2$ at normal load of $50kN/m^2$, $78 - 94kN/m^2$ at $100kN/m^2$, $116 - 120kN/m^2$ at $150kN/m^2$ and $146 - 152kN/m^2$ at $200kN/m^2$. The residual shear strength values however show a greater variation, with values of $35 - 43kN/m^2, 52 - 75kN/m^2, 65 - 88kN/m^2$ and $70 - 105kN/m^2$ over the same normal loads respectively. It is important to note that none of the boxes reads consistently high or low, and any variation appears random. The stiffnesses in reaching peak do however follow the same trend at each normal load with the four boxes reading in the following order: $4 > 2 > 1 > 3$.

Normal Stress	Peak Shear Strength			Residual Shear Strength		
	mean	st.dev	r.d	mean	st.dev	r.d
50	49.4	1.6	3	34.5	3.3	9
100	73.5	4.4	6	54.8	6.3	12
150	100.1	1.7	2	68.6	9.4	14
200	130.1	10.4	8	69.1	6.7	10

relative deviation (r.d.) $= \frac{st.dev}{mean} \times 100\%$

Table 6.1: Summary of results from precision tests on small shear boxes.

The poorest agreement is seen at normal loads of $200kNm^{-2}$, but this is largely due to the use of inappropriate proving rings. For routine testing, a stiffer proving ring was mounted on one machine which was then used for all high loading tests.

Table 6.1 shows the means and standard deviations of peak and residual shear strengths, and also the relative deviations in the observed values. It has already been noted that there is more variation in the data for residual shear strengths. It is debateable whether the samples have reached a true residual value after only $120 - 160mm$ displacement, since typical displacements of $100 - 500mm$ are required for obtaining residual strength in over-consolidated clays (Skempton, 1985). This may be compared to distances of $0.5 - 3mm$ and $3 - 6mm$ to reach peak strength for overconsolidated and normally consolidated clays, respectively. Skempton (1985) states that the change in water content starts at a typical displacement of $5 - 10mm$. The values that are obtained may in fact be the constant volume (critical state) value (Atkinson 1989, *personal communication*). However, as shown in Plate 6.4, in all cases the samples split easily along planes to reveal shiny striated surfaces. It is therefore assumed that the samples are nearing their residual strengths. This point is further discussed in Section 6.4.2 in connection with the large shear boxes. It is important, too, to note that these samples are remoulded and so will yield lower peak strengths and will reach the residual state more rapidly due to the more disordered structure of the soil.

Other checks made on the apparatus to ensure precision and accuracy are listed below.

a. Regular calibration of linear variable differential transducers (LVDT) and proving rings.
b. Fluctuation of the above with time/temperature.
c. Stiffness of the shear box.

6.4.2 Tests on 'standard materials' – Fresh Coal Measures Shale and fresh Kimmeridge Clay

The purpose of this set of tests was to use standard materials to identify differences between the large and small shear boxes, to establish whether multi-stage testing would be applicable and for observation of the effect of particle size in the large shear box. Data was also provided for the comparative study of a typical 'brittle' and 'less brittle' shale, Coal Measures shale and Kimmeridge Clay, respectively.

Fresh samples of the two materials were collected, then air dried and mechanically crushed and sieved to produce samples with the appropriate particle size ranges.

Figure 6.6 shows the shear strength – normal stress data for tests on the Westphalian Shale, while that for the Kimmeridge clay is in Figure 6.7 and the results are further summarised in Tables presented in each section.

6.4.2 a *Large vs Small box*

small shear box					large shear box				
	normal load					normal load			
shear strength	50	100	150	200	shear strength	70	150	220	300
peak	43	72	94	105	peak				209
residual	34	57	67	78	residual				82

Table 6.2 : Results of direct shear box tests: Coal Measures shale.

small shear box					large shear box				
	normal load					normal load			
shear strength	50	100	150	200	shear strength	75	150	320	
peak	84.7	104.9	125.1	145.3	peak	58	115	205	
residual	22.8	31.6	40.5	49.3	residual			145	

Table 6.3 : Results of direct shear box tests: Kimmeridge Clay.

Figure 6.7 clearly shows that peak and residual shear strengths obtained from the large shear box are slightly higher and lower, respectively, than those from the small shear box when a less than $2mm$ fraction is tested in each box.

The shear strength envelopes shown in Figures 6.6 and 6.7 appear strongly curved and pass through the origin. Several authors have noted that this is generally the case for rock fill (Marsland, 1973). Hawkins and Privett (1985) report the same effect in cohesive soils. Taylor and Garrard (1984) noted that whilst individual tests on coarse colliery discards yielded curvilinear shear strength envelopes, the composite data from over a thousand tests showed little curvature. They employed a statistically tested power fit of the general form $\tau = m(\sigma')^z$ (or $log_{10}\tau = zlog_{10}\sigma' + log_{10}m$) where m and z are constants to describe the curved envelopes. The cause of the curvature is thought to be due to attrition between particles - for example the abrasion of corners. This will be common in a shale fill where particles are tabular and angular. The effect increases with increasing normal pressure. Figure 6.8 shows the results of particle size analysis on a shale fill pre and post shear box testing and demonstrates the breakdown observed. It is of interest to note that Taylor and Garrard (op cit) found that this effect was more pronounced in the results of shear box tests than in triaxial tests.

6.4.2. b *The Effect of Particle Size*

passing 2mm					passing 20mm				
	normal load					normal load			
shear strength	CM	300	KC	300	shear strength	CM	300	KC	300
peak		209		200	peak		217		250
residual		82		145	residual		119		

Table 6.4 : Results of direct shear box tests: Coal Measures shale and Kimmeridge Clay – samples of different maximum clast size.

In both Kimmeridge Clay and Westphalian Shale the results from large shear box tests show slightly higher peak strengths in the coarser particle size samples. There is a much larger difference in residual strengths – this being a reflection of the larger clay fraction present in the less than $2mm$ fraction. Peak strength is less dominated by this effect since packing, pore water pressure and composition, and rate of deformation as well as particle shape and friction are all controlling factors.

It is interesting to note that the residual strengths of the Westphalian Shales passing $20mm$ and passing $2mm$ lie slightly above, and significantly below, that of the small shear box samples. It would appear that the ratio between the maximum particle size and the shear plane area has an effect on the ultimate strength of the sample.

This ratio appears to be important in determining the morphology of the shear plane produced. This is demonstrated in Plates 6.5 to 6.8 which show the effect of increasing maximum particle size on the type of shear plane produced. In the less than $2mm$ size sample the shear plane is flat and smooth, whereas in the passing $20mm$ sample it is ridged and hummocky. Figure 6.9 is a comparison of vertical profiles drawn across the shear surface of tests on Kimmeridge Clay using these two size fractions. The greater surface area of the rougher surface may also have some bearing on the measured shear strength. Plate 6.9 shows an undisturbed sample of Namurian shale (discussed in Section 6.4.4) in which the shear plane has been forced around a large clast. The sample fails through the path of least resistance and thus passes through the the relatively soft clay and silt, rather than through the solid shale clast, despite the fact that the geometry of the box will try to force a plane which should pass straight through the clast. This phenomenon was observed in several of the undisturbed samples tested.

6.4.2 c *Comparison of brittleness*

Tests on the less than $2mm$ and less than $20mm$ size ranges in the large shear boxes under a normal stress of $300kNm^{-2}$ were used to demonstrate some differences between the two standard materials. Even though both samples were crushed and remoulded so that none of the original structure was present and able to influence the peak strength it is clear that there are significant differences in the shear behaviour of the two samples. Figure 6.10 shows the shear stress – displacement curves for two of the four tests; the resulting Brittleness Indices for the samples (calculated from equation 5.2 in Chapter 5) are listed in Table 6.2

material	Brittleness Index	
	size range less than 2 mm	size range less than 20 mm
Kimmeridge Clay	34%	
Westphalian Shale	61%	44%

Table 6.5 Comparison of Brittleness Index in remoulded samples of Kimmeridge Clay and Coal Measures shale.

The peak strengths are within $40kNm^{-2}$ of each other, and show similar initial stiffnesses. A much greater range in shear strength, nearly $100kNm^{-2}$, is observed in the residual strengths. This is reflected in the range of brittleness indices, the latter being directly dependent on the former.

The samples appear to reach their residual strengths over the same displacement, probably due to their remoulded nature.

6.4.2 d *Comparison of data from triaxial apparatus and shear boxes*

Data obtained from consolidated drained triaxial tests on remoulded samples of Coal Measures shale and Kimmeridge Clay have also been plotted on Figures 6.6 and 6.7 for comparitive purposes. Peak shear strength envelopes for both materials appear linear, with measurable cohesion intercepts, unlike the envelopes drawn from data from shear box tests. This is in agreement with the findings of Taylor and Garrard mentioned earlier. It would appear that the forcing of shear plane formation and the imposed movement during the shear box test causes a greater amount of particle breakdown, expressed as greater curvature of the shear strength envelope, than that in the triaxial apparatus during testing. It should be borne in mind however that observation of naturally occurring failures reveals attrition of particles in the shear zone.

The triaxial data appear to be comparable to that from the shear box tests with shear strength parameters, determined graphically, as follows: Coal Measures shale – $c'_p = 17kN/m^2, \phi'_p = 30°$, and Kimmeridge Clay – $c'_p = 25kN/m^2, \phi'_p = 32.5°$.

6.4.3 Gale Common – Weathered Westphalian shale

The location of Gale Common and the sample lithologies, mineralogies and general engineering properties have been described in Chapter 3.

Figures 6.11 to 6.14 show shear stress – normal stress relations for small and large shear box tests on less than 20*mm* size fraction, from fresh, stockpiled, 5 and 15 year old samples.

small shear box					large shear box				
	normal load					normal load			
shear strength	58.4	109.2	156.5	200.1	shear strength	70.2	116.2	208.3	300.5
peak	49.6	93.5	126.0		peak	62.5	105.7	168.7	
residual	35.6	65.5	75.1		residual	32.6	45.6	75.0	120.1
brittleness index	28	29	67		brittleness index	48	57	56	-

Table 6.6 : Results of direct shear box tests: Gale Common fresh shale.

small shear box					large shear box				
	normal load					normal load			
shear strength	58.4	109.2	156.5		shear strength	72.25	200	200	200
peak	48.4	81.8	125.9		peak	50.35			
residual	39.9	74.8	91.5		residual				
brittleness index	18	9	27		brittleness index				

Table 6.7 : Results of direct shear box tests: Gale Common stockpile.

The fresh samples show trends observed in the standard samples – markedly curved envelopes with the large shear box results straddling those of the small box. Only one large shear box test result is available for the stockpiled material but allowing for a margin of error the results from the small shear box are similar to those from the fresh material.

small shear box					large shear box				
	normal load					normal load			
shear strength	58.4	109.2	155.4	204.4	shear strength	75.3	124.7	198.9	296.4
peak	50.0	81.6	113.1	118.0	peak	44.8	-	103.3	135.6
residual	32.6	65.9	-	78.8	residual	-	30.6	47.5	67.8
brittleness index	35	19	-	33	brittleness index	-	-	54	50
undisturbed					peak	112.8(u)	188.9(u)		
sample					residual	99.9			
					brittleness index	11			

Table 6.8 : Results of direct shear box tests: Gale Common five year weathered. ((u) - undisturbed sample)

In the five year old material, the peak strengths measured in the large shear box are approximately equal to those from the small boxes. However there is still a significant difference in the measured residual strengths. Both sets of results are markedly lower than those of the fresh material.

Two undisturbed samples were taken from the five year old part of the embankment. The results are also shown on Figure 6.15. The peak shear strength envelope, although still probably curved, is much higher than in the remoulded samples and yields friction angles about twice as high. The residual shear strength at $70kNm^{-2}$ is extremely high (ϕ_r of 51° compared to 13° in the remoulded sample) this being due to the presence of a large clast across the 'shear plane' which forced failure away from the central part of the box – this effect was discussed in 6.4.2 b.

small shear box					large shear box				
	normal load					normal load			
shear strength	58.4	109.2	158.2	204.4	shear strength	75.3	124.7	296.4	200
peak	48.1	71.4	106.3	125.6	peak	47.54	89.7	125.4	
residual	30.6	61.5	81.9	95.1	residual	-	-	109.0	
brittleness index	37	14	23	24	brittleness index	-	-	13	

Table 6.9 : Results of direct shear box tests: Gale Common fifteen year weathered.

Figure 6.16 demonstrates again the marked curvature of failure envelopes in the 15 year old material. Strengths are lower than in the fresh material although slightly higher than in 5 year old samples. This may be related to the highly weathered appearance of parts of the 5 year old profile, which as discussed in Chapter 3 may possibly be attributed to the presence of a roadway.

6.4.4 East Pennine Dams – Weathered Namurian Shale

Several dams in the Weardale and Teesdale area of the east Pennines, County Durham were described in Section 3.5.5. The results from four sites are presented below, and in Figures 6.15 to 6.18

small shear box					large shear box				
	normal load					normal load			
shear strength	58.4	109.2	156.5		shear strength	200	200	200	200
peak	39.7	78.2	120.7		peak				
residual	29.9	67.0	90.8		residual				
brittleness index	25	14	25		brittleness index				

Table 6.10: Results of direct shear box tests: Grassholme Dam.

small shear box					large shear box				
	normal load					normal load			
shear strength	58.4	109.2	156.5	204.4	shear strength	75.0	124.7	208.0	300.5
peak	40.7	70.8	100.7	113.6	peak	64.4	110.3	153.0	195.8
residual	35.9	62.4	96.7	81.7	residual	30.4	-	86.6	102
brittleness index	12	12	4	28	brittleness index	53	-	43	48

Table 6.11: Results of direct shear box tests: Balderhead Dam.

small shear box					large shear box				
	normal load					normal load			
shear strength	58.4	109.2	158.2	204.4	shear strength	75.3(u)	200	300.5	300.5
peak	34.2	73.9	88.5	116.0	peak	55.9		225.7	224.5
residual	31.6	61.2	65.4	91.2	residual	-		149.6	120.5
brittleness index	8	17	26	21	brittleness index	-		34	46

Table 6.12: Results of direct shear box tests: Grassholme Hury tunnel spoil heaps. ((u) - undisturbed sample)

small shear box					large shear box				
	normal load					normal load			
shear strength	58.4	109.2	156.5	204.4	shear strength	200	200	200	300
peak	40.3	79.2	112.5	122.0	peak				245.9
residual	39.6	70.6	86.1	89.3	residual				165
brittleness index	17	11	23	27	brittleness index				33

Table 6.13: Results of direct shear box tests: Burnhope Dam.

Small shear box tests on samples from Grassholme Dam (Figure 6.14) show a near linear shear stress - normal stress relationship. This may reflect the more clayey nature of the material causing it to be less prone to breakdown and attrition during shearing. A series of tests on samples from $1m$ depth on the Grassholme – Hury tunnel spoil heaps, however, yielded the expected curved failure envelope with noticeably lower values of both peak and residual strength. Samples from Burnhope Dam also gave rise to curved envelopes having shear strength values closer to those from Grassholme. Values from Balderhead dam follow a similar pattern, but are slightly lower.

As in earlier testing, large shear box peak and residual shear strengths fall respectively above and below those from the small shear boxes, although it should be noted that the data set is smaller so it is possible that the trend may be coincidental. Mechanical problems following the intensive use of the large shear boxes prevented the undertaking of more than the one completed multi-stage test in this group, so producing the isolated values for the spoil heap and Burnhope samples.

It was only possible to take one undisturbed sample – from the surface of the more northerly Spoil Heap where a significant clayey matrix provided the necessary cohesion. Below this level, the fill was not sufficiently compacted to allow a sample to be taken. In the dams themselves, the presence of boulders, that is, clasts with a diameter larger than 200mm (B.S. 5930 (1981)), prevented the insertion of the cutting shoe. Peak friction angles from this undisturbed sample and remoulded material from the same source are within a close range of 0.5°. It is also interesting to note that the clast size in the remoulded sample is not excessively large and it would therefore appear that there is little difference between the samples.

6.5 Chemical aspects of shear box testing

6.5.1 The problem

During the testing of five year old samples from Gale Common in the large shear box, growths of a white to buff crystalline substance were noted around the lid of the shear box. These are shown in Plates 6.10 and 6.11. Fragments of the growth were removed, taking care not to damage the crystal structure and after preparation, observed under the Scanning Electron Microscope. Plate 6.12 shows a micrograph of a typical part of the growth. The view comprises a dominant equant, blocky species with subordinate elongate varieties.

A further sample of the growth was subjected to X-Ray Diffraction Analysis. This revealed the composition to be a mixture of halite ($NaCl$) and thenardite (Na_2SO_4) (JCPDS 5-628 and 5-631 respectively.) A second sample from the same box and test also contained small amounts of quartz and clay minerals. Subsequent large shear box tests also yielded such growths, the compositions of which are listed in Table 6.14 below.

Test Number	Mineralogy
GCLSBa4	Thenardite, Halite, Quartz, Gypsum, Clays
GCLSB2 (u)	Thenardite, Halite, Quartz, Clays
GCLSBb4	Quartz, Gypsum, Clays, Gibbsite
GCLSBb4	Thenardite, Halite, Quartz, Gypsum, Clays

Table 6.14 : Minerals identified in crystalline growths formed during large shear box testing.

Thenardite is at first glance a somewhat unusual mineral to find under these circumstances. Normally it is associated with alkaline lakes and playas in arid areas where it is deposited from sodium sulphate rich waters (especially warm brines) for example the western USA, Canada, Siberia, northern Africa and central Asia (Dana, 1951). Other occurences may be as an efflorescence in arid soils, and it has also been associated with other alkali halides and sulphates adjacent to fumaroles and as an encrustation on recent lavas. However, thenardite has been reported from the surface of old coal spoil heaps (Billing, 1987) with halite. Here it was thought that thenardite was able to exist due to the preferential crystallisation of sulphates over halides (Weast and Astle, 1983)

The occurrence of halite is less puzzling. Sodium chloride rich brines are well known to be associated with coal bearing strata and old mine workings. This condition is thought to be worse on the eastern side of Britain.

It was therefore decided to observe the composition of water in the shear box and to look at change in composition with time during testing.

6.5.2 Modifications to Shear Box Apparatus

Samples of water were taken from around the shear box at various stages of testing, for chemical analysis. Before this could be done however, several potential problems had to be dealt with.

Firstly, it had become visually obvious that the shear boxes were rusting badly despite being cleaned and repainted with hamerite paint prior to each test. It was clear that the paint was suffering abrasion from shale clasts even after only two or three runs of the shear box. There was therefore a strong risk of serious contamination of water samples by rust, by flakes of paint and by elements leached from the metal of the box itself. It was decided to construct two new boxes and reservoirs entirely of stainless steel. These were designed to fit on the existing frameworks and thus used the original motors, gearboxes and hydraulic loading system. Similar problems were encountered by Chuay (1986) when testing the effect of various solutions on the consolidation/swelling of clays. Stainless steel equipment had to be manufactured to allow testing in an inert apparatus which would not react with the solutions being tested.

It was important to confirm that the stainless steel itself was not contributing to the ionic composition of the water in the reservoir. Thus a series of tests were performed in which the apparatus was left for several days containing only distilled water. Samples were taken at regular intervals and tested as described below.

Another potential source of contamination was dust in the atmosphere of the laboratory itself. The effect of this was to a large extent prohibited by covering the box and reservoir with polythene cling film. To moniter the amount of dust likely to have settled on the surface of the water in the reservoir, a container was placed next to the shear box, filled with distilled water and covered in the same way. Before the potential problem of dust was realised, several tests were run without cling film covering. In all cases, samples were taken from the 'control' container at the same time as those from the shear box itself.

6.5.3. Method of Testing

Samples were taken by pipette from several positions around the reservoir to ensure an average composition was collected. These were then stored in $125ml$ screwtop plastic sample bottles which were kept in a dark, cool place until testing. Testing was carried out as soon as possible after collection.

An initial batch of samples was run on the Varian AA-575 Atomic Absorption Spectrophotameter at Newcastle University in February 1988. A second set of samples was also run at Newcastle University in May 1988 but after this, further use of the machine was not possible. Rather than use another machine and thereby have to re-run several of the original samples to confirm that the two pieces of equipment were comparable to each other, all the samples were tested using Inductively Coupled Plasma-Atomic Spectroscopy (ICP-AS). The machine used was a Philips PV 8050 Emission Spectrometer with a Philips PV 8490 ICP Source Unit at Royal Holloway and Bedford New College (RHBNC), University of London. This equipment allows much more rapid testing as the sample is scanned for all elements simultaneously, whereas conventional machines have to be set for a particular element. All samples are then analysed for this element before re-setting for the next element.

Raw data is given in intensity from which concentrations were calculated in parts per million, using a programme 'WATERS' specially developed by RHBNC for analysis of cations in water. Intensities are compared to those in a known standard which is scanned at regular intervals during testing (every ten samples) to identify machine drift. After each scan of the standard the machine may be recalibrated, although in this case it was rarely necessary. The conventional atomic absorption spectrameter produces results in a similar way, *that is:* by comparison to standards of known composition. Again, the standards are run at regular intervals and the machine recalibrated to correct for drift. It was found that this was necessary after every five samples for most elements, and every other sample for aluminium and silica. The method of Atomic Absorption Spectroscopy is described in Tucker (1989).

6.5.4. Results

Preliminary results obtained in the manner described above are tabulated and also shown graphically in Appendix V. Although of interest, as they may give information on both the breakdown of shales during shearing under saturated conditions, and also shed light on the relationship between shear strength and ionic composition of porewaters they are not however directly related to the immediate matter in question and therefore have been presented as an Appendix to this thesis.

The time taken for a complete large shear box test to residual strength has already been mentioned, and due to the number of different sample locations the first priority was to test material from each of these. However, the possibilty of related testing was considered. The stainless steel shear box, being inert, would readily lend itself to other chemical testing, for example the effect of changing water composition on shear properties of material in the shear box. The work of Kenney (1968) (see Chapter 7) for example, is well known. The potential problem with undertaking such work on shales is the amount of time taken for change in water composition to permeate throughout samples although recent work by Cripps and Anderson (1990) on Namurian Shale showed that acid solutions were capable of leaching out many elements within days. Similarly, McDermott (1989) was able to demonstrate that nitrate and phosphate rich solutions were able to alter consolidation characteristics of various clays within a few days.

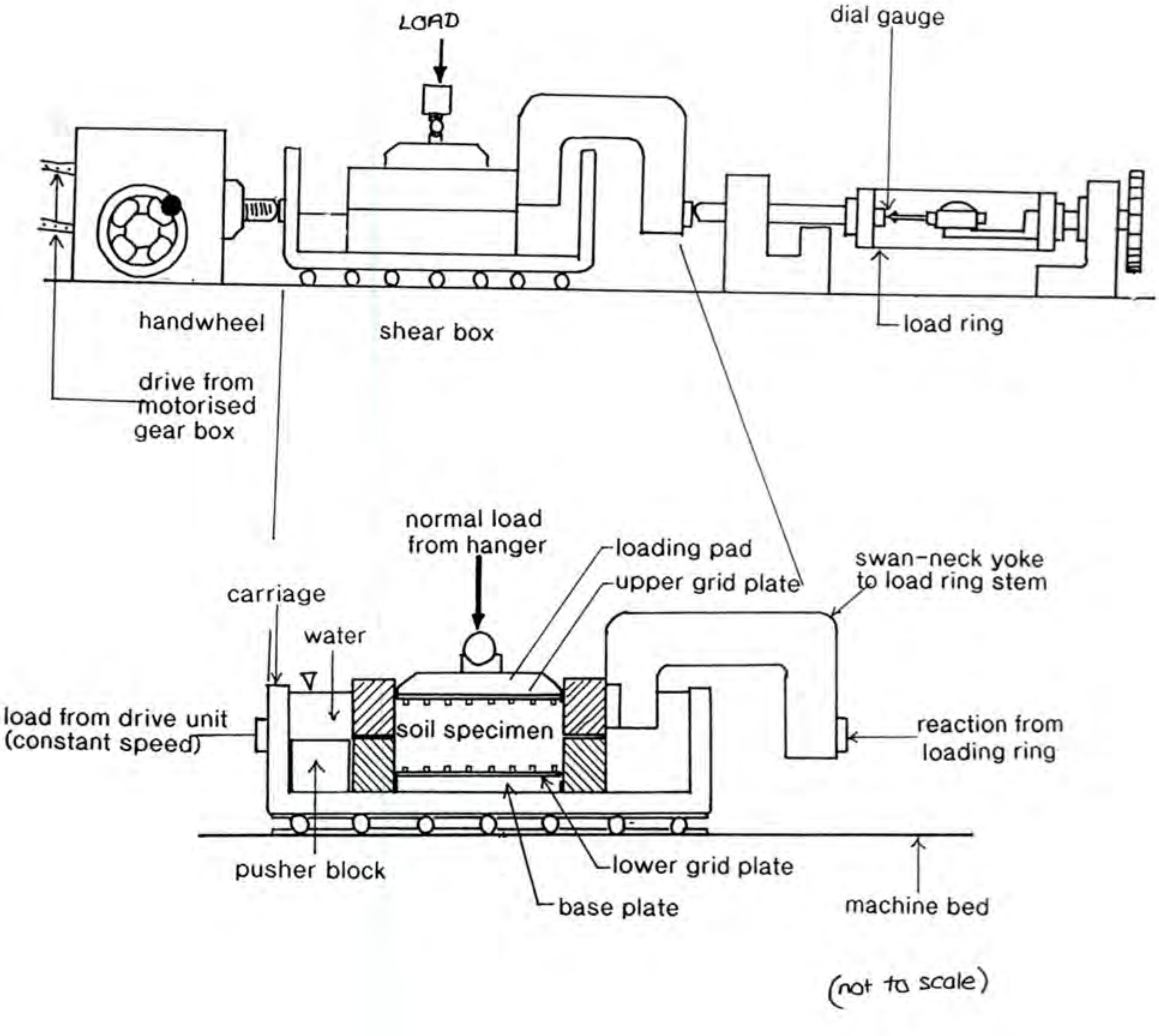


Figure 6.1: Basic design of shear strength apparatus

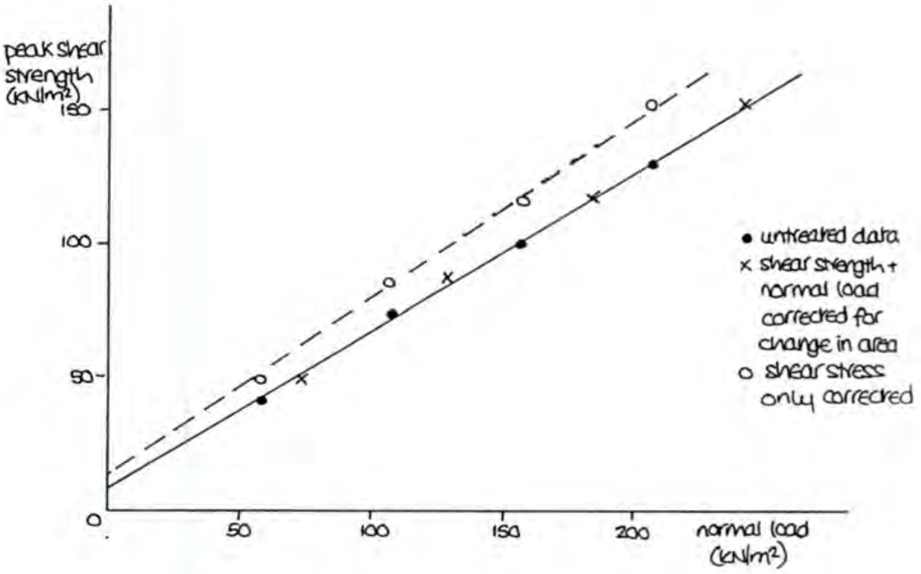


Figure 6.2: Data from shear box test, and same data corrected for change in area during testing.

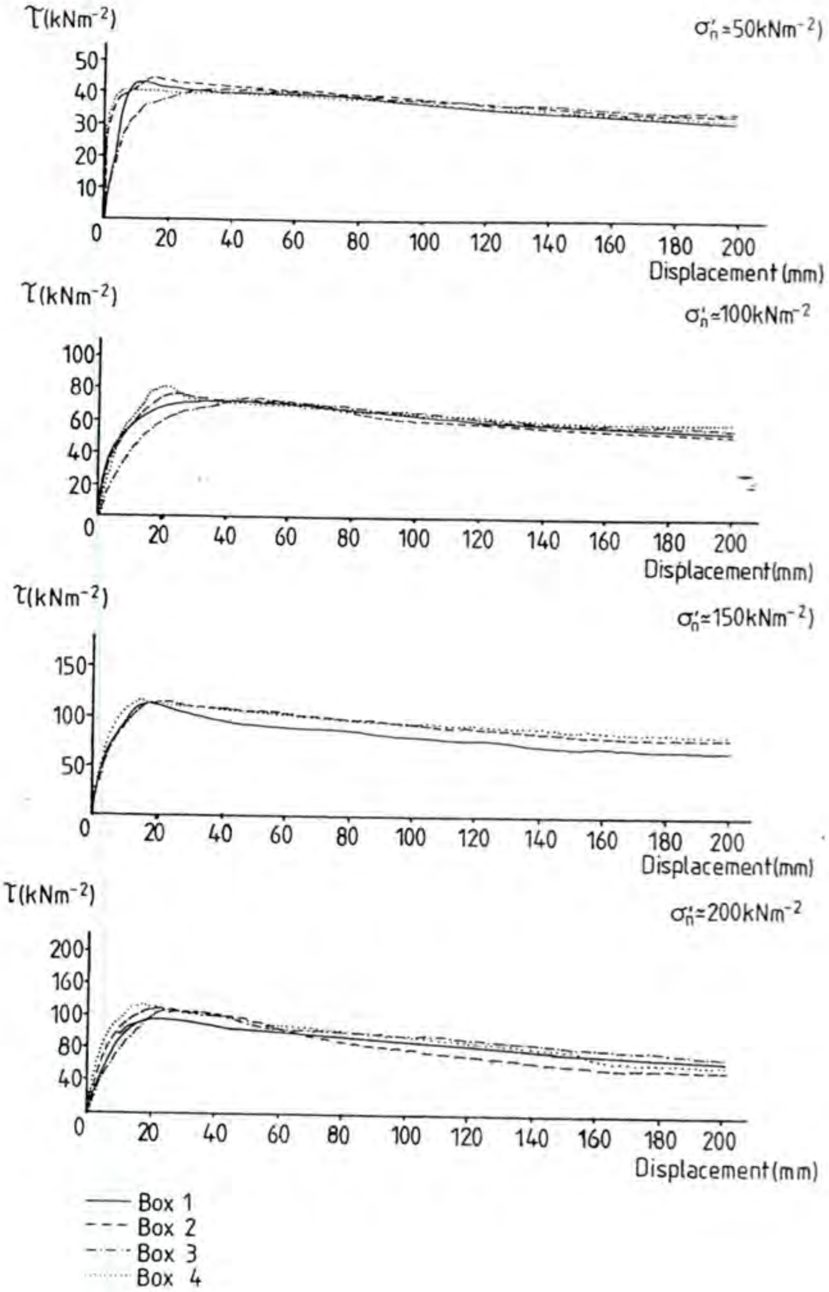


Figure 6.3 a – d: Shear stress – displacement curves: Coal Measures shale passing 2mm, at different normal loads. Tests to check precision of small shear boxes.

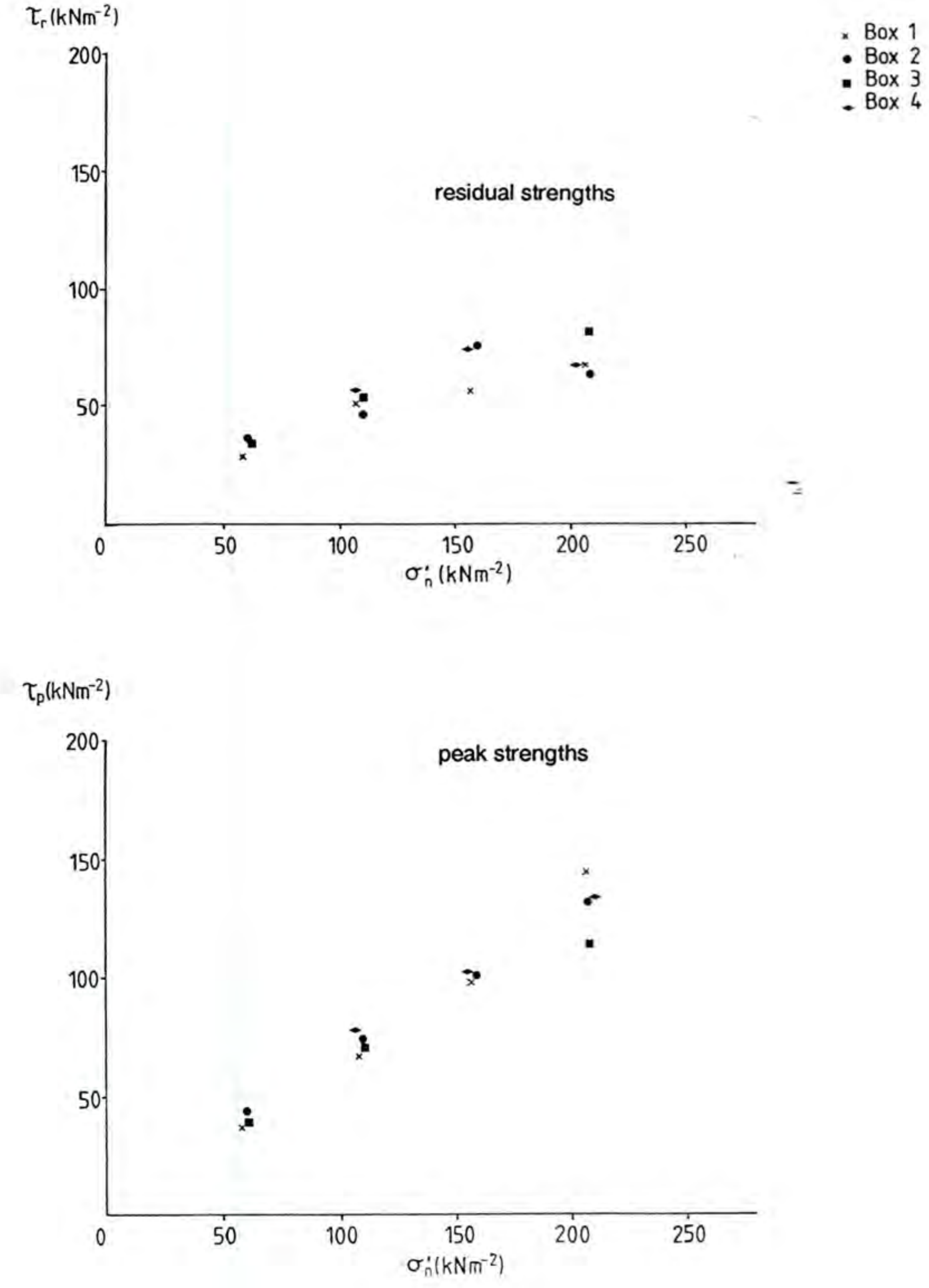


Figure 6.4 a and b: Shear stress – normal load: Coal Measures shale passing 2mm. Tests to check precision of small shear boxes.

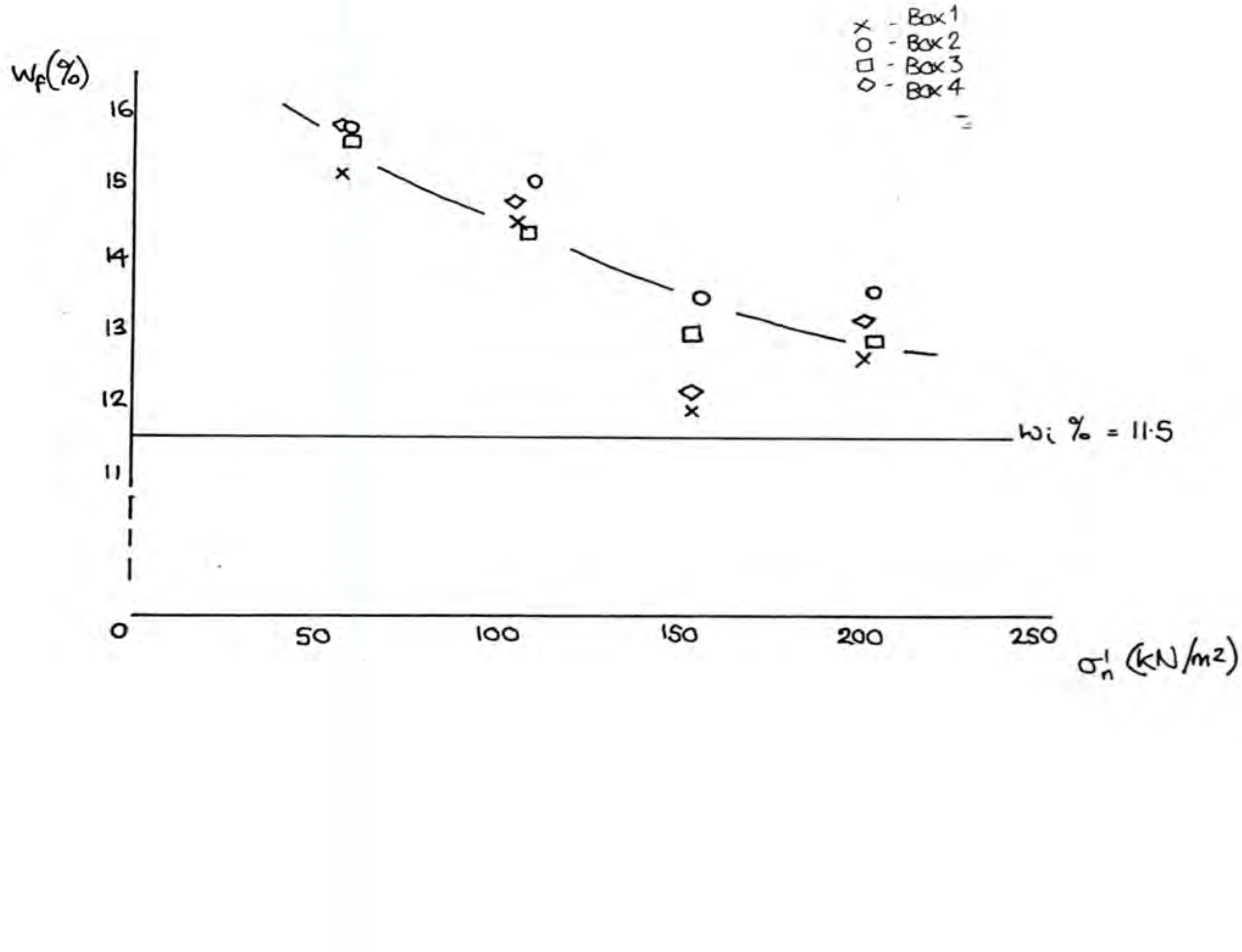


Figure 6.5: Initial and final moisture content: Coal Measures shale passing 2mm. Tests to check precision of small shear boxes.

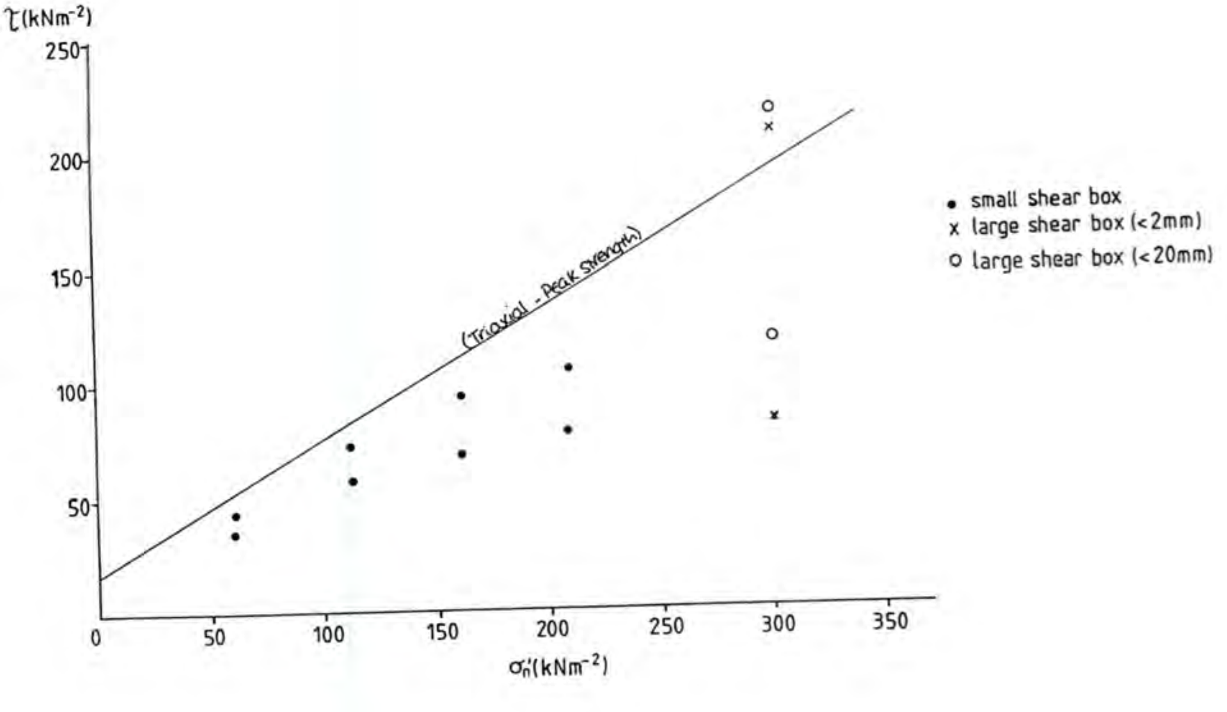


Figure 6.6: Shear strength – normal load: Coal Measures shale.

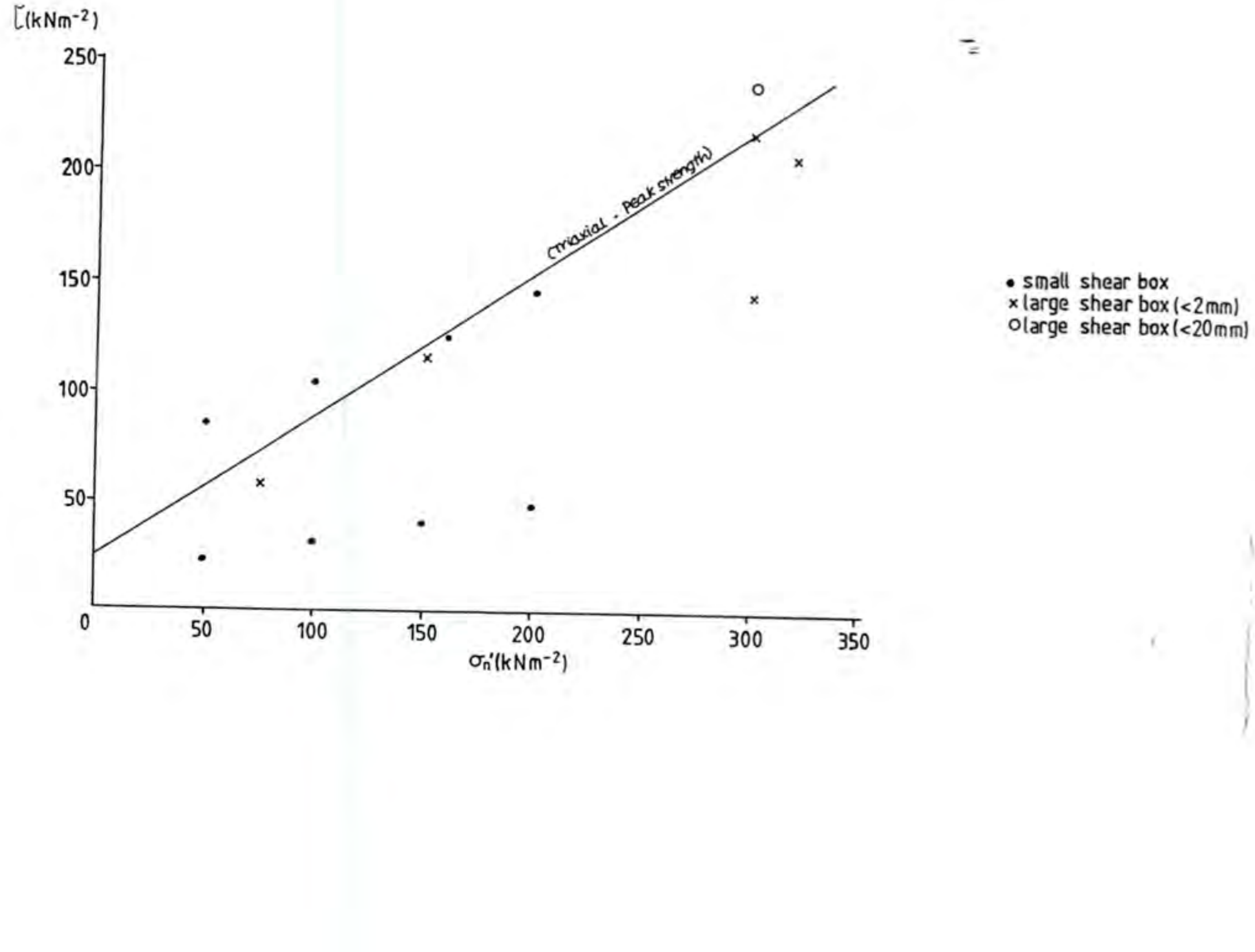
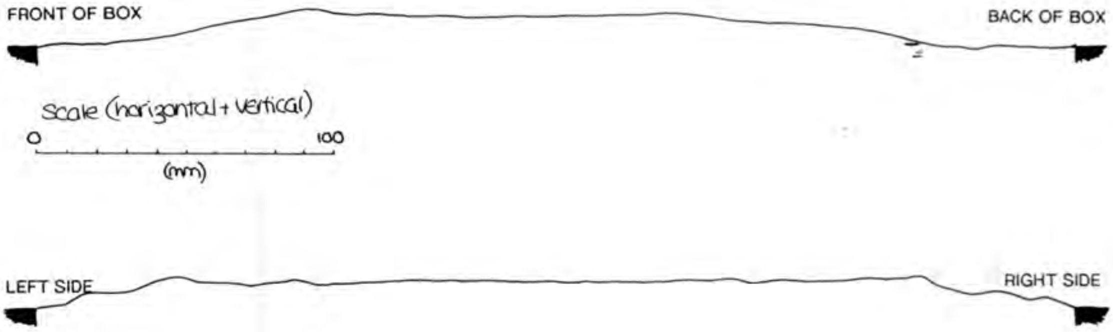


Figure 6.7: Shear strength – normal load: Kimmeridge Clay.

Kimmeridge Clay - Large shearbox test, sample passing 2mm



Kimmeridge Clay - Large shearbox test, sample passing 20mm

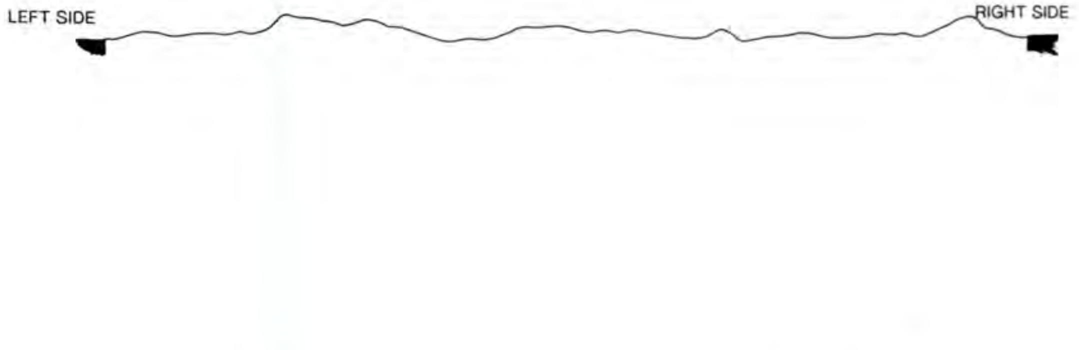


Figure 6.9: Profiles across shear planes produced in Kimmeridge Clay (a) passing 2mm, and (b) passing 20mm.

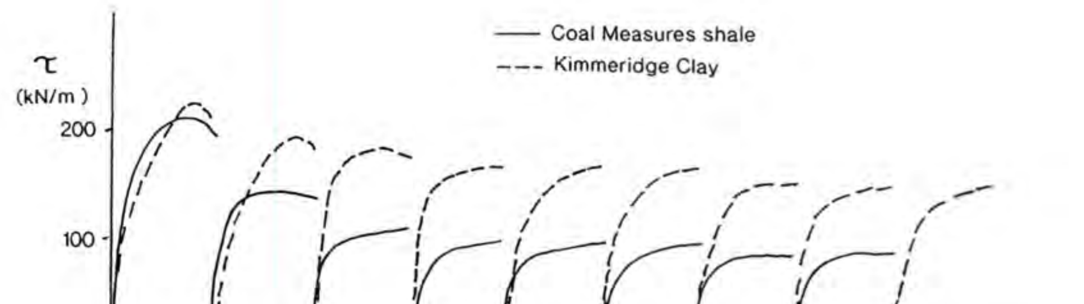


Figure 6.10: Shear stress – displacement curves: Coal Measures shale and Kimmeridge Clay passing 2mm.

Key to Figures 6.11 to 6.18

small shear box (particle size <2mm)	peak shear strength	●
	residual shear strength	○
large shear box (particle size <2mm)	peak shear strength	✱
	residual shear strength	×
large shear box (particle size <20mm)	peak shear strength	■
	residual shear strength	□
large shear box (undisturbed sample)	peak shear strength	◆
	residual shear strength	◇

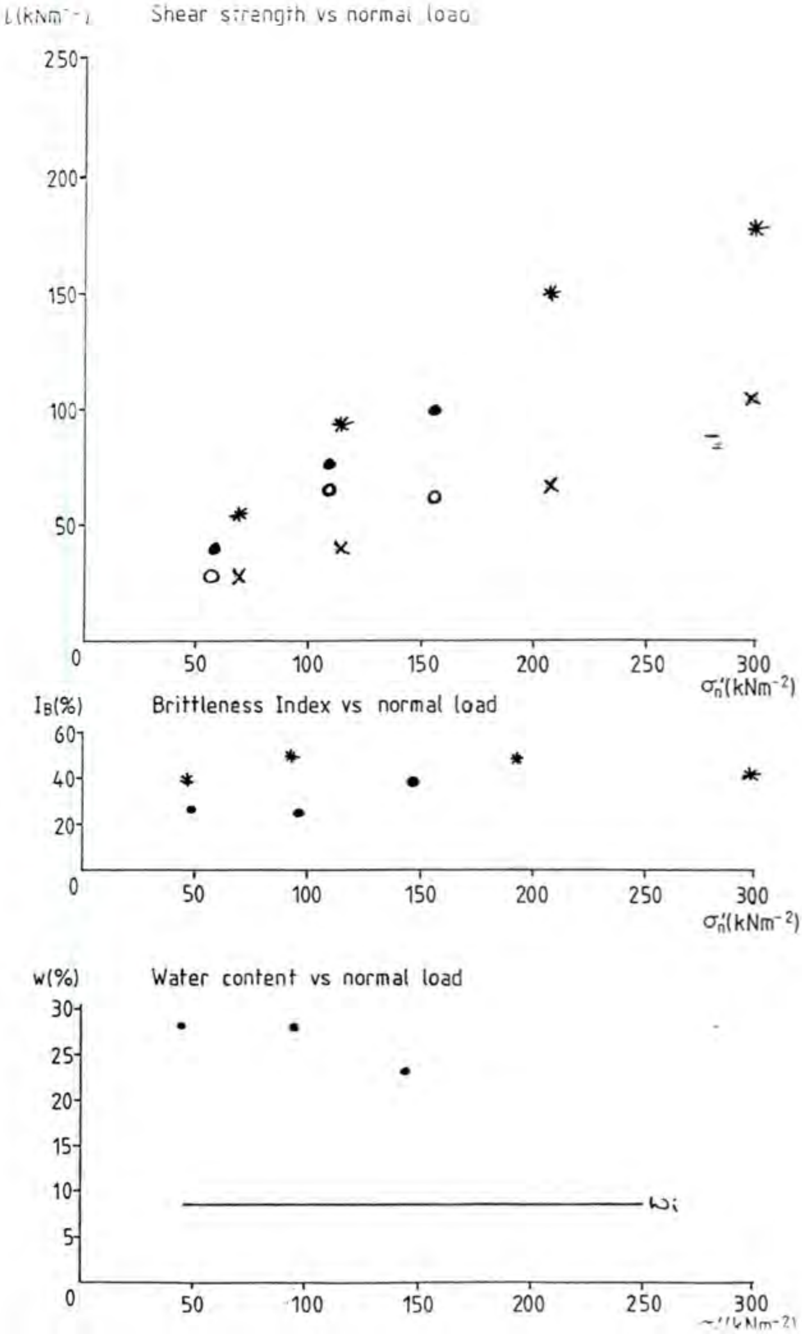


Figure 6.11: Shear stress – normal load, brittleness index – normal load and moisture content – normal load: Gale Common fresh shale.

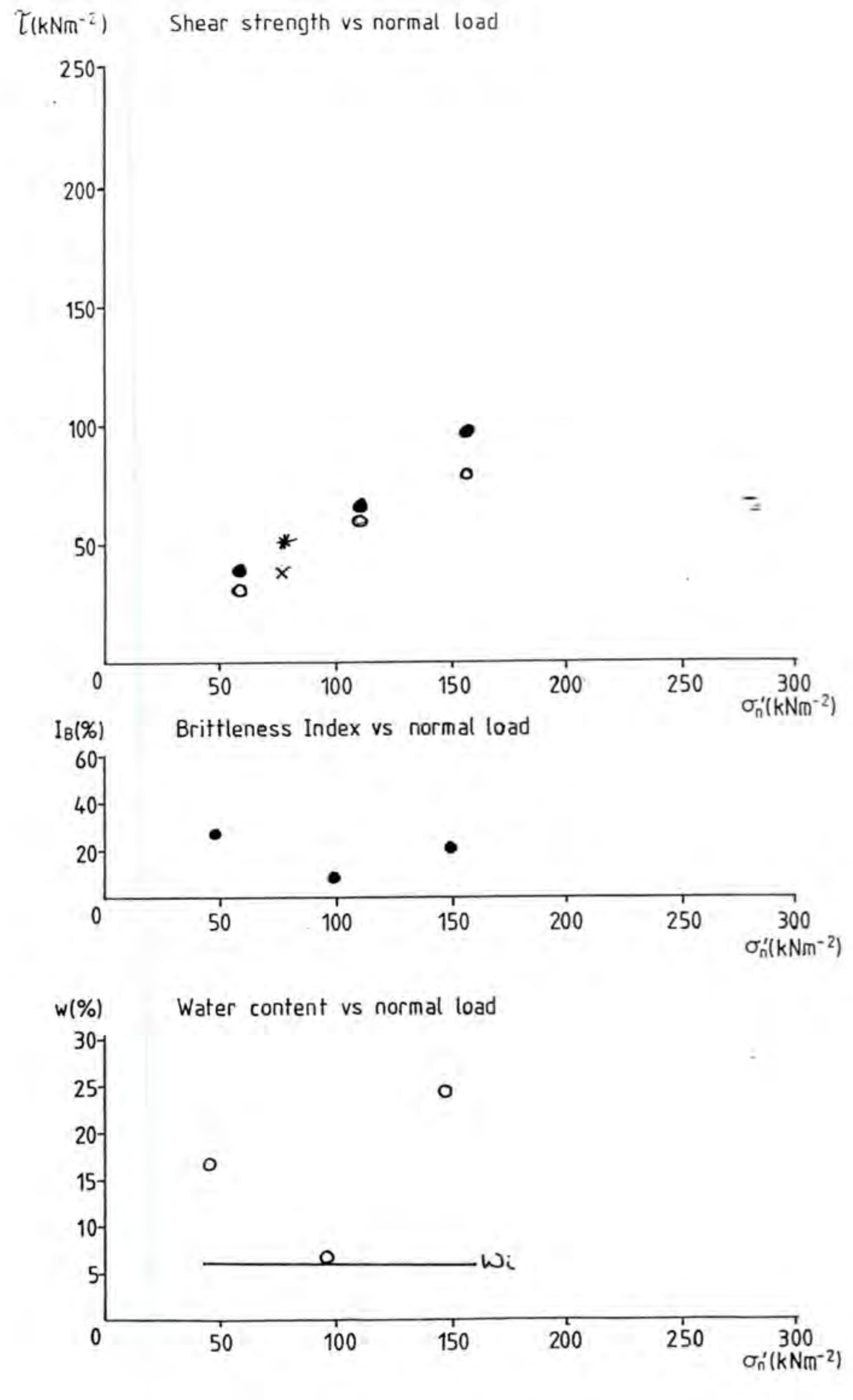


Figure 6.12: Shear stress – normal load, brittleness index – normal load and moisture content – normal load: Gale Common stockpiled shale.

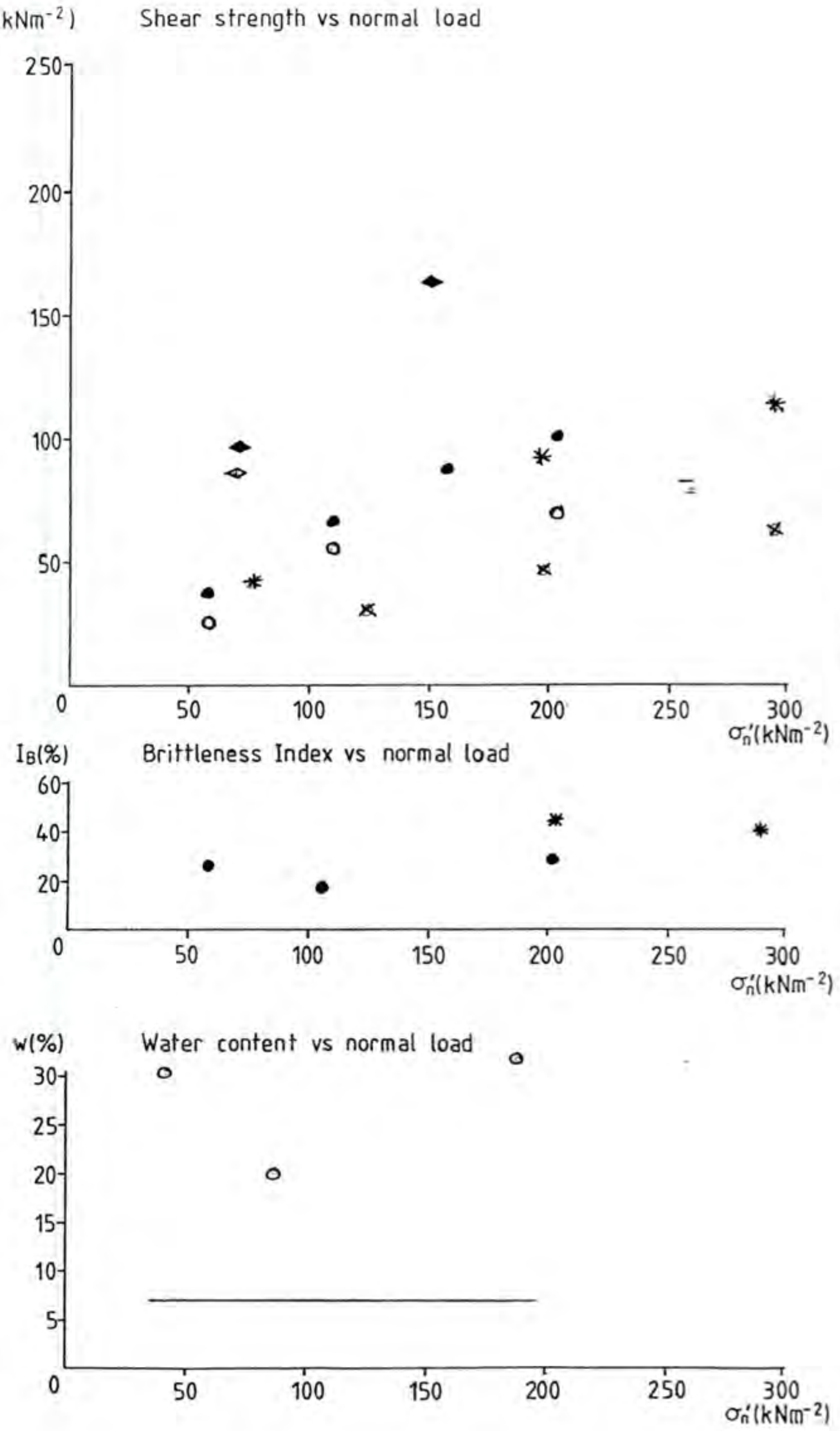


Figure 6.13: Shear stress – normal load, brittleness index – normal load and moisture content – normal load: Gale Common five year weathered shale.

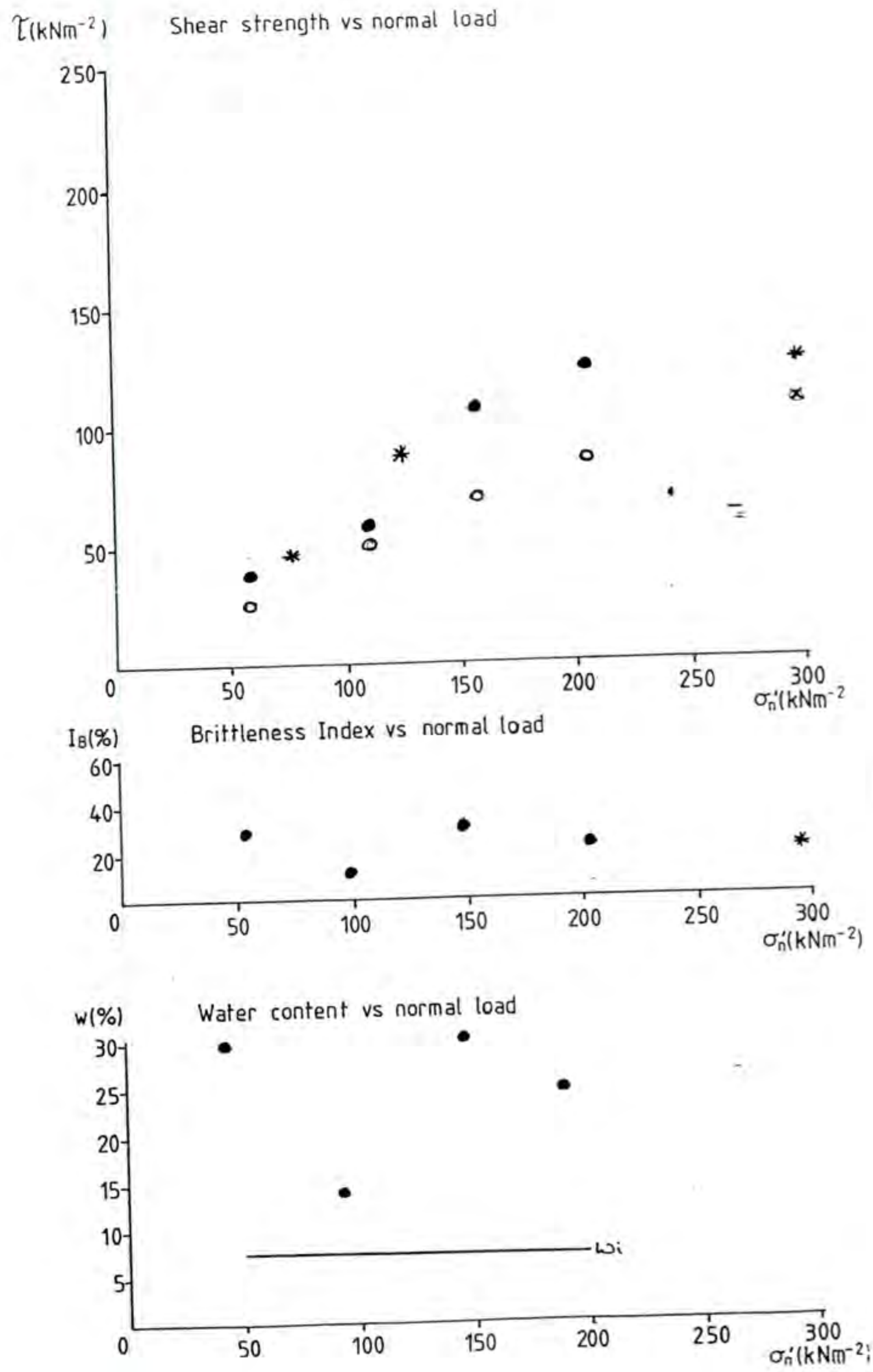


Figure 6.14: Shear stress – normal load, brittleness index – normal load and moisture content – normal load: Gale Common fifteen year weathered shale.

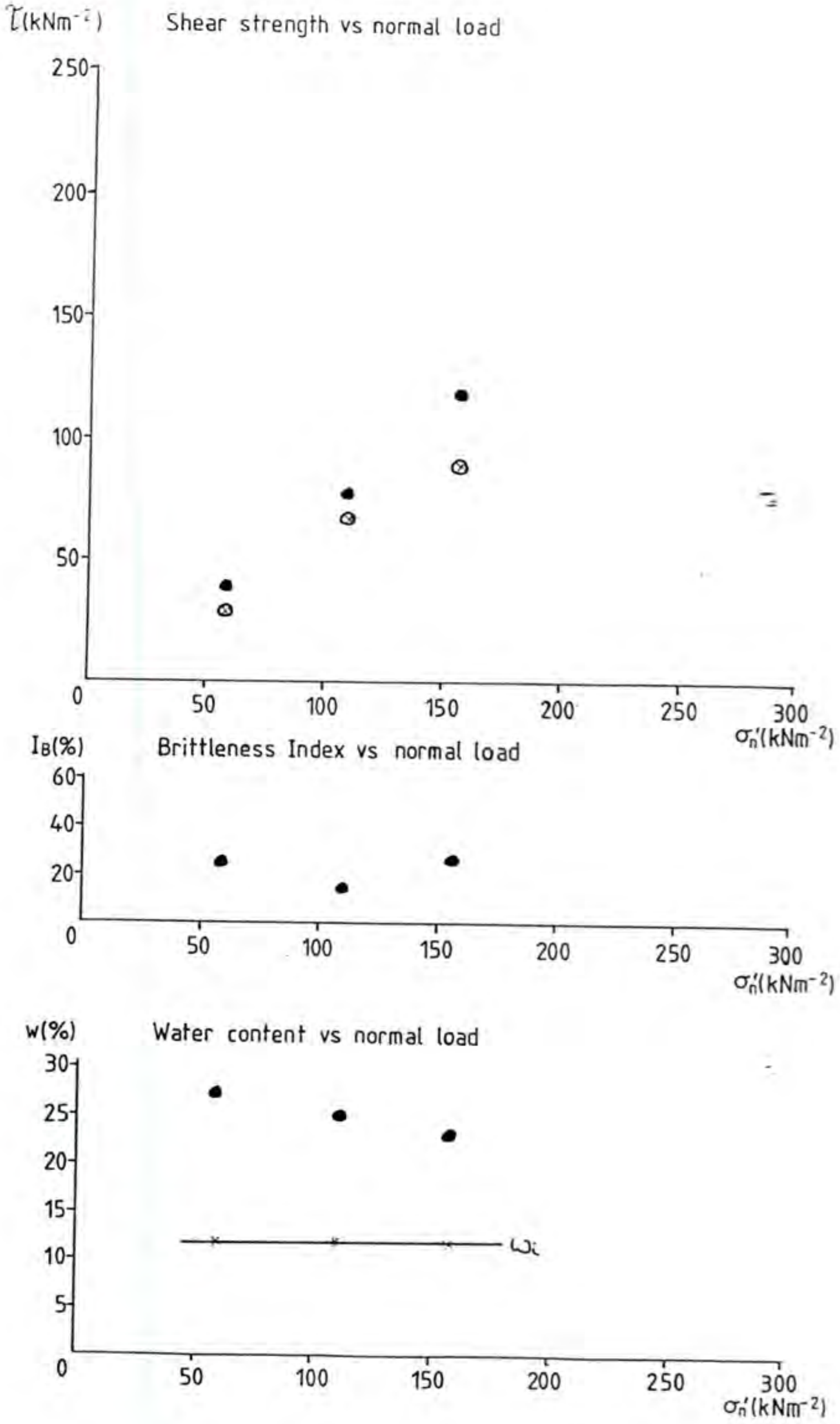


Figure 6.15: Shear stress – normal load, brittleness index – normal load and moisture content – normal load: Grassholme Dam.

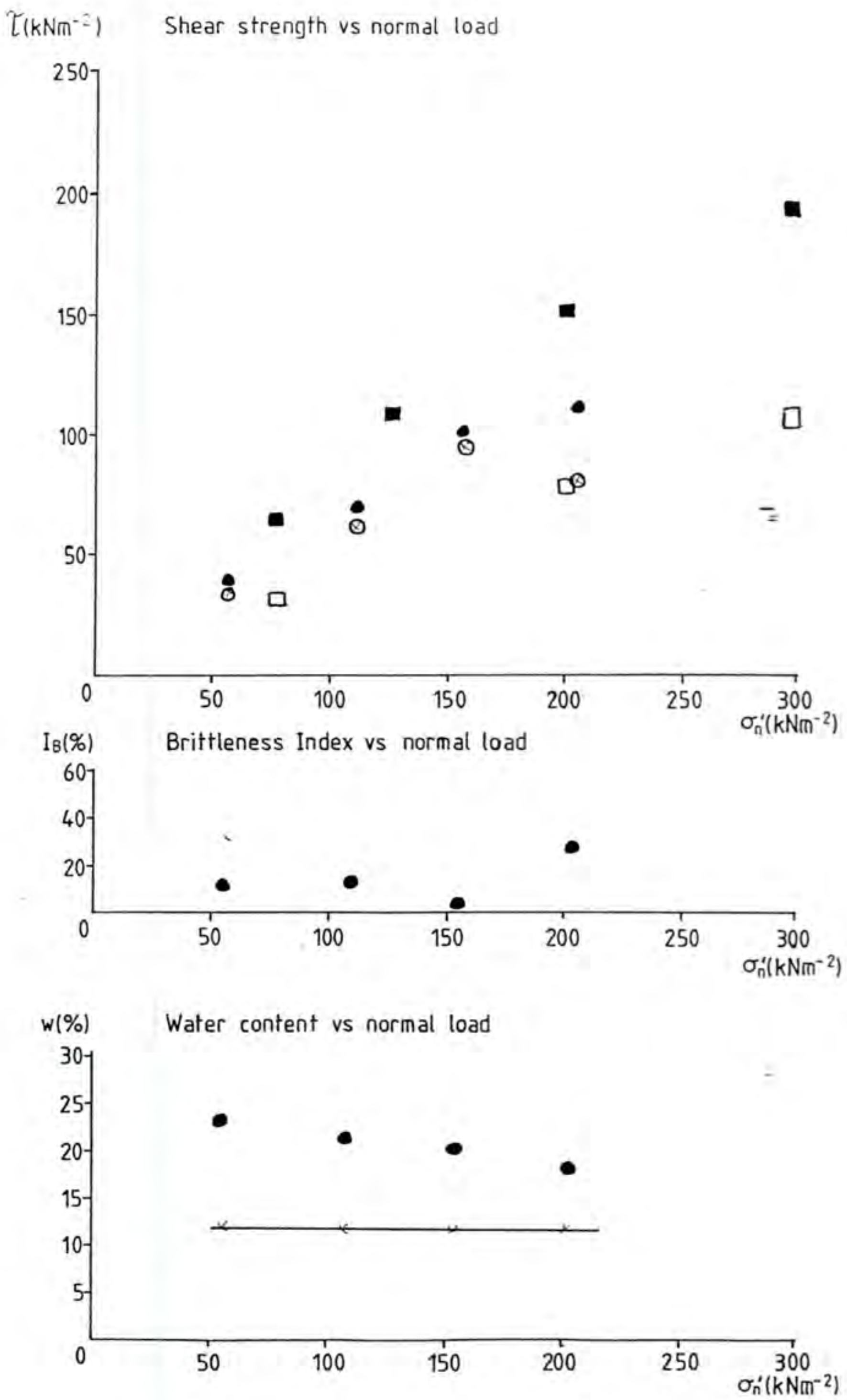


Figure 6.16: Shear stress – normal load, brittleness index – normal load and moisture content – normal load: Balderhead Dam.

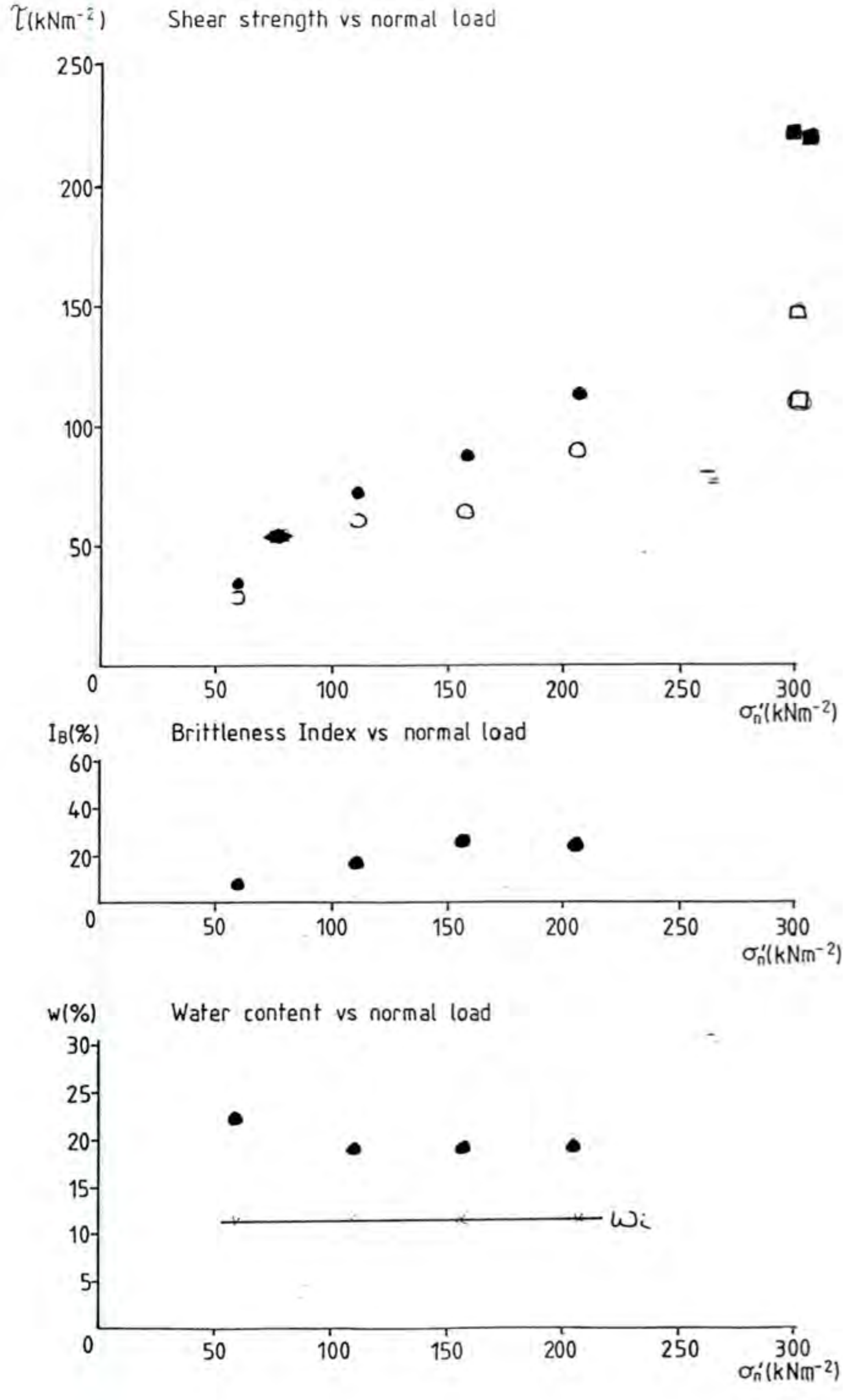


Figure 6.17: Shear stress – normal load, brittleness index – normal load and moisture content – normal load: Grassholme – Hury tunnel spoil heaps.

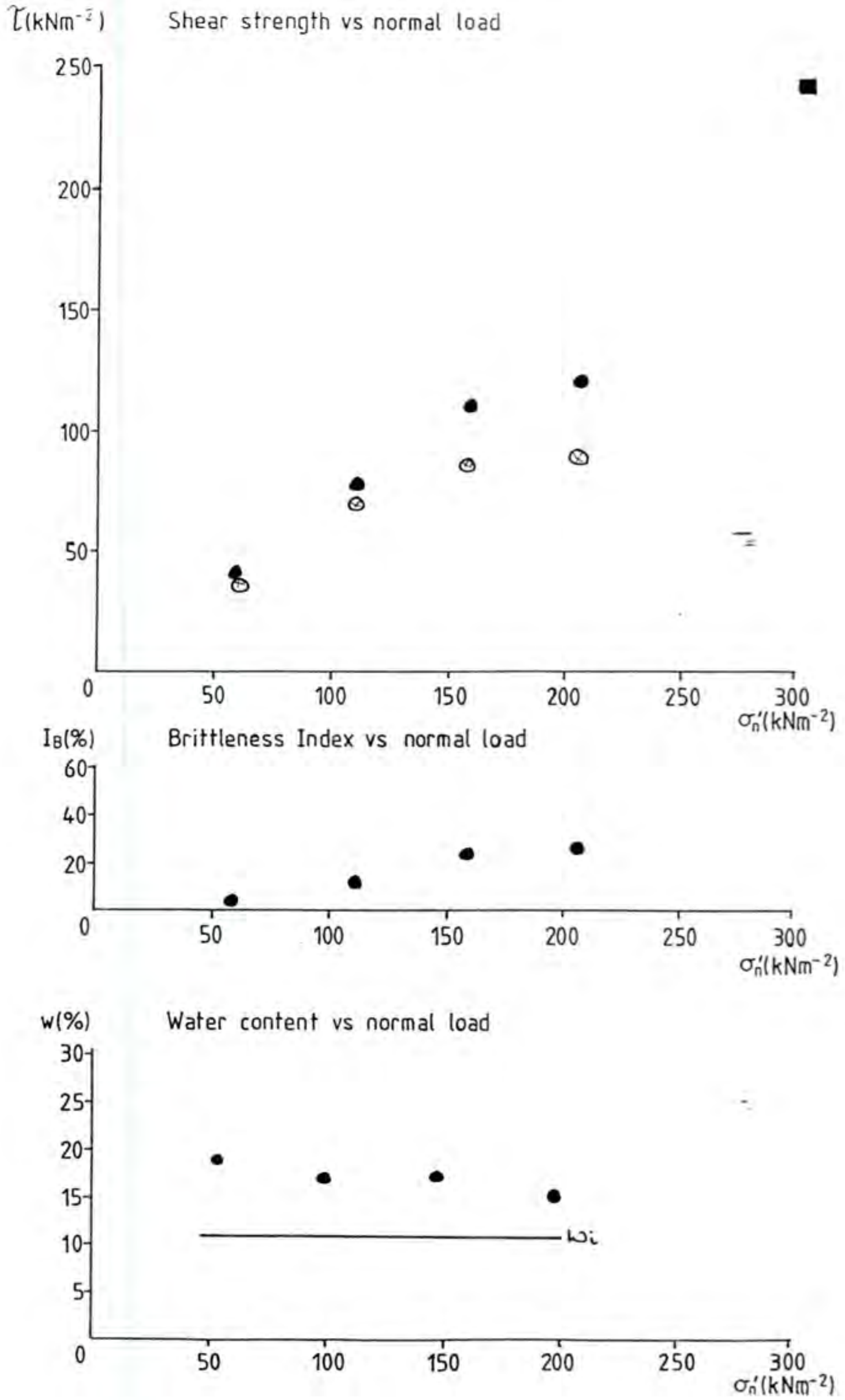


Figure 6.18: Shear stress – normal load, brittleness index – normal load and moisture content – normal load: Burnhope Dam

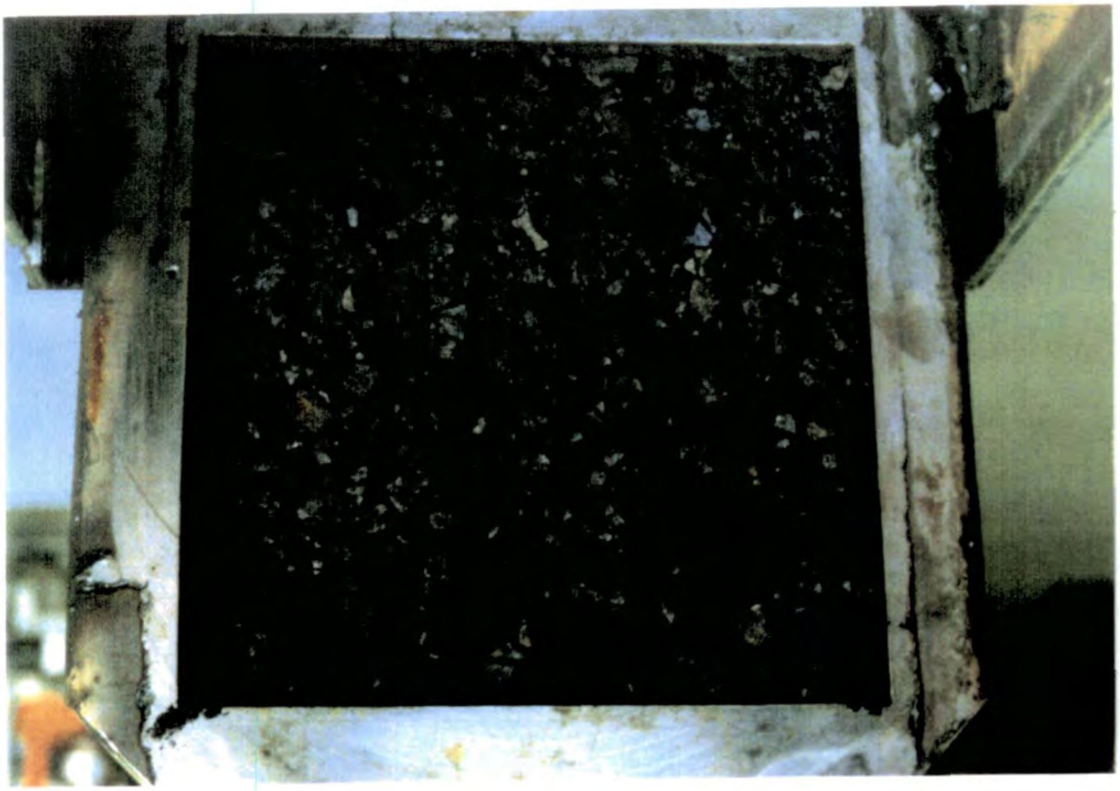


Plate 6.1: Clast size range used in large shear box test

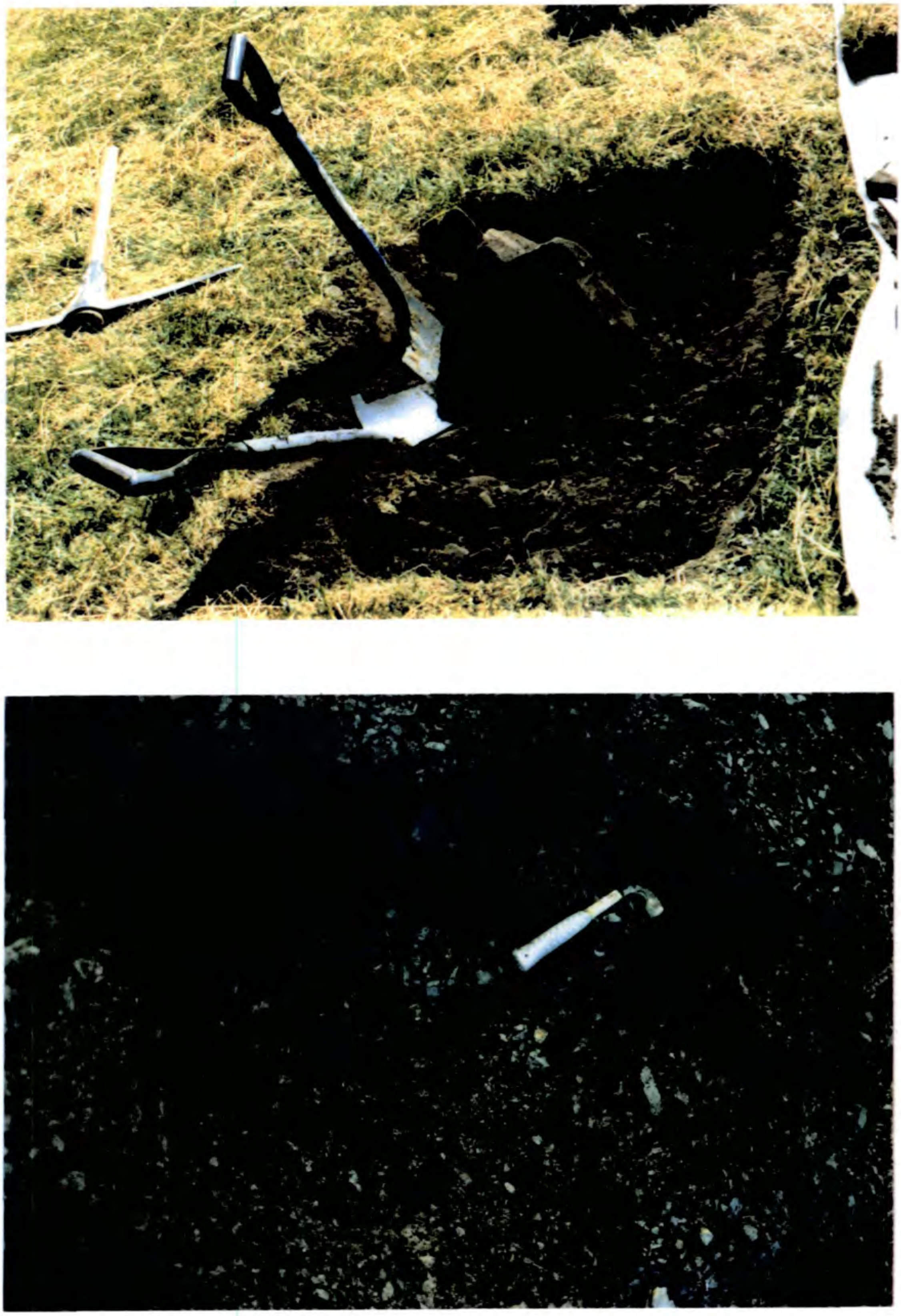


Plate 6.2: Clast size range used in typical pits dug in shale fill dams.

Plate 6.3: Interruption of shear plane by large clast in shear box test (undisturbed sample, Gale Common 5 year weathered shale)

Plate 6.4: Sample from small shear box split after testing to reveal shear plane (Coal Measures shale passing 2mm sieve).

Plate 6.5: Shear plane from large shear box test (fresh Coal Measures shale passing 2mm sieve).

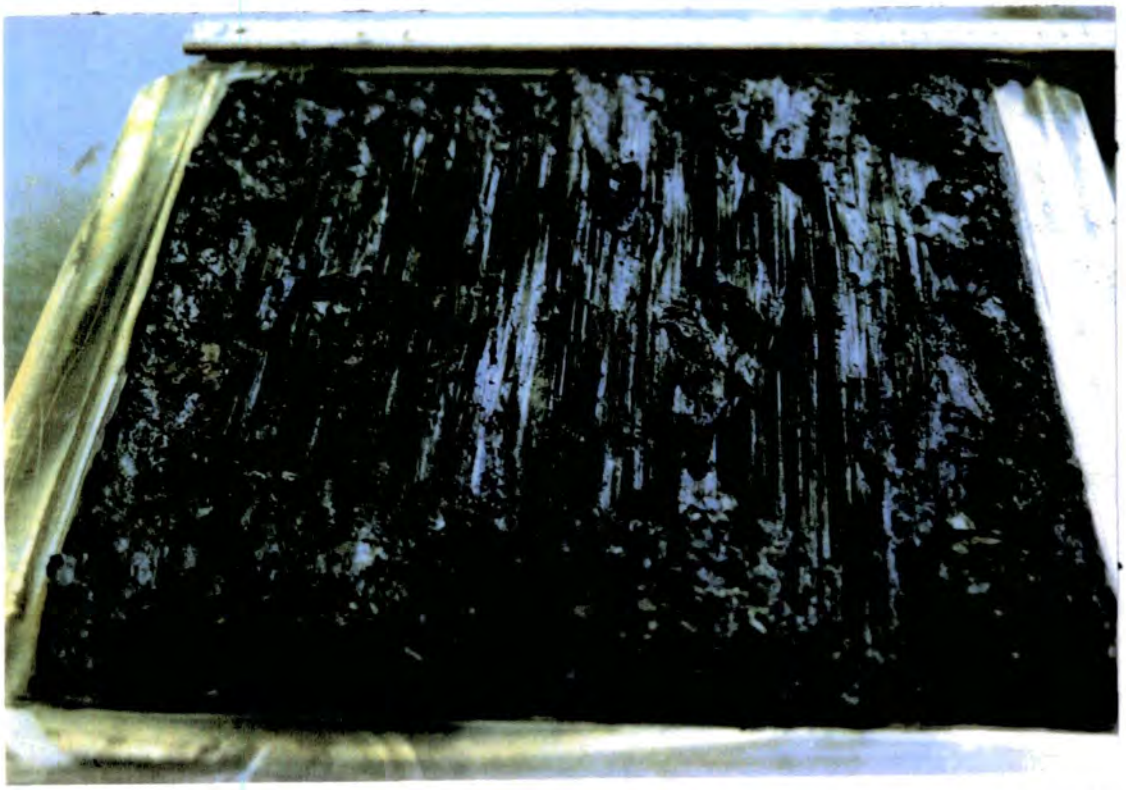


Plate 6.6: Shear plane from large shear box test (Gale Common 15 year weathered shale passing 20mm sieve)

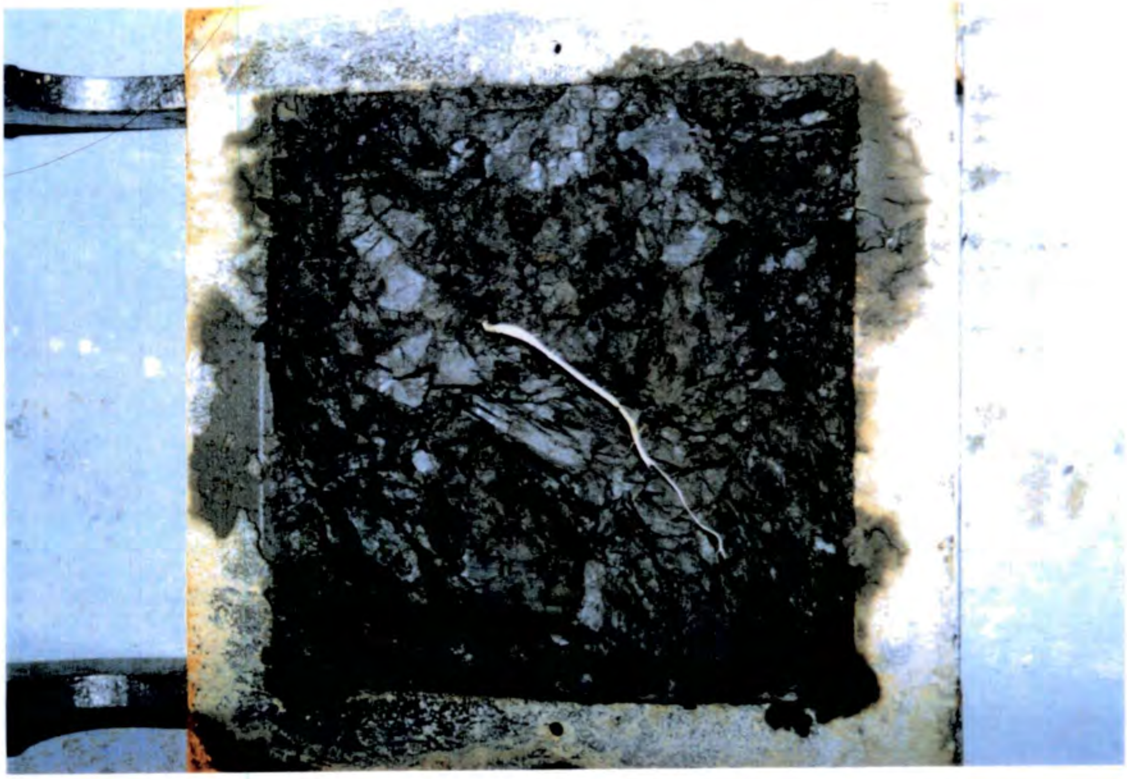


Plate 6.7: Shear plane from large shear box test (undisturbed Namurian shale from Grassholme Hury tunnel spoil heap.

Plate 6.8: Close up of shear plane from Plate 6.7.

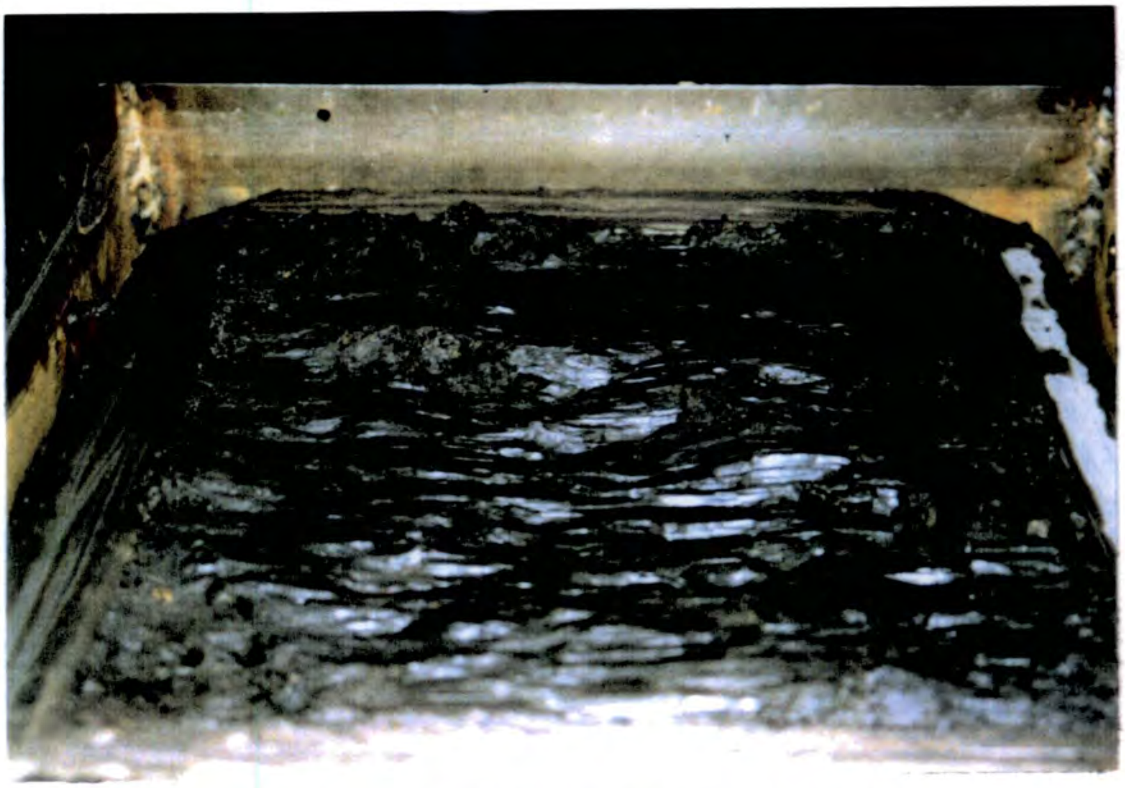


Plate 6.9: Shear plane in undisturbed sample of Namurian shale showing distortion of plane by large clasts.

Plate 6.10: Crystalline growths around box lid observed during large shear box test (Gale Common 5 year weathered shale).

Plate 6.11: Close up of crystalline growth shown in Plate 6.10.

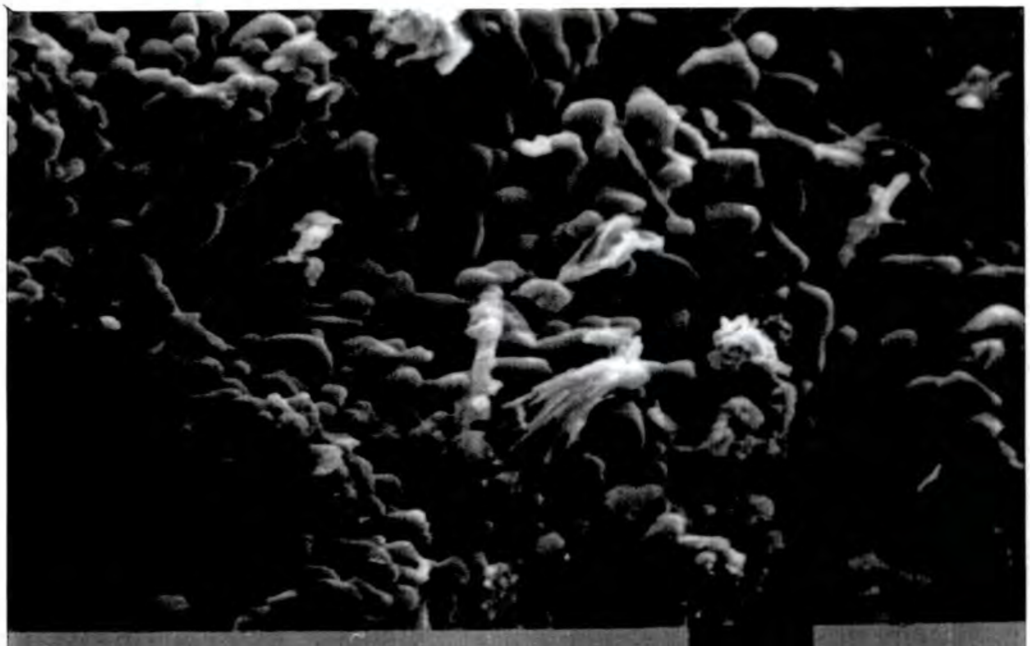


Plate 6.12: Electron micrograph of growth shown in Plate 6.10 showing thenardite and halite crystals.

Chapter 7. DISCUSSION

7.1 Integration of geotechnical and mineralogical data

The general geotechnical properties of the samples tested were determined according to the methods described in BS 1377 (1975), and were presented in Chapter 3. In addition to this, geological and chemical characteristics were discussed in Chapter 4. This Chapter reviews the nature of possible correlations between such properties and shear strength, and makes note of any such correlations in the samples tested.

7.1.1 Shear strength and geotechnical properties.

There have been numerous attempts in the past to link shear strength with various geotechnical properties. For example, residual shear strength with clay fraction, residual strength with plasticity index, and residual strength with liquid limit. Kenney (1968) links residual shear strength with plasticity index and with grain size, although in both cases stresses that there is no unique relationship. Similarly, Lupini *et al* (1981) note that residual shear strength does change significantly with increasing clay content, but again make no general correlations with index properties. They proposed two extremes of shear behaviour: Sliding shear in which flat (clay) particles slide over each other, and turbulent shear which occurs when equant (non-clay) minerals attempt to move past each other. Most natural soils, containing a mixture of clay and non-clay minerals, will exhibit a hybrid form of behaviour. The mechanism, however, explains why the increase in clay fraction will alter shear properties.

Hawkins and Privett (1985) suggest that correlations may indeed be valid, but only at certain stress levels. They also show that where the clay fraction exceeds 60% and the plasticity index is greater than 45%, the residual angle of friction is low, and there is little scatter observed in the data. Consequently correlations will be valid. However where clay fractions were in the range 35 – 60% and plasticity indices between 25 – 45% the data showed wide scatter. Under these conditions, which are those typically encountered in British soils, it is inappropriate to employ the use of correlations.

On account of the large amount of scatter experienced, Collotta *et al* (1989) proposed a combination measure 'CALIP' which is calculated as shown below on the basis that the properties discussed are all related.

$$CALIP = (CF)^2 \times LL \times PI \times 10^{-5} \qquad 6.1$$

where CF = clay fraction

The measure was tested in a study of 150 silty clay soils from the Appenines and hard Pliocene clays, flysch and clay shales from various Italian sites and was found to show less scatter when plotted against residual shear strength than the individual soil properties. Furthermore, for soils with CALIP greater than 60 the ring shear and direct shear box test results were found to be roughly similar. For values less than 60 then direct shear box test results exceeded those from the ring shear box by 15 – 20%.

Relevant geotechnical data for the samples tested are summarised in Table 7.1 and graphs these parameters plotted against shear strength given in Figure 7.1. The shear strength data for all sites has been collated in Figure 7.2 in its normal form, and in log form in Figure 7.3. Brittleness Index data, again for all the sites tested, is summarised in Figure 7.4.

7.1.2 Shear strength and Mineralogy

The conclusions drawn by Kenney (1967 and 1977), are well known, and form the basis of the sliding versus turbulent shear model discussed above. These are that layer lattice minerals tend to have lower shear strength than massive minerals, and that particle size and shape is an influential factor in the behaviour of massive minerals.

Having established that residual shear strength is influenced to some degree by the amount of clay minerals present, it is then necessary to determine whether the type of mineral present will have any effect. Skempton (1985) stated that if the clay fraction is greater than 50% then the mineralogy becomes important. The following typical values for residual angle of friction are often quoted;

Kaolinite	15°
Illite	10°
Montmorillonite	5°

The possible relationships between mineralogy and shear strength of the samples tested are summarised in Figure 7.5.

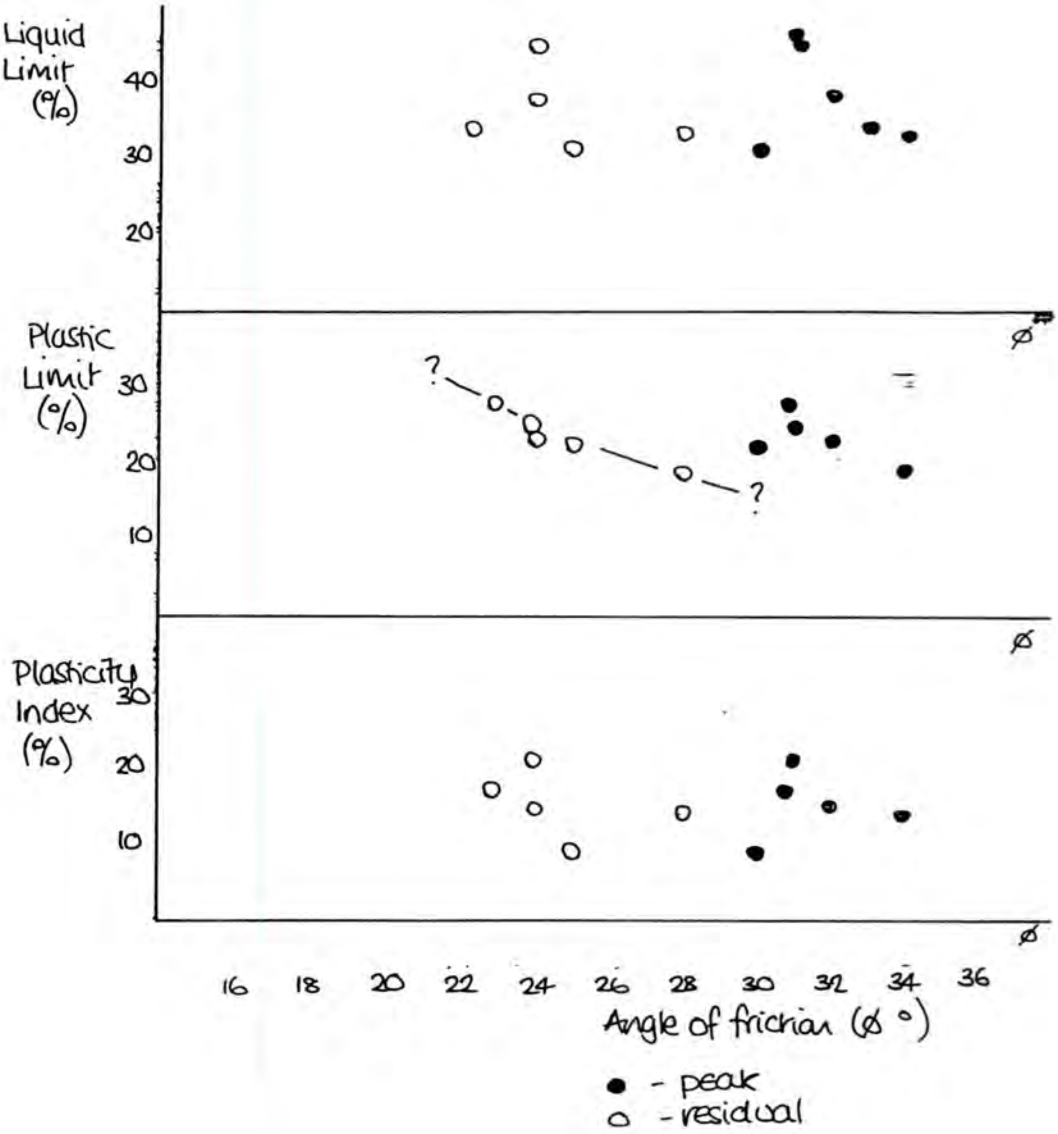


Figure 7.1 : Relationship between shear strength and other geotechnical data.

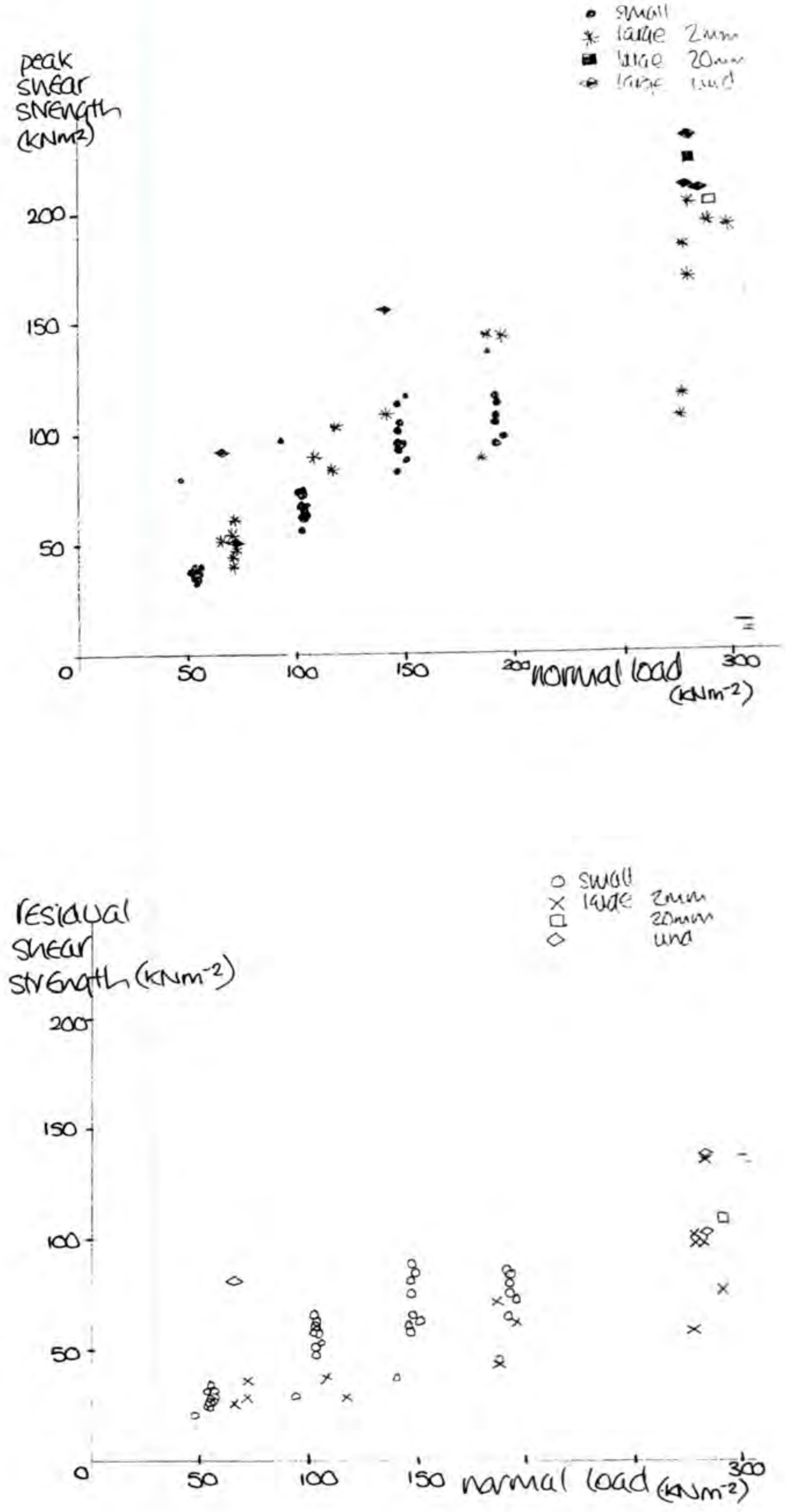


Figure 7.2 : Summary of shear strength data for all samples tested.

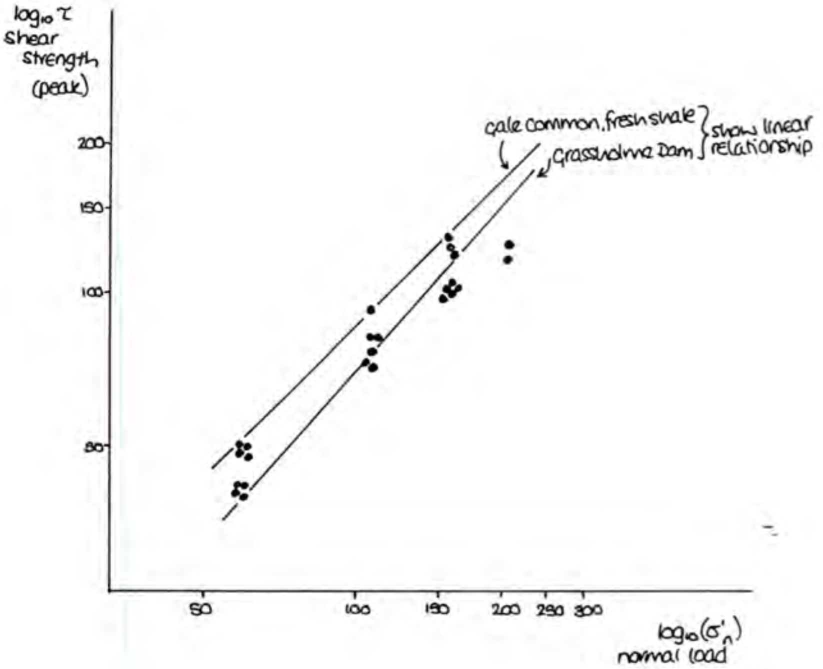


Figure 7.3 : Summary of shear strength data for all samples tested – in logarithmic form.

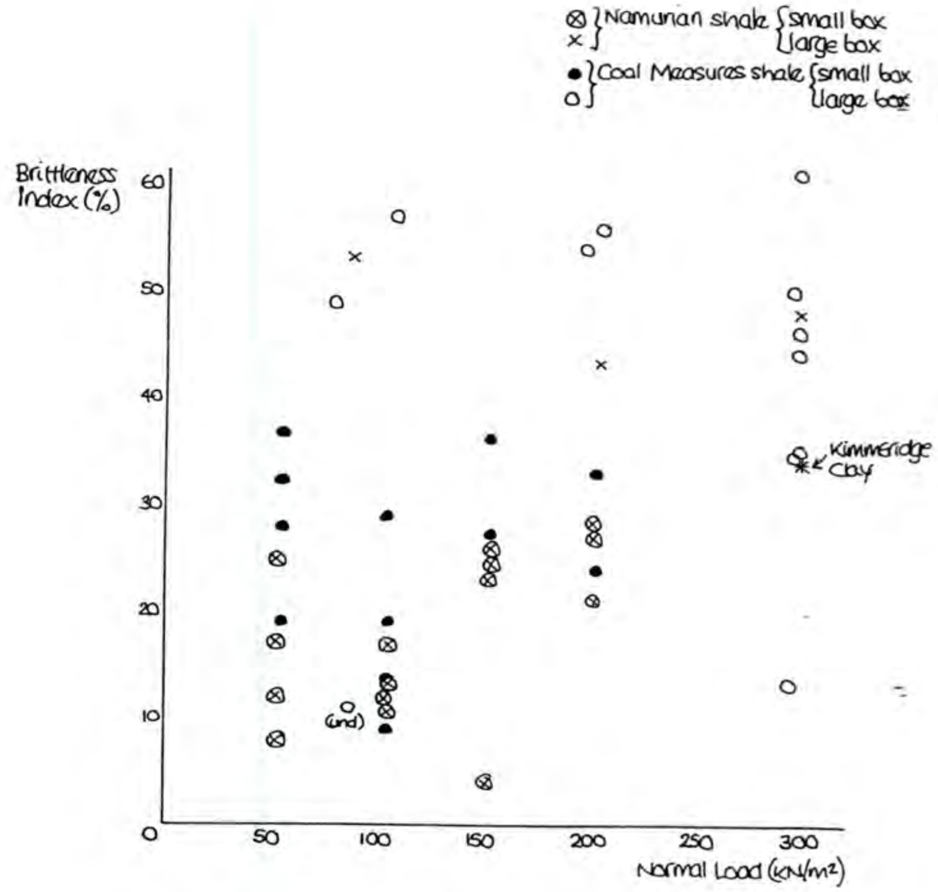
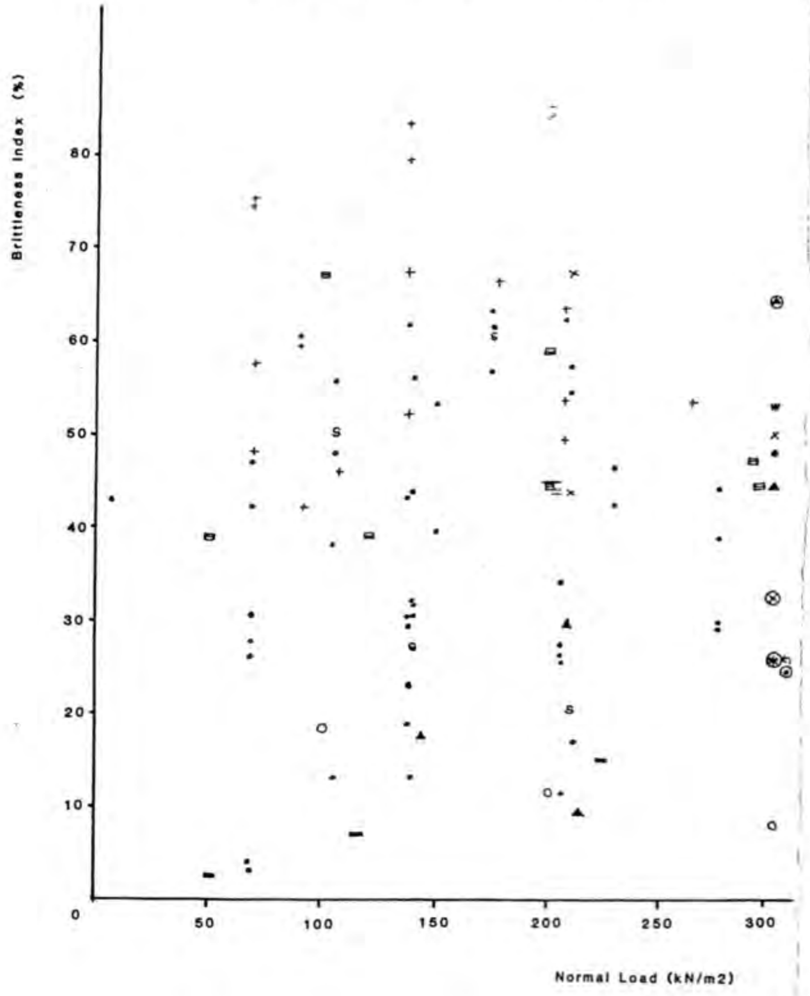


Figure 7.4 : Summary of brittleness data for all samples tested.

BRITTLENESS INDEX AGAINST NORMAL LOAD

USING DATA FROM PAST MSc DISSERTATIONS



KEY

▬	Greenschist Facies Slate (U.Devonian)
▲	Namurian Shale
•	Coal Measures Shale
S	Seat Earth
×	Kimmeridge Clay (U.Jurassic)
✱	Weathered Kimmeridge Clay
+	Ampthill Clay (U.Jurassic)
o	Boulder Clay (Quaternary)
⊟	Laminated Clay (Quaternary)

Chapter 8. SUMMARY AND CONCLUSIONS

The failure at Carsington dam, described in Chapter 1 as a means of introduction to this work, provoked a great deal of interest in several aspects of earth fill dam behaviour. The general consensus of the research and review into the causes of the failure is that the collapse was due to progressive failure triggered by a number of mainly geometrical factors and the low shear strength of certain materials. However, not everyone agrees with this conclusion, for example Rowe (1992) attributes the failure to the presence of shears caused by the movement of heavy plant on wet plastic clay. Using the finite element methods now available workers have re-analysed several other dam failures, for example that at Chingford (Dounias, personal communication).

On the face of it, the situation at Aberfan appears to have been completely different. The failure there occurred over a much shorter time span, the failed mass of material travelled over a much greater distance, and there was loss of life as a direct result of the failure. In the case of Carsington Dam the main problem was the inconvenience caused and the resultant increase in costs. There was no direct threat to public safety although secondary effects of pollution to water and air did cause problems. However the two failures can be considered as end members in a spectrum of the expression of brittle behaviour in soils. In both cases excavation of the failed mass revealed a shear plane on which the shear strength had been reduced to its residual value.

More specifically, within the scope of the work in this thesis, there has been the need to understand the behaviour of shale fills if they are to be used satisfactorily in embankment construction. Both mechanical and mineralogical/chemical factors need to be considered. It was proposed that there are two methods by which the strength of a shale fill reduces with time: First by progressive failure, and second by chemical and physical degradation. Overstressing, initiated by an external factor such as anomalously high porewater pressures, a geometrical corner or boundary, will cause stress to build up at that point. This then causes the loading of adjacent material which will fail and in turn transfer stress onto surrounding material. This process, progressive failure, is repeated in a chain reaction throughout the structure so that at any one time some of the soil is at its residual strength value, some will be at or close to peak strength, and some will not have been stressed at all. Complete failure will occur when the maximum *available* shear strength is unable to counteract the applied stress. The alternative process is by a combination of physical and chemical degradation and weathering which leads to an increase in void ratio of the shale, and therefore a corresponding loss of strength. It may be possible to reduce the available shear strength of a shale or overconsolidated clay to its critical state if a large enough amount of dilation occurs, so that the material can only mobilise the equivalent of a normally consolidated shear strength. In clayey materials further mechanical shearing is required, however, to bring the shear strength to its residual value because particle alignment is required to reach this condition. Marine mudrocks appear to be particularly prone to this latter process of shear strength reduction because they are generally thinly laminated and/or fissile (and as such, therefore, should be classified as shales) and thus more liable to breakage and slaking. They also tend to contain minerals such as pyrite and calcite which render them particularly vulnerable to chemical decomposition. Taylor (1988) reports that concentrations of pyrite as low as 5% can have a significant effect. The random clay mineral fabric frequently observed in marine clay deposits is often invoked as a cause of instability in marine mudrocks. However, much of this disorder becomes ordered during compaction and diagenesis, and furthermore, Smith (1978) reports that the presence of smectite appears to be important in maintaining such a structure. In the Carboniferous rocks considered in this study smectite was absent and the structure was ordered and stable. *

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* Specimens of shales were studied under the scanning electron microscope to confirm this; however no photographs are available for inclusion in the thesis

The Oxford English Dictionary defines the word brittle as 'easily cracked, snapped or broken; fragile'. A selection of materials science and engineering text books yielded several references to brittle fracture, but few to brittleness. Kempster (1975), in stating that the 'tendency to fracture without visible plastic deformation is called brittleness' is an exception. Brittleness to the 'person in the street' is something instantaneous, or at least very rapid (even explosive), and involves complete breakage or fracture. This does not seem to correspond with observations of slope failures where failure may take place over a long period of time, and although the failure may result in the formation of a shear plane which separates the slipped mass from that below, it does not produce two obviously-broken halves. Moreover, although the shape of stress-strain curves for brittle solids and soil are generally similar up to the point of failure, the drop in strength, followed by a constant residual value is not observed in solid materials. It may prove necessary to define a less misleading term if brittleness in soils is to be discussed in a full materials setting.

Brittleness in soils and fills is caused by a combination of dilation (increase in void ratio) to reach critical state together with particle breakdown and alignment of clay minerals where present. If these two effects do not take place, then behaviour will be non-brittle. The former is a function of environment and packing rather than of material type, with the latter being controlled by material type. It therefore follows that any attempt to measure brittleness must take into account the fact that the test system will impose a type of behaviour on the sample (as well as considering factors which affect the measured strength). For example, the stiffness of the testing machine and its ability not to inject elastic strain energy into the sample due to imbalance at and immediately after the point of failure is an important consideration. It is essential to model the correct environment, that is, one cannot estimate the brittleness of a rock fill by selecting an aggregate sample and testing under ideal conditions then assuming that that brittleness value will apply in the field. Inadequate compaction for example may lead to failure because packing will control the *in situ* brittleness. If one is trying to avoid progressive failure it is as important not to promote conditions such as this where a material will behave in a brittle manner, as it is to try and use a non brittle material. The

Brittleness Index is rather loosely defined – Bishop admitted that when he defined it, but it is still the only measure of brittleness available. Although not used universally, when it has been adopted it has been applied to a whole spectrum of failures from rapid flow slides to long-term slope stability problems. Limited data gathered on material brittleness suggests that brittleness is not related to age, but decreases with increasing confining pressure or normal load. It is not necessarily related to either peak strength or residual strength, and brittleness in unweathered materials is generally higher than in their weathered counterparts.

Evidence from Carsington dam suggested that there was a significant decrease in shale strength near to the surface of the dam and this occurred during the construction period. Although shale in such a position was not involved in the actual failure it does show that rapid degradation of the materials used in construction can occur in a relatively short time span, and this may be critical.

Mudrocks and shales, as defined in Chapter 3 *, have been little studied until recently despite the fact of their dominance within the geological column. It is essential to study them for engineering purposes because when the amount, distribution and general availability of mudrocks and shales is considered, it is then inevitable that they will be used in the building of embankment structures. They are inexpensive, widespread and relatively easy to work with. Also, even if not employed directly as fill they will inevitably be used in foundations of other structures because they outcrop over such large areas. Moreover large volumes of mudrocks and shales occur as waste materials from various mining and industrial operations, and these must all be tipped and stored somewhere. It is important to distinguish shales from other mudrocks and certainly to comment on laminations since this appears to have a significant effect on the likelihood of rapid disintegration and slaking on exposure to the atmosphere.

Surveys of the literature indicates that experience and past failures have lead both

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* the disagreements outlined in that chapter are academic and not really that relevant to the current dicussion

to increases in design standards and also to legislation and guidelines on design and upkeep of structures. The majority of UK dams are earth fill although the real number is unknown. Only those above a certain size require to be registered, and many tailings dams are not registered.

Data from the literature indicates that failure rates due to shearing whilst the dam is in service are low (14% in the UK), and of all failures only 25% have rendered the dam completely unusable. The chances of a shear failure completely destroying a full dam therefore appear to be small, although if this did happen it could be disastrous! With time, earth fill dams become more stable as the fill material consolidates and the water in the dam loading the toe also increases its stability. The most vulnerable periods of time are near to and at the end of construction, and during any period of rapid drawdown.

It is possible, although it seems a bit improbable, to define average mineralogy and chemistry for mudrocks and shales despite both local and regional variations. The shales used in this study were substantially Carboniferous and therefore generally similar in composition. A Kimmeridge Clay was also used as one of the standards. It was not possible to sample any other dams made of shale. Even Cod Beck Dam, which is partly founded on Lias shales, appears to be composed of glacial materials found locally. Compositions of the fresh shales that were used were within expected limits and show nothing unusual.

Weathering can be considered as the reverse process to diagenesis and leads to an increase in void ratio, moisture content and therefore change in geotechnical properties. It always acts to bring material into equilibrium with prevailing temperature and pressure conditions acting at the material surface in current climatic conditions. Obviously a rock formed under very different conditions from that in present day UK may undergo severe change when exposed; for example, problems occurred when volcanic glass in submarine basalts used as road aggregate rapidly weathered to clay (Wylde, 1976), although on the other hand diagenesis may lead to inceased stability. Weathering is a combination of two processes - physical disintegration and chemical decomposition, the various processes being outlined in Chapter 4.

Classifications are available to describe weathering effects and to assign grades of weathering, but these all refer to *in situ* rock and not to fills. Rock fills present problems because it is not possible to ascertain how much of the physical disintegration has been imposed during mechanical operations such as digging and rock moving. By the time a rock has been excavated, transported and left stockpiled it could theoretically be fairly stable by the time it is used for construction. Little evidence has been found in the literature of deterioration of earth fill in dams actually causing failure or any other major problems. Correspondingly, there is little evidence of major changes occurring on surfaces of colliery spoil heaps. This may be evidence of the relative stability of Carboniferous mudrocks and shales. The fact cannot be ignored that some shales are inherently problematic materials. The best known are those containing pyrite and soluble cements such as calcite as already mentioned. Examples of such material with respect to dam building are at Balderhead where shales containing calcite were rejected, and Winscar where shale slaked rapidly on exposure to the atmosphere, was rejected for fill and only left in the foundations if it was protected.

Shales collected from weathered profiles showed normal moisture content profiles. A difference lies in the grading as a function of time of exposure as noted at Gale Common, although as before, it is not possible to quantify how much of this is natural. A difference in grading was also noted on surfaces of spoil heaps. If it is assumed that the material came out of the ground in a uniform manner it then appears that weathering acts to further break down the shale over this time period. Dams with grass and topsoil do not show this trend, probably because the shale is reasonably protected from the effects of atmosphere by this cover.

Chemistry and mineralogy was fairly consistent in the materials examined and no trends appear to have developed with depth. Expected correlations between mineralogy and chemistry were observed, and some weathering products, for example goethite and siderite, were noted. There also appeared to have been some illite degradation with time.

Shear box testing was used in an attempt to detect any measurable differences in brittleness between the shales that were collected (all under identical conditions). Despite its many disadvantages, such as a non-uniform stress – strain regime, the calculation of only average stresses along the shear surface, the inability to measure porewater pressure, and box geometry controls on the location and direction of failure, it is easy to use, and gives a peak and residual shear strength from the same sample.

Ideally, undisturbed samples should be used, but there are problems both in taking them and in testing due to large clast size. Tests on standard materials show that there are differences in results between the large and the small shear boxes, the large size box giving a higher peak shear strength and lower residual shear strength than the small size box for the same size fraction. In the large shear box, use of an overall coarser size fraction results in higher residual shear strength (therefore lower brittleness) and also has an effect on the morphology of the shear plane produced. Kimmeridge Clay is less brittle than Coal Meaures shale - they have similar peak strengths, but the former has a higher residual shear strength.

Material from Gale Common shows the same overall trends in terms of differences between boxes and size fractions. Fresh and stockpiled samples yield similar results, and the two weathered samples show lower values of shear strength and brittleness. This correlates with changes noted in Atterberg limits and in mineralogy between the different bulk samples. Samples from the Pennine dams all have similar values, a result which is not in the least bit surprising since the mineralogy and chemistry are remarkably consistent.

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APPENDIX I

XRF Precision Tests

Test No 1 : XRF Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
A1	57.23	24.81	10.38	2.40	1.03	0.28	2.32	1.19	0.17	0.22	0.10
A2	57.12	24.80	10.39	2.38	1.03	0.27	2.32	1.19	0.17	0.22	0.10
A3	57.14	24.79	10.38	2.40	1.04	0.27	2.32	1.17	0.17	0.22	0.10
A5	57.26	24.72	10.33	2.38	1.04	0.28	2.32	1.18	0.17	0.22	0.09
A6	57.29	24.66	10.36	2.39	1.03	0.27	2.32	1.18	0.18	0.22	0.10
A7	57.12	24.79	10.34	2.41	1.04	0.27	2.32	1.18	0.17	0.23	0.09
A8	57.22	24.79	10.32	2.37	1.03	0.28	2.32	1.17	0.17	0.23	0.10
A9	57.28	24.71	10.32	2.39	1.04	0.27	2.32	1.17	0.17	0.22	0.10
A10	57.23	24.76	10.36	2.36	1.04	0.27	2.32	1.17	0.17	0.23	0.10
A11	57.26	24.70	10.32	2.41	1.04	0.27	2.33	1.18	0.17	0.22	0.09
A12	57.25	24.72	10.39	2.35	1.02	0.27	2.32	1.18	0.17	0.23	0.09

Test No 1 : XRF Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
B1	59.99	23.70	9.92	1.71	0.01	0.23	2.76	1.04	0.03	0.30	0.30
B2	60.04	23.68	9.89	1.73	0.01	0.24	2.76	1.03	0.03	0.30	0.29
B3	60.06	23.69	9.84	1.74	0.01	0.27	2.76	1.02	0.03	0.30	0.30
B4	60.04	23.70	9.88	1.73	0.01	0.24	2.76	1.02	0.03	0.31	0.29
B5	60.07	23.72	9.83	1.73	0.01	0.24	2.76	1.02	0.03	0.29	0.29
B6	60.08	23.73	9.83	1.72	0.01	0.23	2.77	1.01	0.03	0.30	0.30
B7	60.02	23.74	9.85	1.74	0.01	0.23	2.76	1.03	0.03	0.31	0.30
B8	60.12	23.69	9.83	1.73	0.01	0.25	2.76	1.01	0.03	0.30	0.03
B9	60.08	23.69	9.83	1.73	0.01	0.24	2.76	1.03	0.03	0.30	0.30
B10	60.06	23.71	9.83	1.72	0.01	0.25	2.76	1.03	0.03	0.30	0.30
B11	60.12	23.70	9.80	1.72	0.01	0.24	2.76	1.02	0.03	0.30	0.30
B12	59.97	23.78	9.90	1.71	0.01	0.24	2.75	1.02	0.03	0.29	0.29

Test No 1 : XRF Major Elements

Sample	Element Oxide (%)									
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
C1	57.71	23.54	8.89	2.41	2.97	0.31	2.55	1.11	0.11	0.20	0.20
C2	57.79	23.52	8.89	2.37	2.95	0.31	2.55	1.11	0.11	0.20	0.19
C3	57.67	23.55	8.90	2.42	2.96	0.34	2.59	1.11	0.11	0.20	0.19
C4	57.61	23.66	8.89	2.39	2.98	0.31	2.54	1.11	0.11	0.20	0.19
C5	57.74	23.58	8.85	2.39	2.95	0.32	2.55	1.12	0.11	0.20	0.19
C6	57.72	23.54	8.88	2.42	2.96	0.33	2.55	1.11	0.11	0.20	0.19
C7	57.74	23.57	8.86	2.38	2.97	0.31	2.56	1.11	0.11	0.21	0.20
C8	57.74	23.56	8.86	2.42	2.95	0.31	2.55	1.11	0.11	0.19	0.19
C9	57.68	23.63	8.85	2.40	2.98	0.32	2.55	1.11	0.11	0.20	0.19
C10	57.73	23.57	8.86	2.42	2.94	0.32	2.55	1.11	0.11	0.21	0.19
C11	57.75	23.60	8.83	2.39	2.98	0.31	2.52	1.10	0.11	0.20	0.20
C12	57.64	23.65	8.91	2.39	2.94	0.30	2.55	1.11	0.11	0.20	0.19

Test No 1 : XRF Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
D1	61.68	22.82	8.43	2.27	0.39	0.32	2.67	1.04	0.10	0.22	0.05
D2	61.77	22.79	8.41	2.25	0.39	0.32	2.66	1.03	0.11	0.22	0.05
D3	61.76	22.81	8.40	2.25	0.39	0.31	2.67	1.04	0.11	0.22	0.05
D4	61.73	22.82	8.39	2.27	0.39	0.33	2.66	1.04	0.11	0.22	0.05
D5	61.79	22.76	8.41	2.26	0.39	0.31	2.67	1.03	0.11	0.22	0.05
D6	61.73	22.89	8.36	2.24	0.39	0.32	2.67	1.03	0.11	0.23	0.05
D7	61.74	22.80	8.40	2.27	0.31	0.31	2.67	1.03	0.11	0.22	0.05
D8	61.82	22.82	8.35	2.26	0.39	0.32	2.66	1.02	0.10	0.21	0.05
D9	61.82	22.79	8.37	2.25	0.39	0.32	2.66	1.03	0.11	0.22	0.05
D10	61.76	22.80	8.38	2.29	0.39	0.32	2.66	1.02	0.11	0.22	0.05
D11	61.87	22.78	8.34	2.25	0.39	0.31	2.66	1.04	0.11	0.23	0.05
D12	61.73	22.83	8.42	2.26	0.39	0.32	2.67	1.03	0.11	0.21	0.05

APPENDIX II

XRF Results

Balderhead Dam : XRF Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
BH1	49.50	20.53	5.98	1.52	3.49	0.22	2.29	0.97	0.05	0.13	14.87
BH2	47.33	20.21	6.40	1.55	3.09	0.22	2.18	0.97	0.06	0.14	17.38
BH3	49.50	20.80	6.12	1.54	2.68	0.23	2.21	0.98	0.05	0.14	14.83
BH4	48.35	20.71	6.66	1.56	2.88	0.23	2.17	0.95	0.05	0.12	15.85
BH5	51.19	20.51	6.05	1.52	2.35	0.23	2.25	0.93	0.05	0.13	14.33
BH6	50.28	21.04	6.22	1.53	2.57	0.24	2.23	0.96	0.05	0.13	14.32
BH7	49.88	20.98	6.20	1.59	3.02	0.22	2.30	0.98	0.05	0.13	14.23
BH8	51.11	20.50	5.56	1.57	3.58	0.23	2.39	0.94	0.04	0.11	13.58
BH9	50.39	20.20	5.87	1.54	3.73	0.24	2.35	0.94	0.05	0.11	14.18
BH10	50.75	19.82	5.86	1.53	3.15	0.24	2.35	0.91	0.04	0.12	14.82

Courtyard Dam : XRF Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
CD1	45.26	20.76	3.27	0.95	0.28	0.14	2.54	1.01	0.02	0.06	25.46
CD2	38.47	17.60	3.40	0.82	0.33	0.11	2.24	0.99	0.03	0.08	35.48
CD3	36.91	17.22	3.52	0.80	0.44	0.12	2.26	0.93	0.03	0.07	37.11
CD4	39.08	17.56	3.41	0.81	0.55	0.14	2.31	0.95	0.01	0.06	34.62
CD5	35.53	16.27	3.50	0.74	0.41	0.12	2.14	0.94	0.03	0.07	39.54
CD6	39.12	19.31	3.34	0.84	0.45	0.13	2.22	0.97	0.02	0.08	33.14
CD7	39.32	19.21	3.54	0.81	0.47	0.12	2.27	0.97	0.02	0.07	32.69
CD8	44.95	21.59	4.40	0.93	0.44	0.15	2.61	1.11	0.03	0.07	23.14
CD9	44.78	21.75	4.59	0.88	0.45	0.15	2.75	1.12	0.04	0.08	22.66

Gale Common (general) : XRF Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
GCF gen	40.34	18.12	4.67	1.06	0.62	0.43	2.84	0.80	0.02	0.07	19.19
GCS gen	40.33	18.39	3.99	1.03	0.43	0.49	2.78	0.80	0.03	0.08	30.63
GCA gen	32.93	14.50	5.37	0.87	0.27	0.30	2.74	0.72	0.02	0.07	39.84
GCB gen	35.65	15.24	5.84	0.90	0.73	0.33	2.77	0.74	0.02	0.09	35.80
BR1	47.74	19.60	4.32	1.46	1.15	0.56	3.34	1.05	0.02	0.11	18.92
SR1	57.62	21.78	4.65	1.73	0.29	0.75	3.64	1.07	0.04	0.132	8.29
SR2	59.46	20.39	5.03	1.53	0.30	0.84	3.34	1.05	0.07	0.150	7.81

Gale Common (15 year old fill) : XRF Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
GCL1	42.71	18.05	6.85	1.11	0.14	0.39	2.87	0.85	0.03	0.08	25.52
GCL2	43.42	18.16	7.12	1.18	0.35	0.31	2.98	0.83	0.04	0.09	24.27
GCL3	43.31	18.12	6.40	1.11	0.38	0.32	2.92	0.84	0.03	0.10	25.18
GCL4	47.41	19.99	6.79	1.24	0.12	0.26	3.20	0.92	0.04	0.12	18.93
GCL5	47.07	19.62	6.38	1.23	0.21	0.25	2.99	0.86	0.04	0.10	20.62
GCL6	43.32	19.46	5.39	1.14	0.20	0.25	3.11	0.85	0.03	0.10	25.37
GCL7	44.18	19.56	5.56	1.06	0.18	0.22	2.90	0.87	0.03	0.11	24.54
GCL8	45.92	18.12	5.16	1.19	0.26	0.24	2.92	0.85	0.03	0.09	24.55
GCL9	42.83	18.76	6.38	1.10	0.08	0.29	2.96	0.84	0.03	0.11	25.61
GCL10	44.76	18.98	6.26	1.20	0.16	0.27	3.02	0.87	0.03	0.10	23.41
GCL11	43.51	18.37	5.23	1.16	0.35	0.23	2.93	0.87	0.03	0.08	26.03
GCL12	42.71	19.06	4.57	1.17	0.27	0.24	3.02	0.82	0.03	0.09	27.29
GCL13	41.33	18.74	5.02	1.08	0.42	0.23	2.93	0.80	0.02	0.08	28.22
GCL14	43.29	19.31	5.38	1.16	0.38	0.24	3.14	0.86	0.03	0.09	24.92

Grassholme Dam : XRF Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
GH1	49.85	21.12	10.23	1.29	0.21	0.16	2.30	0.87	0.08	0.21	13.61
GH2	50.33	22.36	8.37	1.33	0.25	0.15	2.35	0.94	0.13	0.23	13.53
GH3	55.37	18.94	9.41	1.09	0.23	0.26	0.20	0.84	0.18	0.20	11.46
GH4	50.89	22.41	7.90	1.32	0.18	0.15	2.38	0.94	0.07	0.23	13.50
GH5	52.27	20.63	9.10	1.27	0.23	0.19	2.40	0.87	0.14	0.28	12.59
GH6	54.81	20.45	7.16	1.32	0.05	0.19	2.85	0.85	0.04	0.19	11.94

Grassholme - Hury Spoil Heap : XRF Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
1-1	60.0813	20.4397	7.3665	2.4076	5.0093	0.3539	2.7398	0.9861	0.0734	0.1799	0.3625
1-2	59.2458	20.6705	7.4376	2.2655	5.6521	0.2942	2.8469	0.9355	0.0748	0.15550	0.4220
1-3	58.5712	20.8341	7.8173	2.2851	5.7282	0.3188	2.8129	0.9369	0.0767	0.1734	0.4456
1-4	61.7629	22.6069	9.1112	1.7120	0.1472	0.2781	2.7797	0.9951	0.0355	0.3106	0.2608
1-5	59.0403	20.7686	7.5049	2.2727	5.6013	0.3228	2.8567	0.9334	0.0724	0.1735	0.4533
1-6	60.0666	20.1037	7.2663	2.3063	5.2606	0.3313	2.9101	0.9157	0.0597	0.1628	0.6170
1-7	60.2172	20.8752	7.6492	2.2202	4.2453	0.3152	2.8047	0.9798	0.0723	0.1701	0.4508
1-8	59.5299	21.9264	8.1541	2.3778	3.5265	0.3280	2.5381	1.0529	0.0926	0.1750	0.2985
1-9	59.8966	21.0415	7.3082	2.4017	4.7168	0.3301	2.7170	0.9759	0.0778	0.1506	0.3839
1-10	59.3250	20.2932	7.4891	2.4319	5.6250	0.3190	2.8058	0.9350	0.0752	0.1556	0.5453
1-11	59.8242	20.9983	7.4241	2.4299	4.7229	0.3549	2.5802	0.9317	0.0802	0.1851	0.4685
1-12	57.8199	23.4522	8.8463	2.4275	2.9691	0.3294	2.5602	1.1016	0.1098	0.1896	0.1944

Grassholme - Hury Spoil Heap : XRF Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S+H_2O+C$
1NG1	60.3885	25.9804	7.1802	1.9543	0.2069	0.2739	2.4081	1.2972	0.0265	0.2062	0.0779
1NG2	60.6819	23.1056	9.6434	1.7201	0.0861	0.2617	2.7839	1.0087	0.0341	0.3017	0.3729
1NG3	60.1603	23.6100	9.8346	1.7396	0.0000	0.2323	2.7704	1.0235	0.0342	0.2990	0.2961
2-1	61.7042	22.8694	8.3821	2.2569	0.3868	0.3197	2.6661	1.0302	0.1063	0.2295	0.0488
2-2	57.3229	25.6051	9.7745	2.4086	0.6080	0.2538	2.4058	1.1888	0.1379	0.2179	0.0766
2-3	59.4373	23.7755	8.8980	2.2842	0.6530	0.2392	3.2545	1.0355	0.0975	0.1892	0.1361
2-4	58.0070	24.4621	9.2669	2.4508	1.3867	0.3018	2.5135	1.1941	0.1356	0.2069	0.0745
2-5	57.0927	24.4884	10.6204	2.5210	1.0730	0.2648	2.2669	1.1748	0.1748	0.2230	0.1002

Gale Common - Profile A : Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
GCA1	45.13	19.61	5.20	1.20	0.0	0.36	3.26	0.80	0.03	0.08	24.33
GCA2	43.24	19.19	4.82	1.23	0.18	0.33	3.30	0.80	0.04	0.06	26.82
GCA3	42.73	18.80	5.08	1.22	0.26	0.32	3.32	0.82	0.03	0.06	27.36
GCA4	40.51	18.83	5.42	1.17	0.45	0.30	3.10	0.78	0.04	0.06	29.34
GCA5	40.15	18.29	4.63	1.15	0.31	0.30	3.14	0.79	0.04	0.05	31.17
GCA6	39.96	19.55	4.27	1.15	0.24	0.27	3.40	0.77	0.03	0.06	30.30
GCA7	45.59	19.10	4.88	1.30	0.29	0.33	3.27	0.86	0.04	0.06	24.28
GCA8	43.99	18.40	4.96	1.18	0.19	0.33	2.96	0.87	0.04	0.07	27.03
GCA10	46.53	18.87	6.70	1.49	0.90	0.51	3.33	0.88	0.05	0.11	20.64
GCA11	37.74	17.70	5.49	1.12	0.13	0.35	3.10	0.74	0.03	0.07	33.52
GCA12	29.76	13.94	3.86	0.94	0.36	0.26	2.57	0.62	0.04	0.05	47.60
GCA13	48.63	17.80	6.42	1.18	0.40	0.32	2.91	0.69	0.05	0.08	31.50
GCA14	44.99	20.71	4.78	1.45	0.39	0.36	3.66	0.93	0.06	0.06	22.61
GCA16	37.90	17.03	7.85	1.20	0.68	0.36	2.99	0.76	0.04	0.09	31.10
GCA18	48.15	21.30	4.16	1.47	0.45	0.45	3.50	0.93	0.06	0.08	19.46
GCA19	49.58	21.21	4.05	1.48	0.35	0.50	3.50	0.93	0.05	0.08	18.26
GCA20	45.15	20.31	4.89	1.39	0.59	0.47	3.59	0.93	0.06	0.07	22.51

Gale Common - Profile B : Major Elements

Sample	Element Oxide (%)										
	SiO_2	Al_2O_3	Fe_3O_4	MgO	CaO	Na_2O	K_2O	TiO_2	MnO	PO_3	$S + H_2O + C$
GCB1	45.61	20.07	5.17	1.28	0.05	0.29	3.40	0.84	0.05	0.07	23.17
GCB2	40.58	18.51	4.64	1.05	0.08	0.35	3.25	0.82	0.03	0.06	30.64
GCB3	33.33	15.14	5.19	0.83	0.04	0.35	2.95	0.77	0.02	0.05	41.33
GCB4	35.25	16.35	5.61	0.90	0.21	0.41	2.88	0.73	0.03	0.06	37.58
GCB5	40.05	16.84	5.72	0.97	0.10	0.50	2.93	0.82	0.03	0.08	31.96
GCB6	36.59	16.38	5.35	0.93	0.19	0.40	2.83	0.76	0.03	0.07	36.47
GCB7	32.90	14.95	6.21	0.84	0.33	0.40	2.95	0.78	0.02	0.06	40.57
GCB8	37.58	17.70	6.00	0.99	0.33	0.43	3.30	0.81	0.03	0.08	32.75

Balderhead Dam - XRF Trace Element Analyses

Sample	Elemental Concentration (ppm)																	
	Ba	Nb	Zr	Y	Sr	Rb	Zn	Cu	Ni	Pb	U	Th	V	Cr	Nd	Ga	La	Ce
BH1	299.	18.	133.	26.	170.	120.	62.	28.	60.	29.	3.	23.	94.	86.	45.	28.	58.	112.
BH2	300.	16.	132.	24.	160.	117.	63.	30.	58.	28.	2.	21.	95.	88.	54.	29.	61.	127.
BH3	319.	16.	139.	24.	147.	118.	69.	25.	59.	30.	4.	23.	102.	89.	50.	27.	52.	122.
BH4	282.	16.	119.	25.	146.	115.	65.	25.	60.	25.	4.	21.	105.	91.	48.	28.	53.	109.
BH5	262.	17.	123.	25.	140.	119.	72.	25.	59.	30.	3.	21.	98.	87.	54.	27.	51.	121.
BH6	337.	18.	130.	26.	145.	119.	58.	31.	58.	31.	3.	25.	101.	89.	42.	28.	52.	113.
BH7	297.	18.	130.	24.	154.	121.	62.	27.	63.	29.	3.	24.	96.	91.	59.	30.	55.	122.
BH8	328.	17.	127.	25.	166.	126.	53.	30.	58.	29.	3.	23.	95.	88.	55.	27.	55.	118.
BH9	315.	18.	129.	24.	165.	123.	64.	30.	61.	26.	3.	24.	92.	87.	43.	24.	53.	114.
BH10	310.	17.	117.	25.	154.	123.	61.	28.	56.	26.	4.	21.	96.	85.	45.	26.	56.	109.

Grassholme Dam - XRF Trace Element Analyses

Sample	Elemental Concentration (ppm)																	
	Ba	Nb	Zr	Y	Sr	Rb	Zn	Cu	Ni	Pb	U	Th	V	Cr	Nd	Ga	La	Ce
GH1	441.	16.	139.	24.	125.	120.	46.	22.	47.	38.	2.	22.	106.	92.	55.	27.	59.	125.
GH2	529.	18.	144.	36.	153.	131.	68.	32.	61.	37.	3.	26.	0.	02.	70.	32.	64.	133.
GH3	624.	16.	201.	26.	140.	93.	66.	20.	33.	43.	3.	20.	76.	75.	39.	18.	43.	92.
GH4	463.	19.	145.	33.	146.	132.	42.	24.	57.	35.	3.	23.	113.	97.	62.	32.	59.	139.
GH5	614.	17.	147.	34.	135.	133.	53.	28.	46.	39.	2.	24.	99.	88.	56.	25.	61.	128.
GH6	493.	20.	114.	21.	120.	160.	30.	25.	43.	33.	3.	25.	91.	87.	45.	29.	49.	114.

Courtyard Dam - XRF Trace Element Analyses

Sample	Elemental Concentration (ppm)																	
	Ba	Nb	Zr	Y	Sr	Rb	Zn	Cu	Ni	Pb	U	Th	V	Cr	Nd	Ga	La	Ce
CD1	527.	16.	120.	28.	93.	121.	64.	48.	59.	38.	4.	27.	118.	93.	48.	26.	48.	98.
CD2	465.	17.	123.	28.	166.	103.	79.	66.	57.	48.	3.	24.	116.	80.	47.	25.	55.	124.
CD3	454.	15.	103.	24.	92.	101.	93.	56.	63.	49.	4.	26.	116.	79.	36.	27.	44.	90.
CD4	577.	17.	115.	24.	108.	104.	82.	54.	57.	50.	4.	27.	122.	79.	38.	25.	46.	103.
CD5	459.	15.	97.	23.	90.	96.	78.	56.	65.	49.	3.	27.	119.	76.	37.	26.	42.	81.
CD6	399.	17.	99.	23.	99.	104.	79.	55.	65.	49.	3.	28.	119.	88.	36.	29.	47.	96.
CD7	575.	16.	112.	25.	96.	103.	79.	54.	46.	47.	3.	27.	119.	89.	46.	30.	45.	98.
CD8	507.	18.	129.	30.	108.	123.	126.	65.	78.	59.	5.	31.	145.	98.	41.	35.	59.	113.
CD9	617.	19.	127.	30.	104.	128.	129.	62.	43.	57.	4.	30.	140.	99.	50.	33.	58.	110.

Gale Common (15 yr fill) XRF Trace Element Analyses

Sample	Elemental Concentration (ppm)																
	Ba	Nb	Zr	Y	Sr	Rb	Zn	Cu	Ni	Pb	U	Th	V	Cr	Nd	Ga	La	Ce
GCF gen	341.	12.	109.	29.	146.	121.	43.	41.	41.	28.	3.	20.	104.	75.	42.	25.	47.	89.
GCS gen	323.	13.	118.	30.	152.	124.	44.	45.	46.	28.	4.	23.	100.	73.	40.	24.	45.	90.
GCA gen	282.	12.	100.	25.	125.	122.	25.	43.	26.	33.	3.	21.	89.	58.	33.	25.	42.	81.
GCB gen	438.	13.	116.	24.	134.	119.	158.	65.	27.	54.	2.	21.	100.	64.	39.	22.	39.	81.
BR1	546.	18.	126.	34.	176.	159.	93.	43.	62.	27.	3.	24.	124.	88.	52.	28.	52.	96.
SR1	618.	18.	140.	38.	173.	183.	146.	37.	57.	30.	3.	24.	128.	100.	43.	32.	57.	103.
SR2	528.	19.	156.	39.	191.	156.	137.	32.	51.	25.	2.	15.	111.	83.	0.	23.	52.	199.

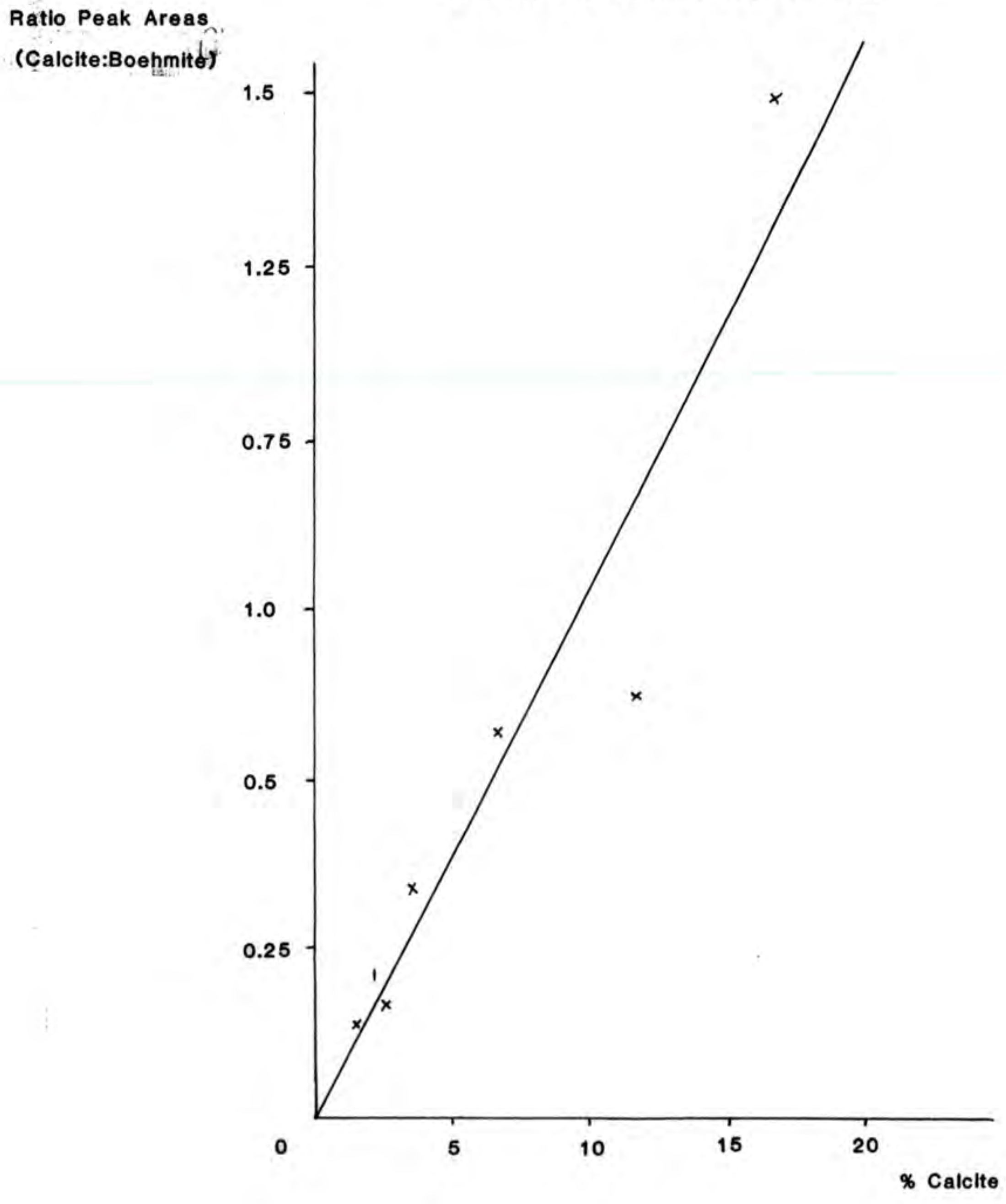
Gale Common (general) XRF Trace Element Analyses

Sample	Elemental Concentration (ppm)																
	Ba	Nb	Zr	Y	Sr	Rb	Zn	Cu	Ni	Pb	U	Th	V	Cr	Nd	Ga	La	Ce
GCL1	491.	12.	115.	28.	159.	123.	7.	26.	26.	35.	2.	13.	92.	69.	31.	25.	44.	90.
GCL2	466.	15.	126.	28.	154.	136.	12.	34.	34.	32.	3.	21.	102.	75.	43.	24.	44.	90.
GCL3	531.	14.	123.	28.	155.	132.	20.	35.	24.	34.	3.	23.	100.	74.	39.	28.	48.	97.
GCL4	560.	14.	130.	33.	153.	147.	41.	52.	45.	34.	3.	22.	112.	77.	42.	26.	54.	85.
GCL5	566.	15.	129.	28.	165.	138.	27.	38.	34.	29.	4.	20.	99.	83.	49.	24.	56.	110.
GCL6	540.	15.	122.	31.	157.	145.	27.	53.	38.	32.	3.	24.	114.	78.	46.	25.	49.	118.
GCL7	499.	15.	119.	34.	156.	140.	44.	51.	34.	31.	4.	24.	112.	76.	41.	30.	55.	96.
GCL8	538.	15.	127.	30.	140.	133.	27.	33.	32.	28.	3.	23.	101.	75.	47.	26.	46.	92.
GCL9	519.	16.	131.	30.	165.	138.	18.	47.	40.	33.	3.	25.	115.	76.	48.	27.	46.	100.
GCL10	531.	15.	127.	31.	143.	135.	27.	42.	33.	32.	3.	21.	112.	75.	43.	26.	50.	113.
GCL11	447.	15.	131.	30.	128.	130.	39.	43.	35.	33.	2.	22.	108.	71.	44.	24.	41.	106.
GCL12	517.	15.	120.	30.	153.	140.	42.	49.	41.	32.	3.	27.	108.	76.	43.	28.	49.	98.
GCL13	463.	14.	114.	30.	145.	130.	37.	48.	36.	33.	3.	22.	107.	72.	44.	25.	53.	99.
GCL14	475.	15.	120.	30.	145.	142.	41.	51.	47.	36.	4.	26.	122.	79.	49.	26.	54.	100.

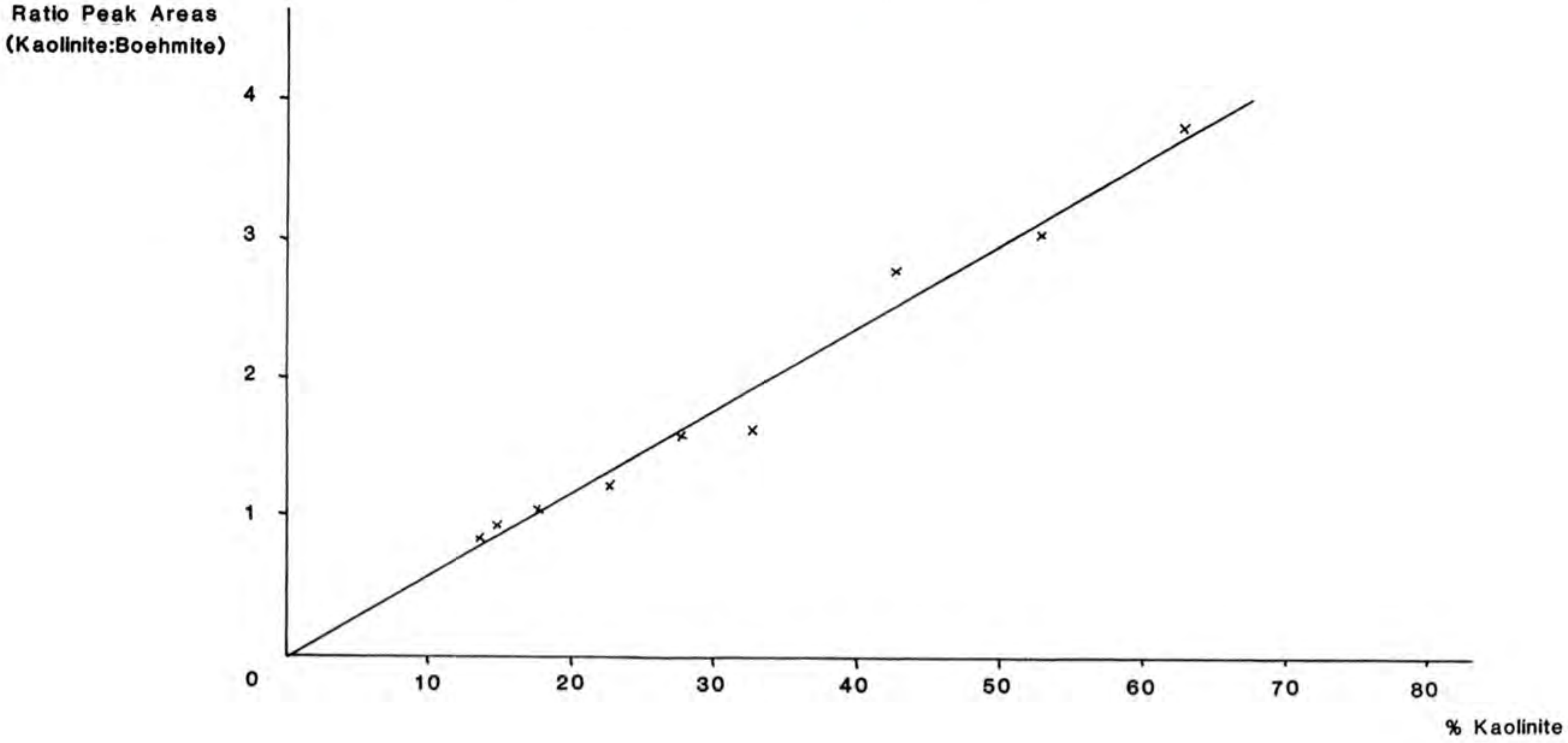
APPENDIX III

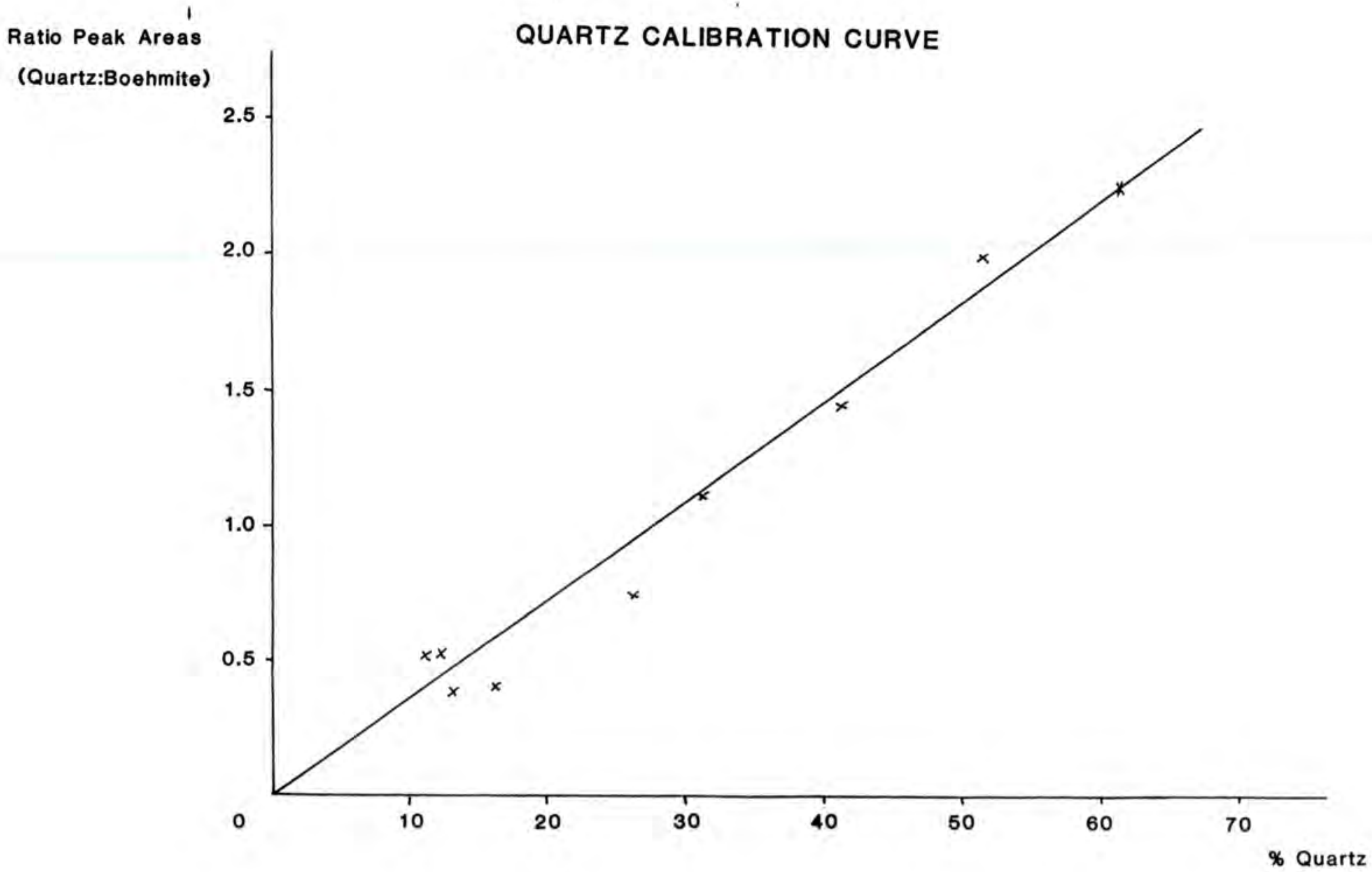
XRD Calibration Charts

CALCITE CALIBRATION CURVE



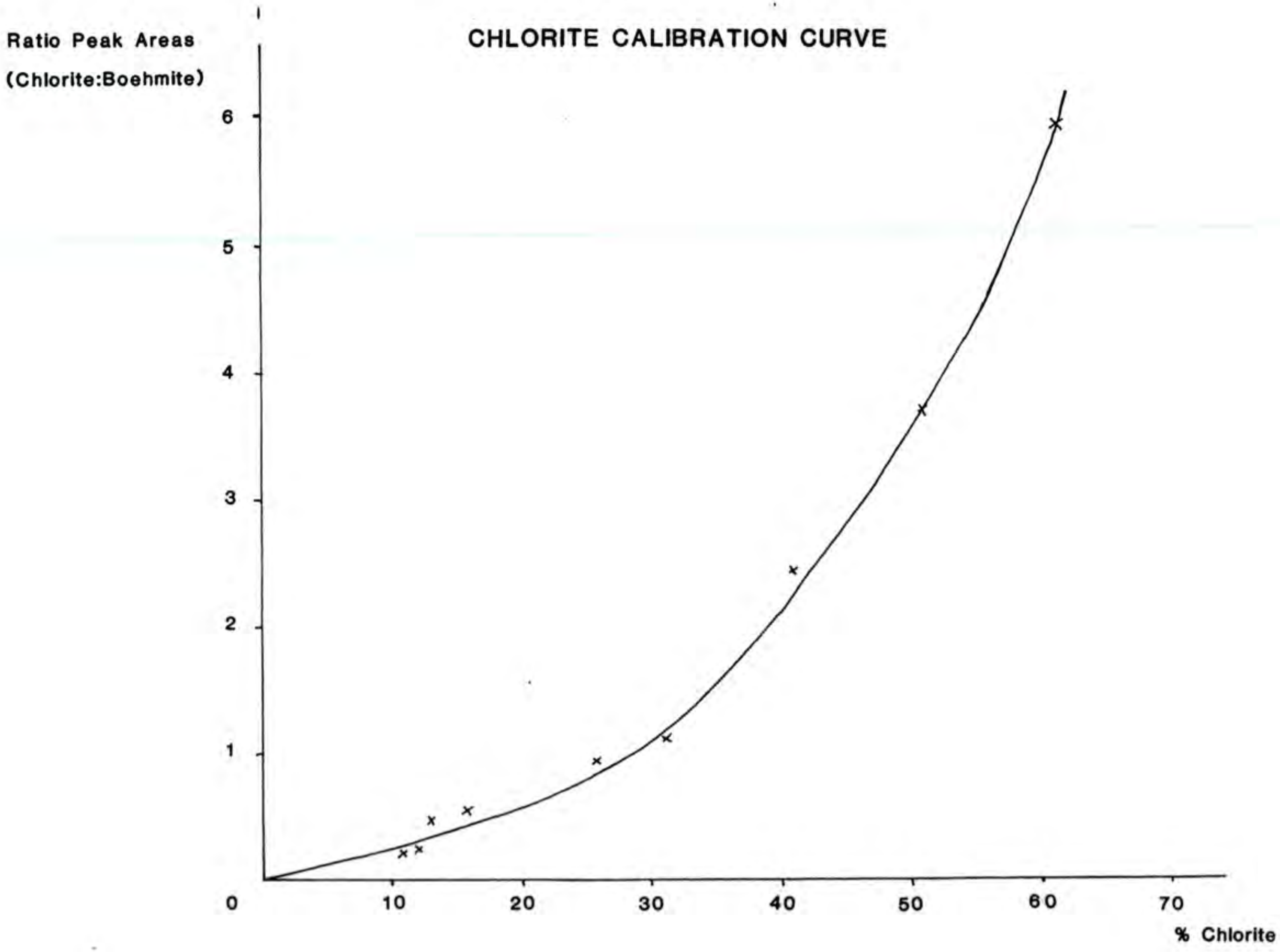
KAOLINITE CALIBRATION CURVE





QUARTZ CALIBRATION CURVE

CHLORITE CALIBRATION CURVE



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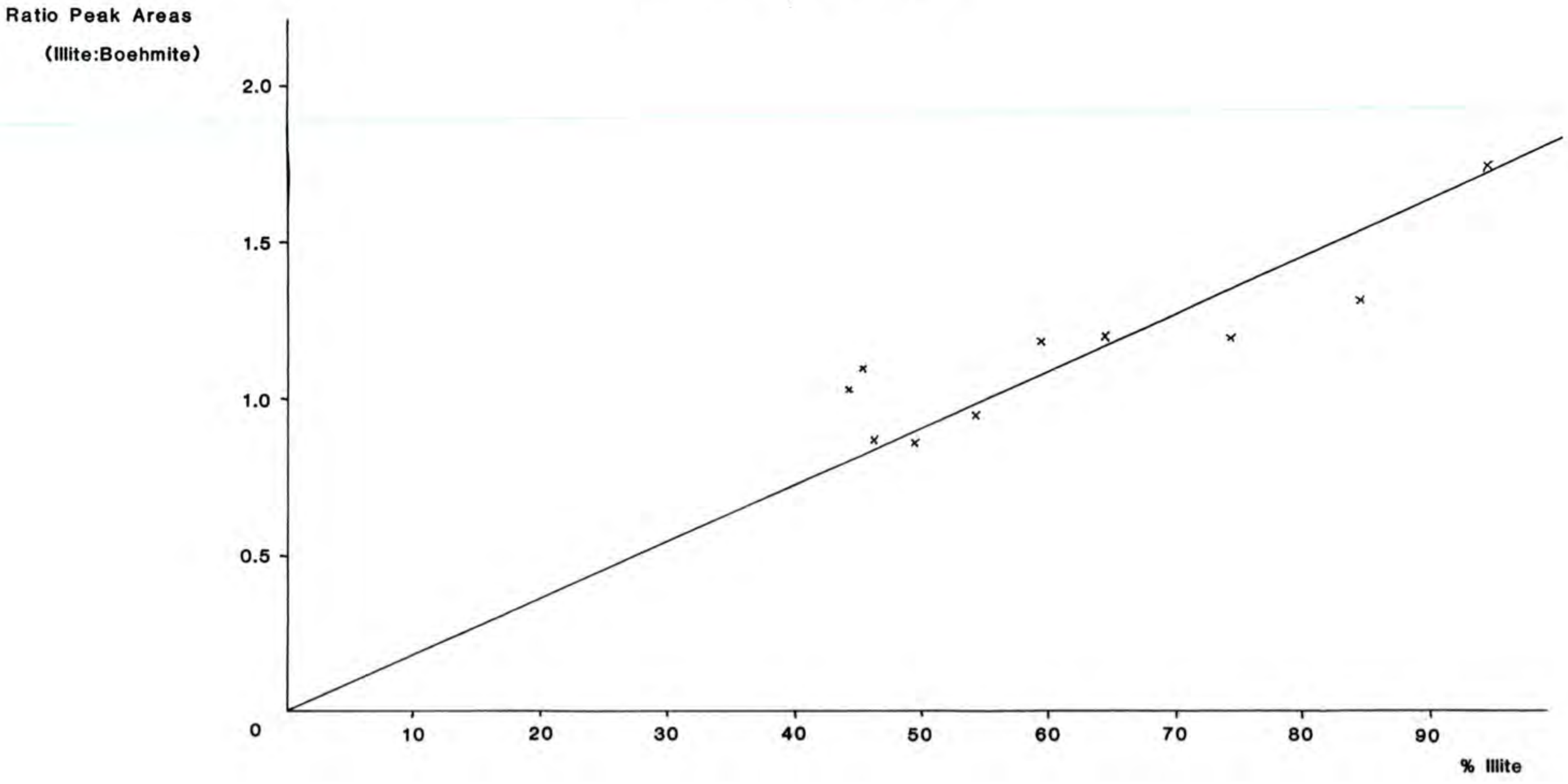

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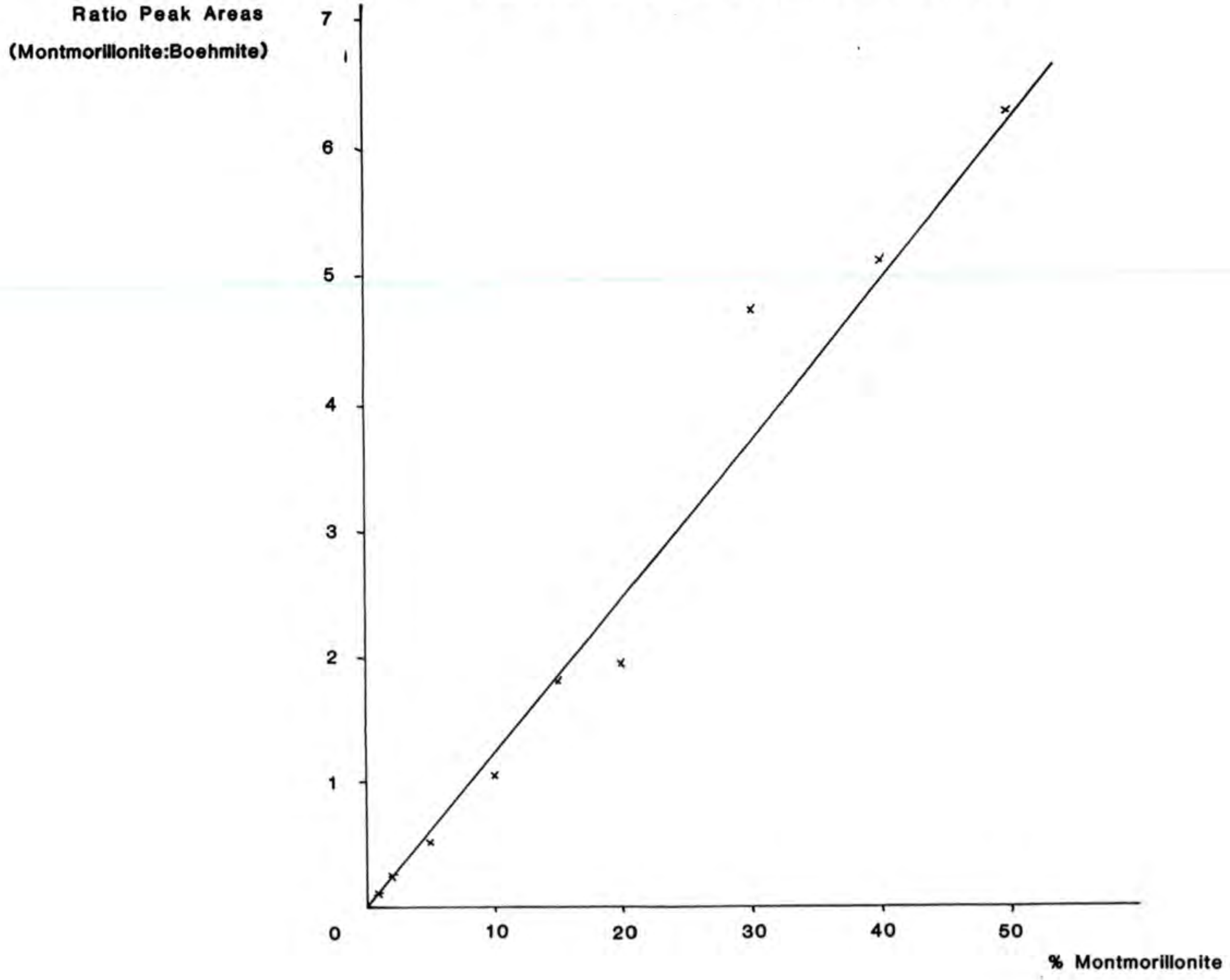
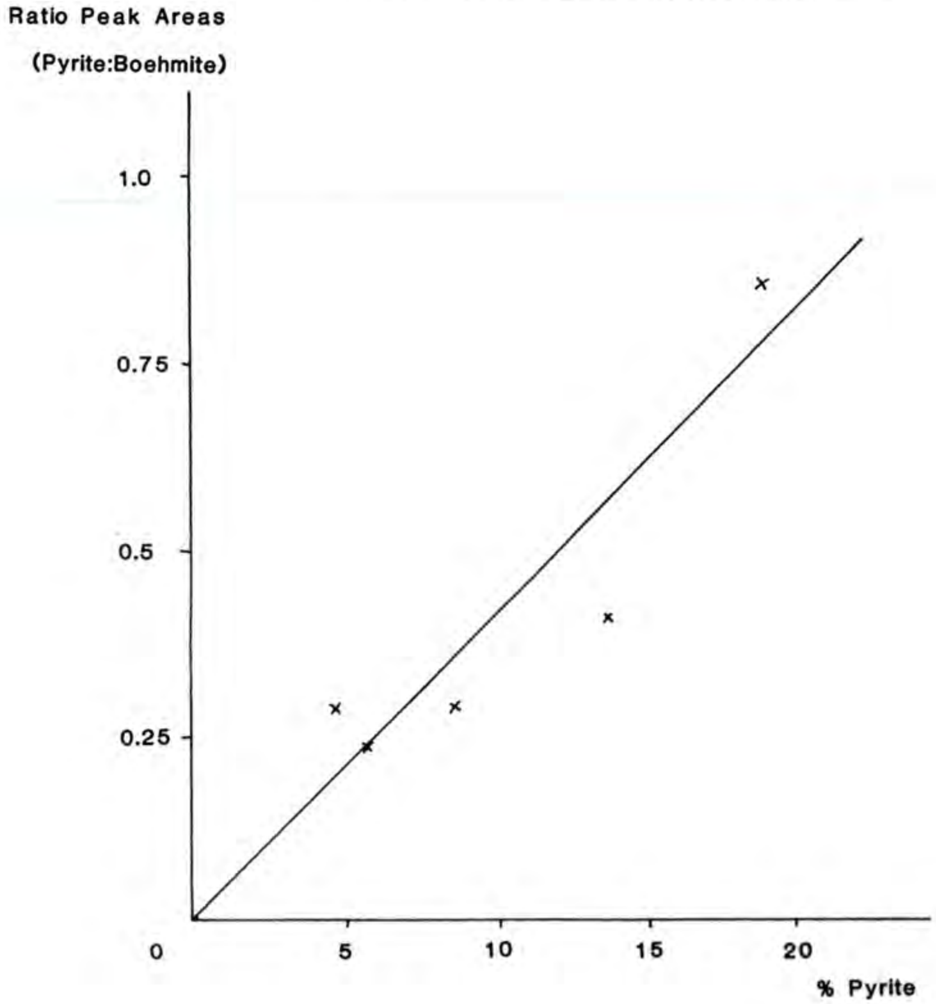
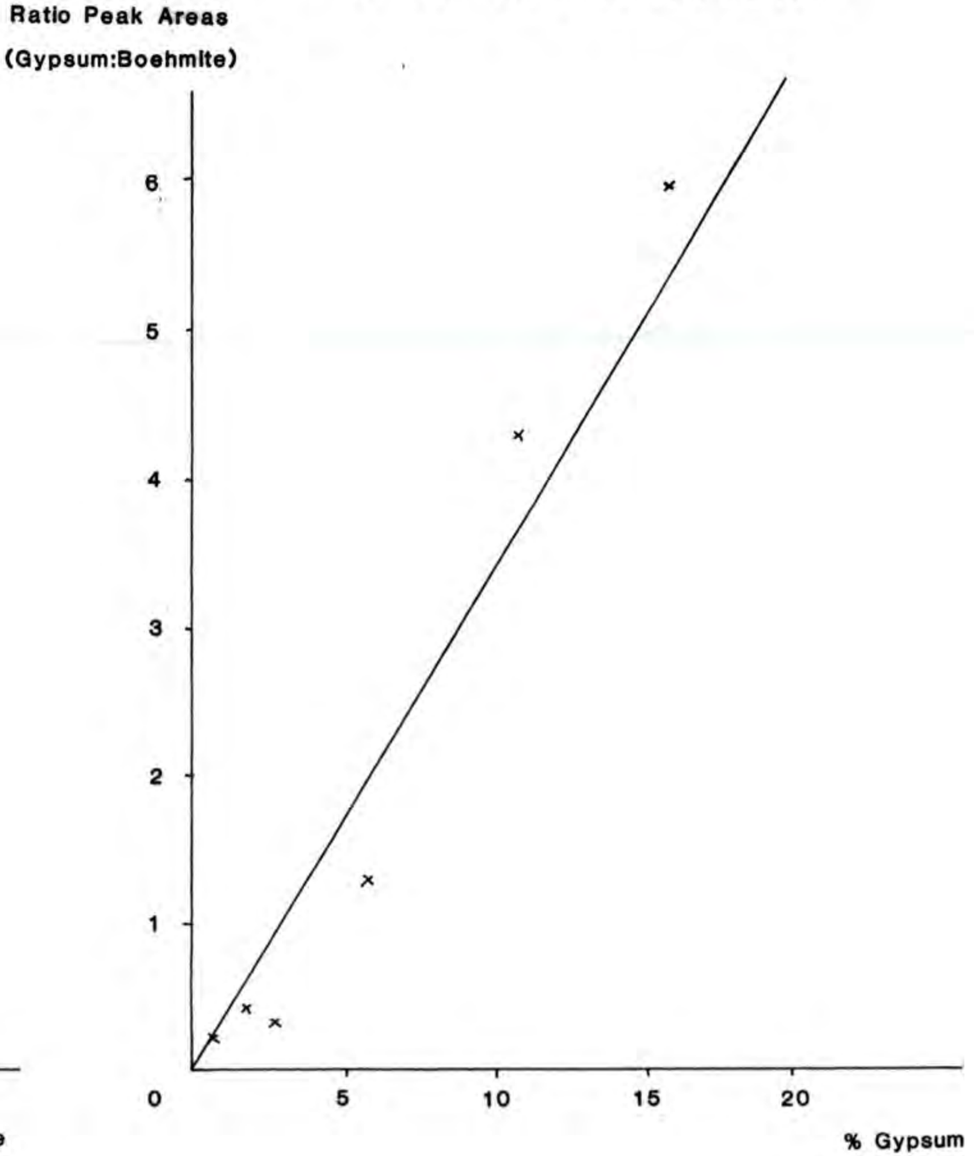


PYRITE CALIBRATION CURVE



GYPSUM CALIBRATION CURVE



APPENDIX IV

XRD Results

Gale Common general: XRD Results

Sample	Mineral (%)								
	Mt	Chl	Ill	Gyp	Kaol	Qtz	Calc	Pyr	Misc
fresh (bulk)	-	0.2	37.0	0.3	14.5	18.5	-	5.5	22.0
stockpile	-	0.2	37.8	0.2	14.4	14.8	-	1.5	29.1
5 year weathered	-	0.9	47.6	0.3	11.0	16.2	-	-	16
15 year weathered	-	0.2	44.4	0.4	10.2	12.5	-	-	30.3
Silkstone	-	4	56.9	tr	19.9	22.2	-	-	0
Beeston	-	4	53.8	tr	18.1	23.3	-	-	0.4

Gale Common gravel sized fragments: XRD Results

Sample	Mineral (%)								
	Mt	Chl	Ill	Gyp	Kaol	Qtz	Calc	Pyr	Misc
fresh	-	3.0	50.3	tr	12.6	27.2	-	4.4	2.5
stockpile	-	8.0	43.7	tr	15.5	39.1	-	tr	0.9
5 year	-	0.5	53.5	15.5	25.9	1.9	3.0	-	-
15 year	-	?	24.0	-	7.3	8.3	0.7	1.4	58.3

Gale Common 15 year weathered profile : XRD Results

Sample	Mineral (%)								
	Mt	Chl	Ill	Gyp	Kaol	Qtz	Calc	Pyr	Misc
GCL1	-	0.3	35.8	-	12.6	32.3	3.5	tr	15.5
GCL2	-	0.5	30.9	0.1	10.9	24.7	2.4	1.0	29.5
GCL3	-	0.2	19.8	0.3	8.7	21.6	-	0.5	48.9
GCL4	-	0.3	14.9	0.3	15.3	23.7	-	-	45.5
GCL5	-	0.4	36.6	0.2	17.4	27.8	0.5	2.5	14.6
GCL6	-	0.3	31.1	0.1	10.5	15.0	-	tr	43.0
GCL7	-	0.4	38.3	-	17.5	22.0	1.2	2.5	18.1
GCL8	-	0.2	64.7	0.4	13.9	19.6	-	1.5	-
GCL9	-	0.4	19.5	tr	8.7	19.5	-	1.3	50.6
GCL10	-	0.4	21.2	0.2	9.3	20.5	-	1.7	46.7
GCL11	-	0.2	27.8	0.4	8.9	24.9	-	1.3	36.5
GCL12	-	0.4	38.0	tr	18.3	18.3	1.4	tr	23.6
GCL13	-	0.3	26.1	0.5	9.0	13.1	1.1	2.1	47.8
GCL14	-	0.3	17.2	0.3	4.8	19.8	1.3	1.1	55.2

Balderhead Dam : XRD Results

Sample	Mineral (%)								
	Mt	Chl	Ill	Gyp	Kaol	Qtz	Calc	Pyr	Misc
BH1	-	-	10.4	tr	24.0	27.2	8.7	-	29.7
BH2	-	0.5	15.4	-	25.1	22.6	3.6	2.7	30.1
BH3	-	0.3	9.5	tr	23.2	22.2	5.1	2.7	36.9
BH4	-	0.4	12.1	-	22.8	19.9	4.7	2.1	38.0
BH5	-	0.5	25.9	tr	24.0	26.3	4.0	1.2	18.1
BH6	-	0.3	9.9	tr	20.0	32.7	6.4	2.6	28.1
BH7	-	0.3	16.0	tr	22.8	27.2	8.9	1.4	23.4
BH8	-	0.4	38.8	tr	20.9	26.5	4.9	1.7	6.8
BH9	-	0.9	25.8	tr	16.3	22.8	7.3	2.1	24.8

Grassholme Dam : XRD Results

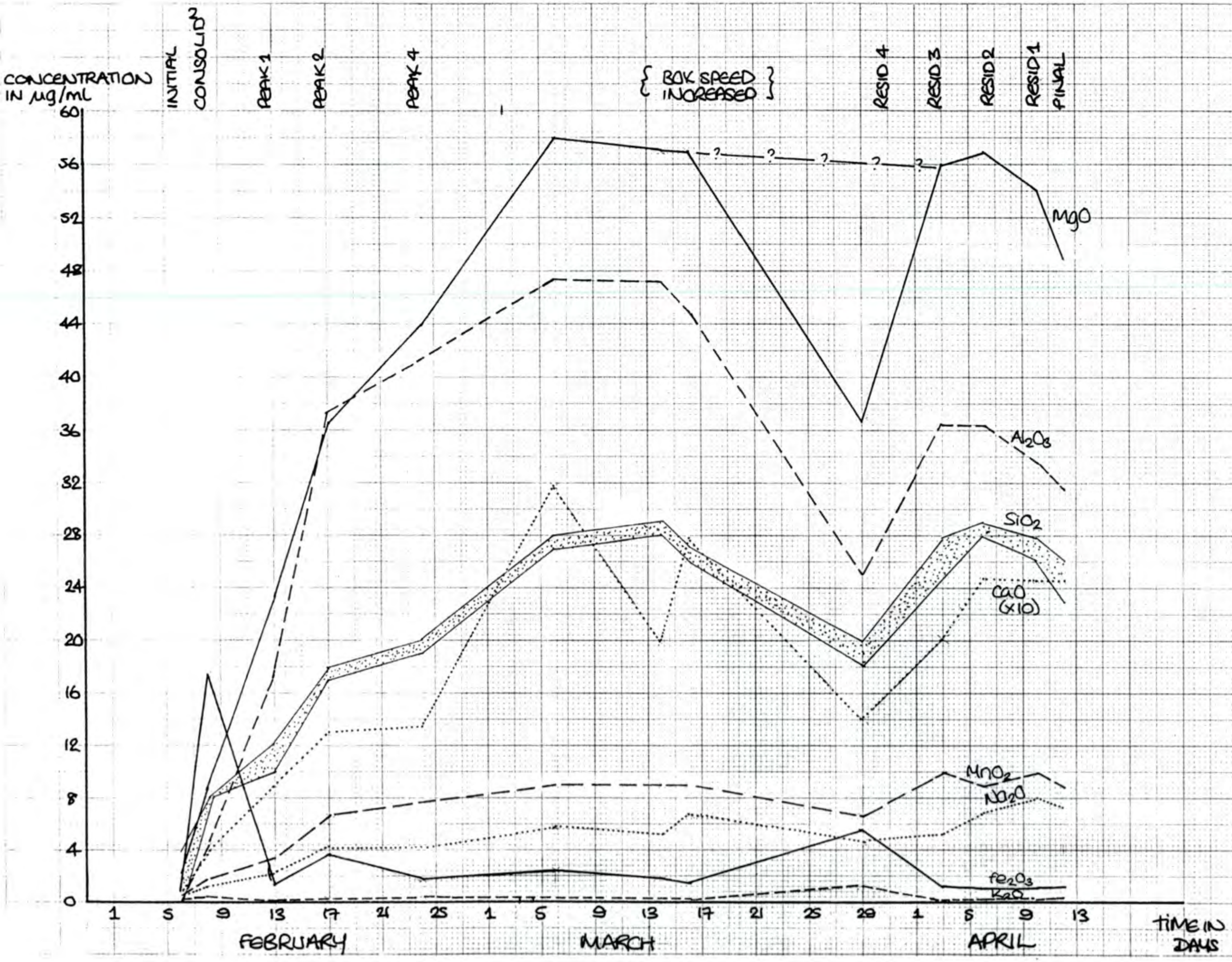
Sample	Mineral (%)								
	Mt	Chl	Ill	Gyp	Kaol	Qtz	Calc	Pyr	Misc
GH1	-	0.9	14.7	-	14.0	29.4	-	-	41.0
GH2	-	0.9	22.7	tr	16.2	25.9	-	-	34.3
GH3	-	0.8	13.2	-	9.7	33.0	-	-	43.3
GH4	-	0.8	4.8	-	17.7	25.4	tr	-	51.3
GH5	-	0.3	11.1	-	15.3	25.0	-	-	48.3
GH6	-	0.7	29.5	tr	13.2	26.9	-	tr	29.7

Courtyard model Dam : XRD Results

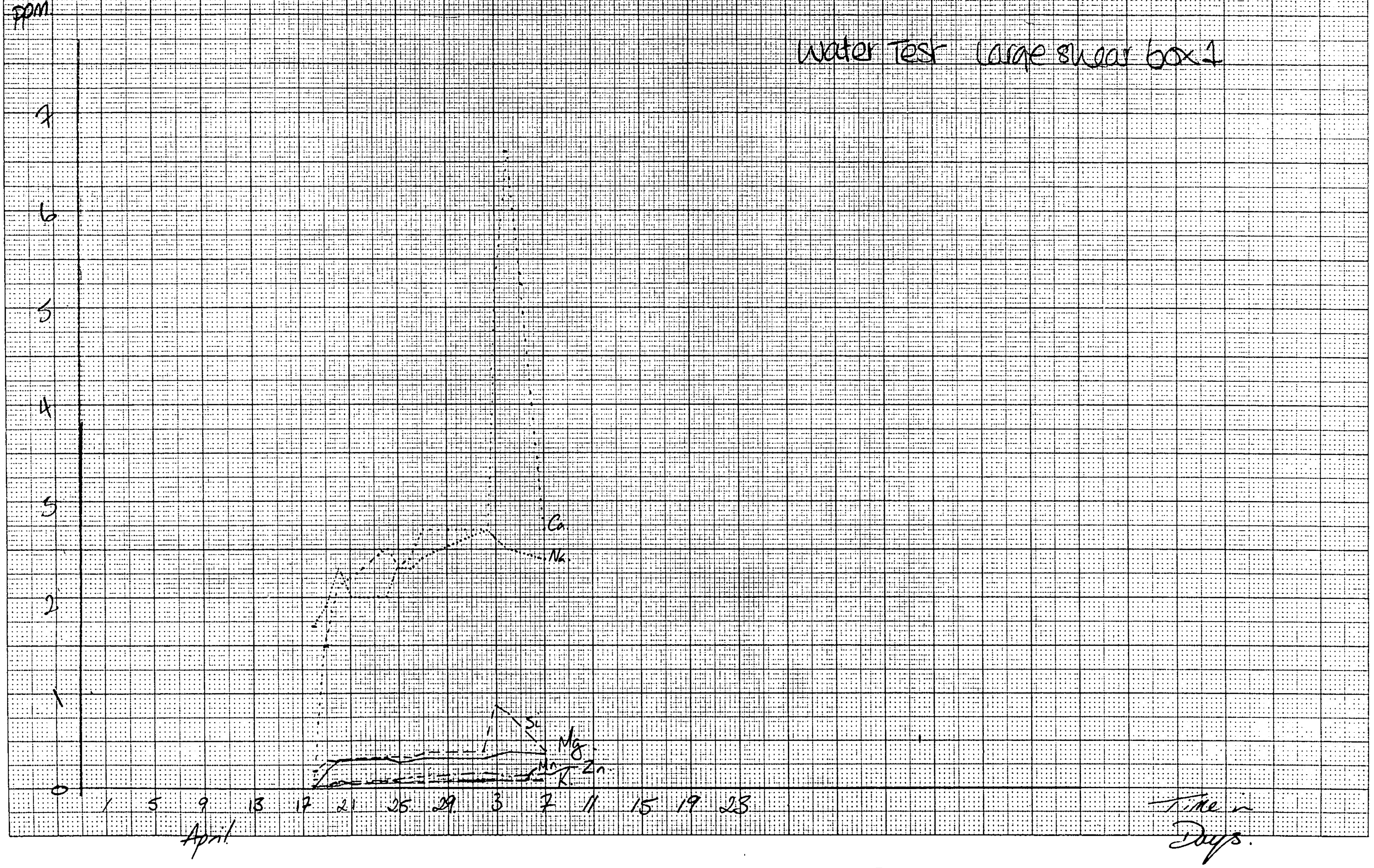
Sample	Mineral (%)								
	Mt	Chl	Ill	Gyp	Kaol	Qtz	Calc	Pyr	Misc
CD1	-	0.6	27.8	1.4	26.6	17.9	-	-	25.7
CD2	-	0.3	23.0	-	15.3	10.8	-	-	50.6
CD3	-	0.1	18.5	0.2	23.2	26.8	-	-	31.2
CD4	-	0.5	21.7	0.5	19.4	23.7	-	-	34.2
CD5	-	-	35.6	-	27.4	4.3	-	-	32.7
CD6	-	0.3	42.0	-	20.3	20.3	-	-	17.1
CD7	-	0.4	38.0	-	31.2	16.8	-	-	13.6
CD8	-	0.4	33.8	0.6	27.4	27.0	-	7.1	3.8
CD9	-	0.3	29.5	0.3	20.8	14.7	-	-	34.4

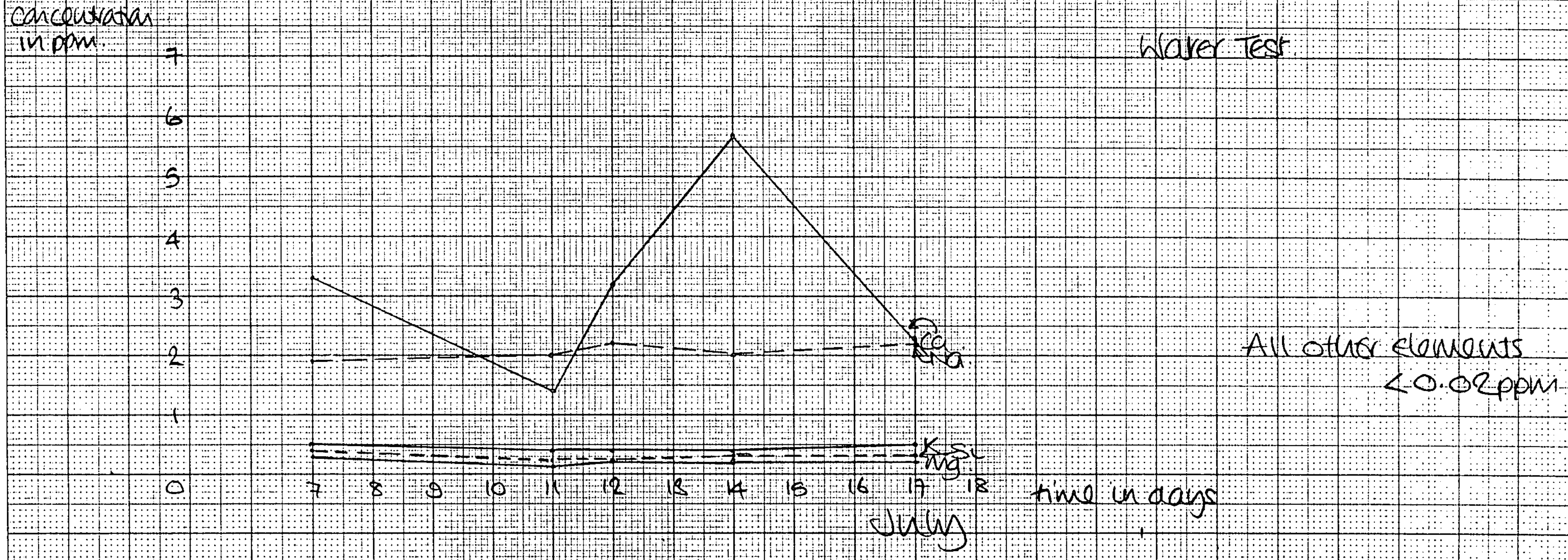
APPENDIX V

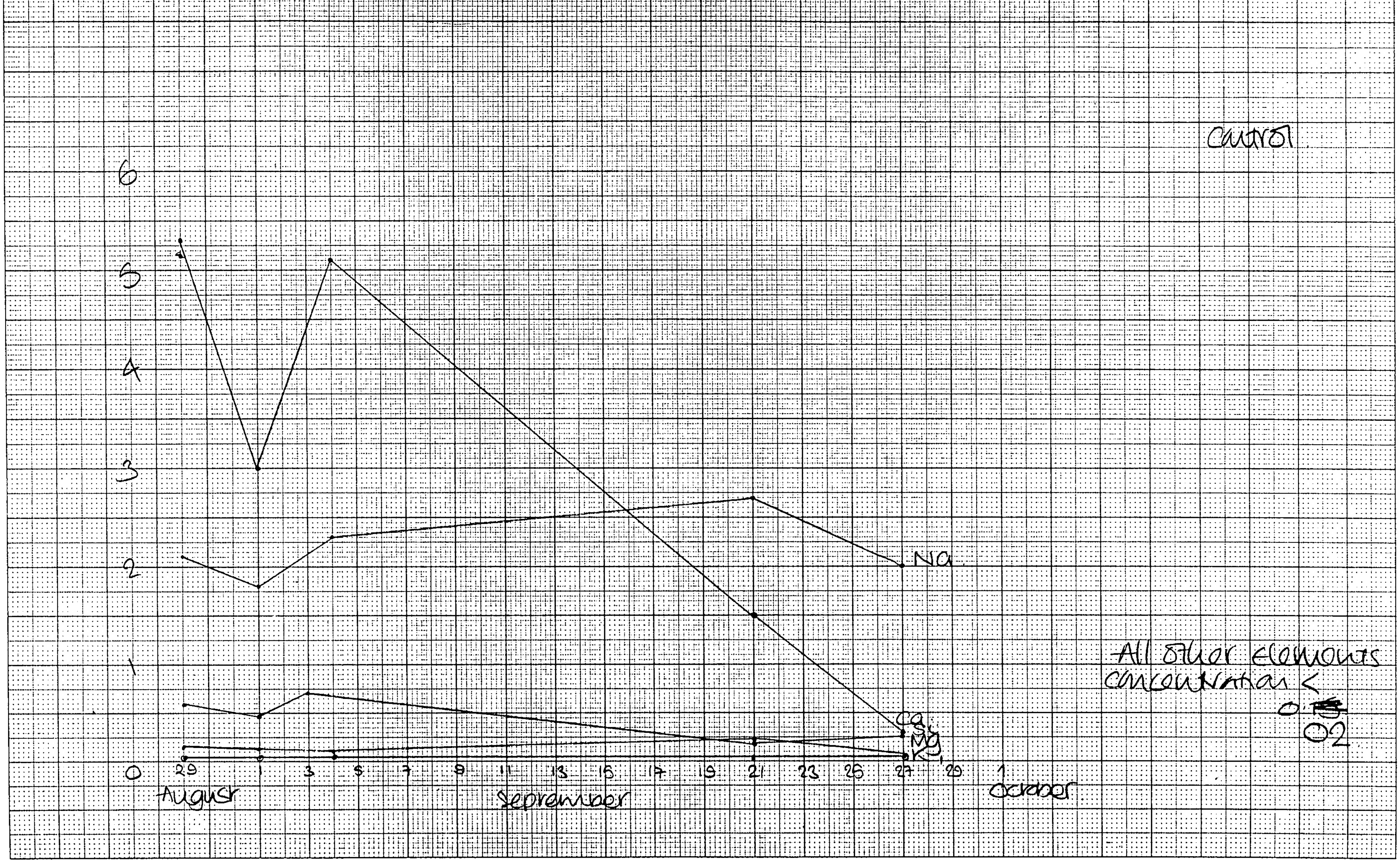
Results of AAs and ICP-AS



SiO_2 shaded ∵ machine is unstable & won't settle to unique value. ∴ range is quoted

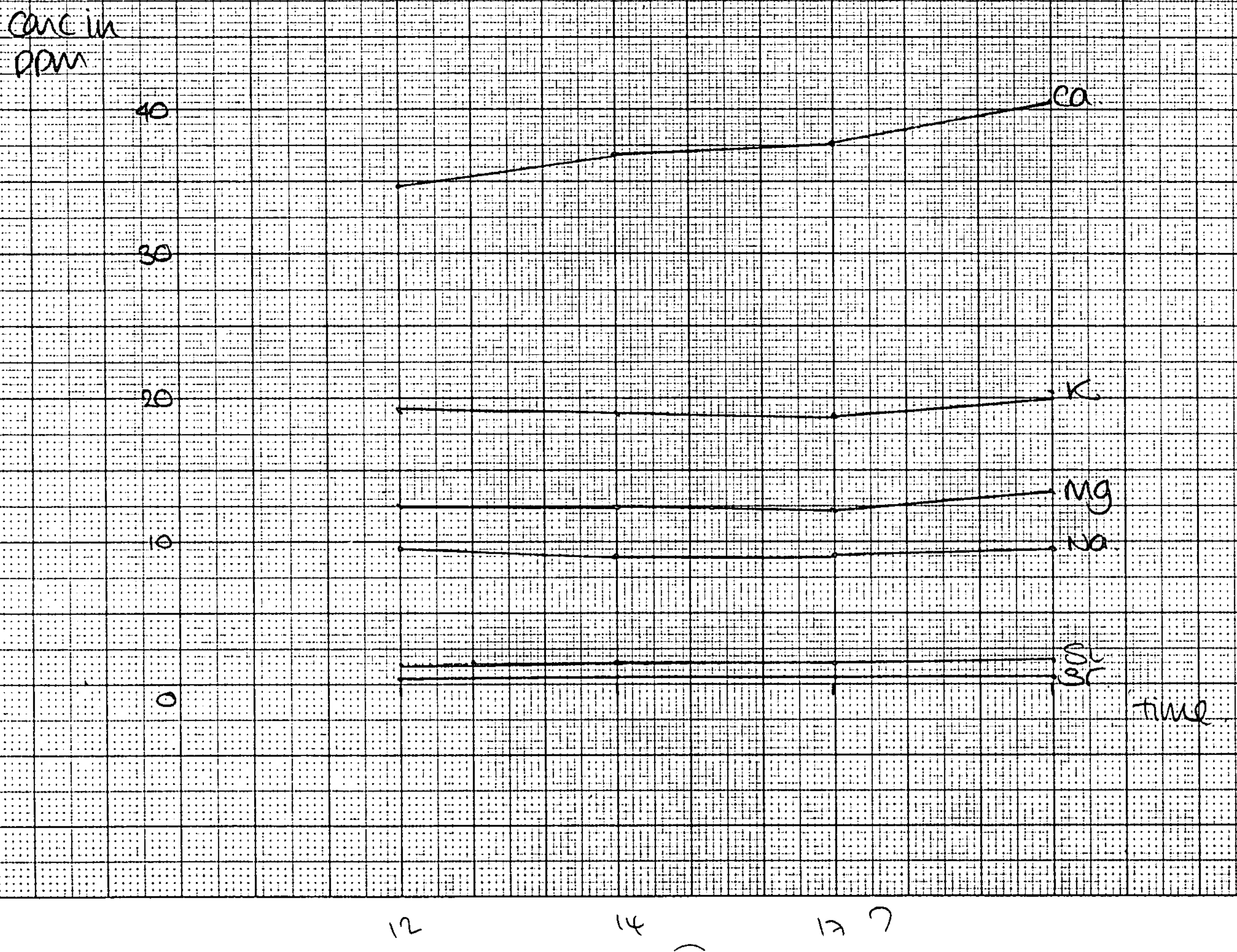

Water Test Large shear box 1

Water Test
concentration in ppm.
7
6
5
4
3
2
1
0
7
8
9
10
11
12
13
14
15
16
17
18
time in days
July
Ca
Na
K
Si
Mg
All other elements
<0.02ppm


Control
6
5
4
3
2
1
0
29
1
3
5
7
9
11
13
15
17
19
21
23
25
27
29
1
August
September
October
Na
Ca
Si
Mg
K
All other elements
concentration <
0.~~15~~
02

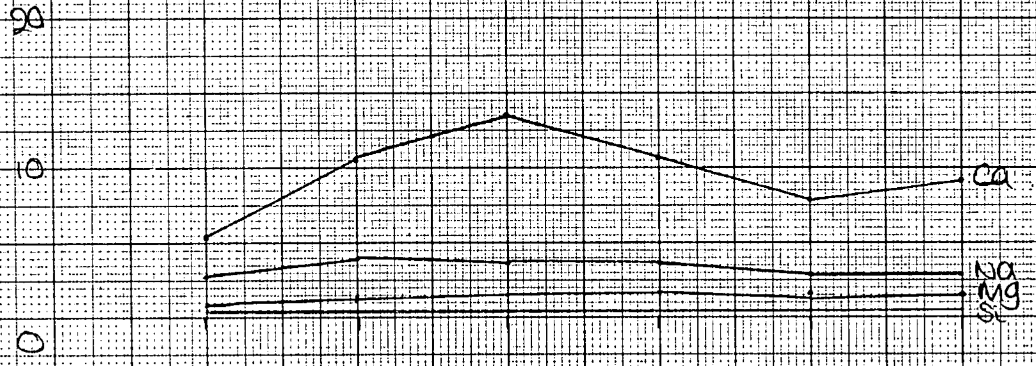
Started July 12th

Coal Measures



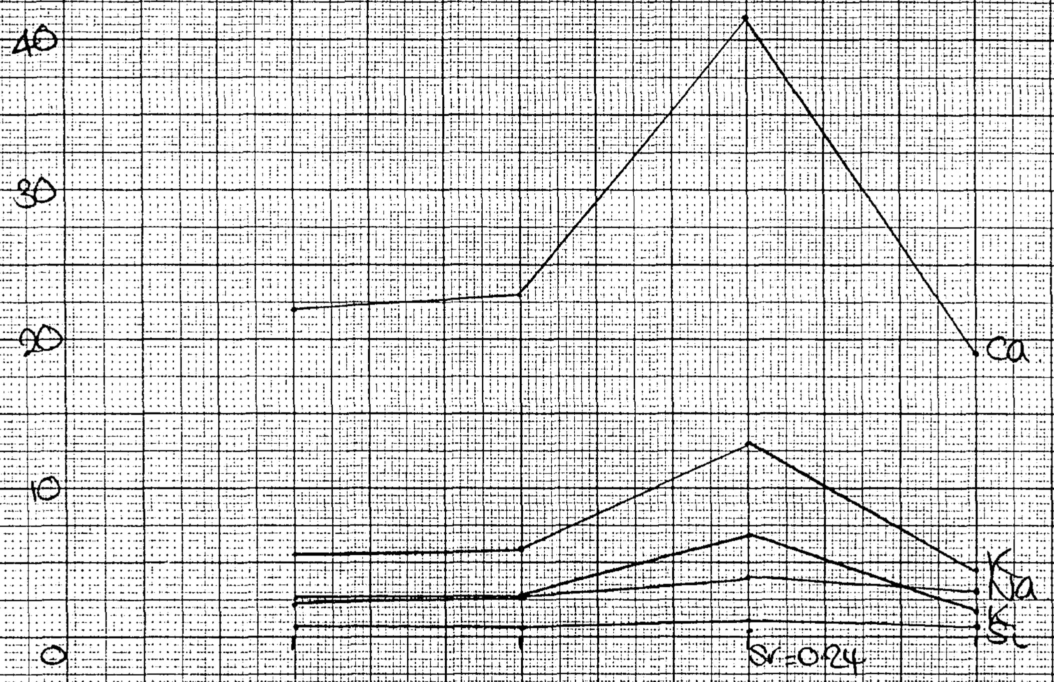
All other elements
<0.10 ppm
& constant
(Ba 0.12)

Burnhope



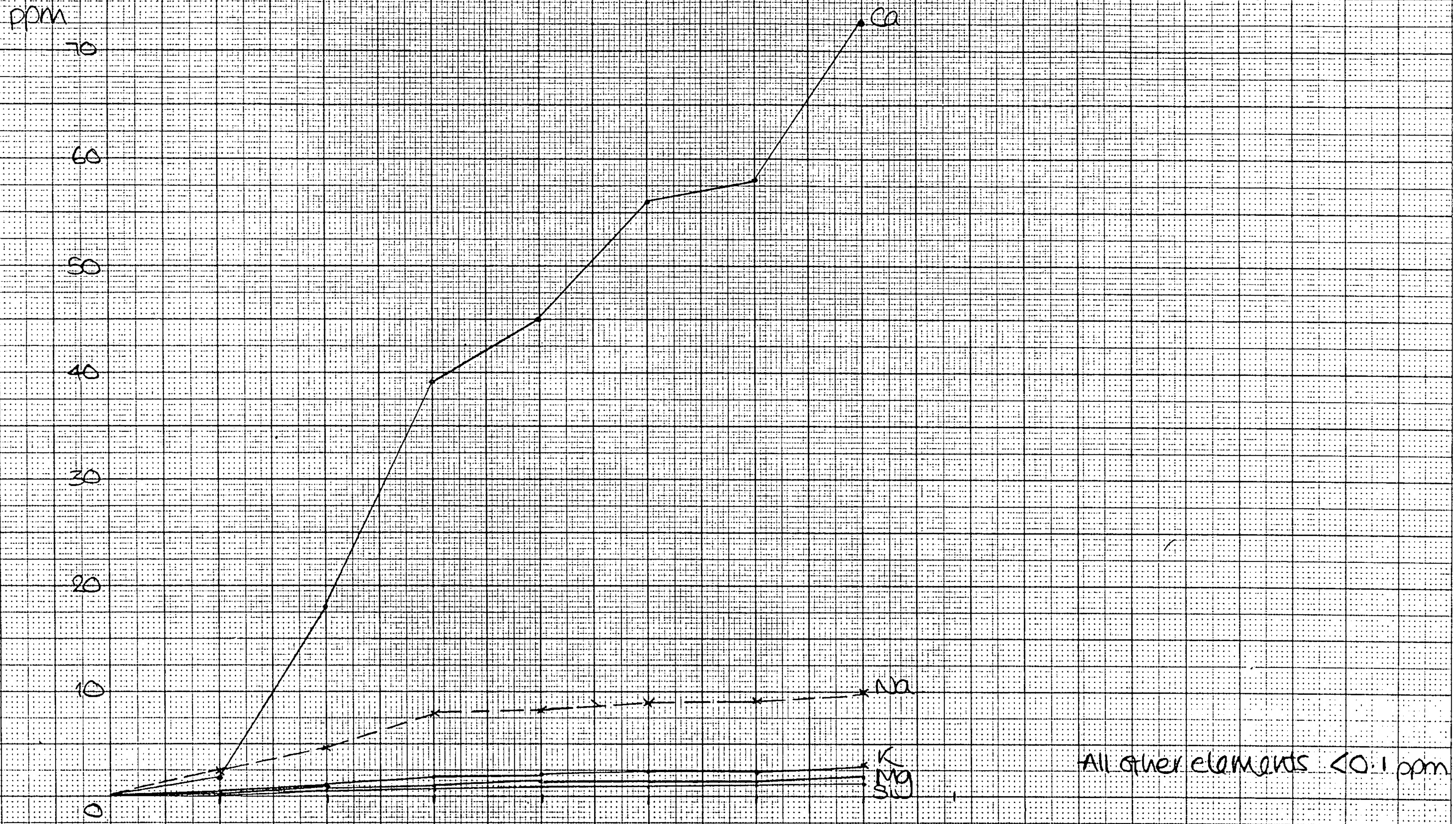
K: 0·8 – 1·0 ppm

"IS"



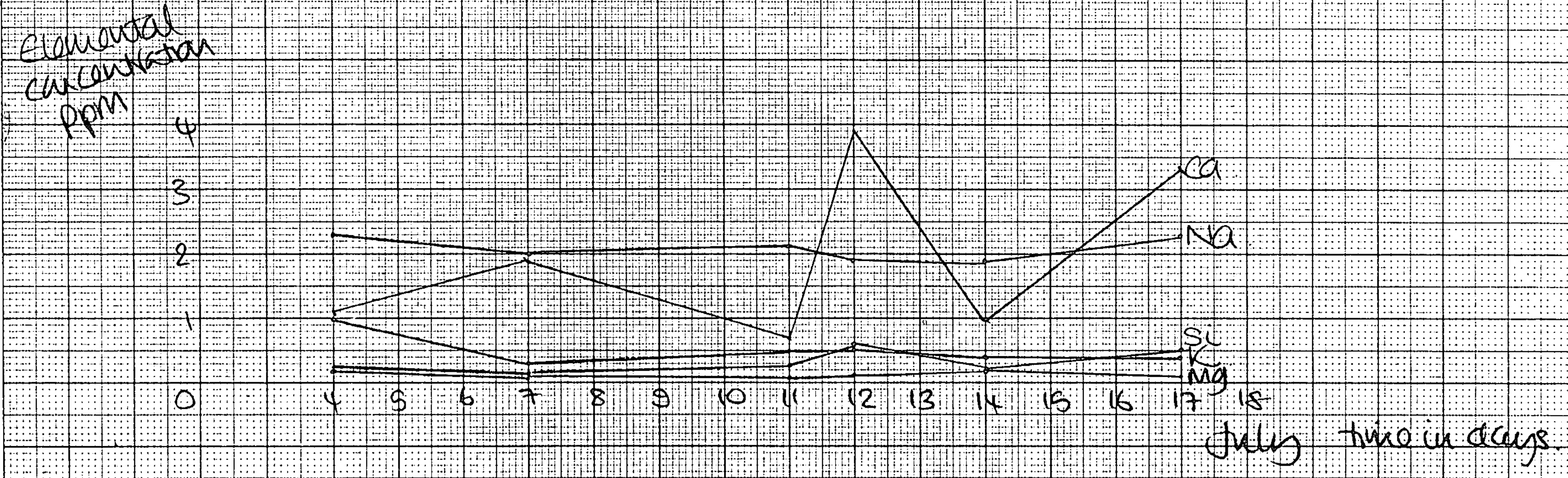
All other elements
<0.10ppm

KCLSB



All other elements <0.1 ppm

Control


Elemental concentration ppm
4
3
2
1
0
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
Ca
Na
Si
K
Mg
July time in days.